THE IMPROVEMENT OF THE LOWER MISSISSIPPI RIVER
FOR
FLOOD CONTROL AND NAVIGATION

Prepared under the Direction of
BRIGADIER GENERAL T. H. JACKSON
President Mississippi River Commission

By
D. O. ELLIOTT
Major, Corps of Engineers

IN THREE VOLUMES

VOLUME II

U. S. Waterways Experiment Station
Vicksburg, Mississippi
May 1, 1932
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CHAPTER VI.

LEVEES.

INTRODUCTION.

A levee may be defined as an earthen embankment extending generally parallel to the river channel and designed to protect the area behind it (the levee) from overflow by flood waters. To accomplish this purpose, the levee must be high enough to be secure against overtopping by flood stages, and broad enough to be secure against destruction by seepage or piping. The levee or dike system for the control of floods is probably as old as recorded history. Levees were built along the Nile River about four thousand years ago. In times of remote antiquity the Tigris and Euphrates Rivers were confined by levee lines. At the dawn of the Renaissance in Italy, levees extended for many miles along the River Po. In fact, levees have been constructed on nearly every important river in Europe and Asia which is subject to flood. Noteworthy levee systems exist along the Rhone, the Danube, the Vistula, the Rhine, the Po, the Arno, the Volga, and the Yellow Rivers.

The levee system is the only method which can be used alone to solve a major flood control problem. The use of any other method of flood control, except as an aid to a levee system, has been unusual. Levees are not a panacea for the alleviation of flood evils. Their use is, however, usually successful to the extent that scientific methods and intelligent planning are employed in their location and construction.

The present chapter is confined to a discussion of the levee system of the Lower Mississippi River. No attempt is made to discuss foreign levee practice or levee practice elsewhere in the United States. In the following pages, frequent reference is made to Mississippi River floods and their effect upon the levee system. A complete summary of recorded floods appears in Chapter III.

THE DEVELOPMENT OF THE LEVEE SYSTEM TO 1879

The need for levees was recognized by the first French settlers in the Alluvial Valley. The City of New Orleans was founded by Bienville in 1717. De la Tour, an engineer who accompanied him, is said to have opposed the location of the city on the site selected by Bienville on the grounds that the new settlement would be periodically flooded by the river. Bienville, however, overruled this objection and De la Tour thereupon undertook the construction of a riverside levee to protect the settlement. This work was not completed until 1727. It was three feet high; 5,400 feet in length; 18 feet wide at the top; and carried a roadway on its crown. The extension of the levee system kept pace with the growth of settlements along the lower river. By 1735 the levee lines on both sides of the river extended from about 30 miles above New Orleans to a point approximately 12 miles below the city. (See Plate XXXII.) The expense of the construction of this system was borne by the riparian landowners over whose property it extended. The works were of insufficient strength and were crevassed at many points by the unusually high water of that year. In 1743 an
ordinance was passed by the French Colonial Government requiring landowners to complete their levees by January 1, 1744 under penalty of forfeiture of their lands to the French Crown. By the year 1812 the levee system extended up the river to Baton Rouge on the east bank and to the vicinity of Morganza (40 miles above Baton Rouge) on the west bank. The notable flood of 1828 caused much damage to this system. By 1844 the west bank levees extended in an almost continuous line as far north as the Arkansas River; while on the east bank many isolated levees had been constructed to protect land in the Yazoo Basin. A few isolated levees also existed on the west bank above the Arkansas. (See Plate XXXII.) In this year (1844) a disastrous flood occurred which overflowed the unprotected basins of the St. Francis and Yazoo Rivers. The weak levees on the Tensas front crevassed and allowed that basin to be flooded. Below the mouth of Red River, little damage was done as the stage in Red River was fortunately low during the flood stages on the main stream. Another flood occurred in 1849 which was the most destructive ever known up to that time. The St. Francis, Yazoo, and Tensas Basins were again overflowed, while below Red River numerous crevasses occurred. An east bank crevasse about 18 miles above New Orleans caused the inundation of the city. A flood occurred during the next year (1850) which again flooded the side basins. The depths of the overflow in the Tensas and Atchafalaya Basins were greater in 1850 than in any year since 1828. Floods in the Arkansas, Red, and Black Rivers aggravated conditions on the Lower Mississippi. It is said that west bank crevasses below Red River discharged into the Atchafalaya Basin for more than four months. A crevasse occurred at Bonnet Carre' (38 miles above New Orleans) which is said to have attained a width of 7,000 feet and to have continued flowing for more than six months.

These disastrous floods created widespread distress in the valley and were followed by the so-called swamp land acts which were passed by the National Congress in 1849 and 1850. The first of these acts granted to the State of Louisiana all swamp and overflowed lands within its boundaries which by reason of their wetted condition were unfit for cultivation. These lands were to be sold by the state and the proceeds used for the construction of levees and drainage works for the reclamation of the lands themselves. The second swamp act extended these benefits to the several states with certain specified exceptions. The provisions of this act were similar to those of the first swamp act. A further discussion of the swamp acts will be found in Chapter XI. Pursuant to passage of the swamp acts, the States of Louisiana, Mississippi, Arkansas, and Missouri organized offices for the sale of the swamp lands, and appointed commissioners to locate and construct the levee lines. Lack of coordination between states and between levee districts, however, prevented the construction of effective levee lines. As a flood protection measure, the swamp acts must be classed as a failure.

During the period 1850-1858 the levee system reached its greatest extent and highest efficiency prior to the creation of the Mississippi River Commission. The levee system as it existed in 1858 is described by Humphreys and Abbot in the report of the Delta Survey. (See Plate XXXIII.) This description may be summarized as follows: on the west bank, the St. Francis levees had been built after 1851. They were weak structures averaging only 3 feet in height. Numerous gaps
aggregating 25 miles in length existed in this line in 1858. On the White River front, the levee line was complete to a point 18 miles below Helena. From that point to the mouth of the White River, gaps in the levees aggregated 14 miles in length. Between the mouth of the Arkansas and Cypress Creek (24 miles below) there were no levees. Below Cypress Creek the west bank line was continuous nearly to the mouth of Red River. Below Red River the west bank line extended to a point about 45 miles below New Orleans. On the east bank, there were no levees of any importance above the Mississippi-Tennessee State line. The Yazoo Basin was covered by a line of weak levees about 4 feet high which had been built since 1853. This line was intact except for one small gap nearly opposite Helena, which had been caused by caving. Between Vicksburg and Baton Rouge some small levees had been built for the protection of bottom lands between the river and the bluffs. Between Baton Rouge and Pointe a la Hache (45 miles below New Orleans) the levees were complete.

Humphreys and Abbot measured a number of levee sections and from these measurements determined the following mean levee dimensions as shown in Table XXIX.

**TABLE XXIX**

**MEAN LEVEE DIMENSIONS 1851**

<table>
<thead>
<tr>
<th></th>
<th>West Bank</th>
<th>East Bank (between Baton Rouge and Carrollton)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sections Measured</td>
<td>38</td>
<td>21</td>
</tr>
<tr>
<td>Top Width</td>
<td>4.7 feet</td>
<td>4.0 feet</td>
</tr>
<tr>
<td>Base Width</td>
<td>14.6 &quot;</td>
<td>10.5 &quot;</td>
</tr>
<tr>
<td>Height</td>
<td>4.7 &quot;</td>
<td>4.3 &quot;</td>
</tr>
<tr>
<td>Freeboard above 1851 high water</td>
<td>1.4 &quot;</td>
<td>1.0 &quot;</td>
</tr>
</tbody>
</table>

Although these levees were very weak, Humphreys and Abbot mention several massive structures. One short levee line near the northern boundary of Crittenden County, Arkansas, was 40 feet high, 40 feet wide at the top, and 320 feet wide at the bottom. The levee across the Yazoo Pass about 5 miles below Helena, was 1,152 feet long, 28 feet high, and had a base width of 300 feet.

In 1858 a disastrous flood occurred. Miles of levee were swept away by the overflow of the Upper St. Francis Basin and by return flow into the Mississippi above the mouth of the St. Francis.

During the Civil War period, the levees were not kept in repair. The flood of 1862 attained heights greater than those of 1858 below Cairo, while the flood of 1865 was almost as high as that of 1858 between Memphis and Vicksburg. These two floods caused widespread damage to the levee lines and overflowed wide areas in the lower valley. In December, 1865, General A. A. Humphreys, an engineer officer, was
directed by the Secretary of War to make an examination of the levee system with the view to repair where urgently needed. He estimated that the repair and completion of all existing levees would necessitate 9,750,000 cubic yards of earthwork at about 40 cents per yard. No action was taken by the Federal Government to rebuild the levee system as a result of General Humphreys' report. The local interests did not possess the means to make repairs to the levee system. Little re-construction was undertaken.

The flood of 1867 made a further disastrous attack upon the levee lines, which were at that time in much worse condition than in 1858. In addition to unclosed gaps caused by previous floods, many new crevasses occurred. In Louisiana, four old crevasses were reopened and twelve new ones occurred. After this flood, little or no repairs were effected and the flood of 1874 caused still further damage to the levee lines. After this flood, the aggregate lengths of west bank crevasses were: 23.9 miles between Commerce and New Madrid, 68.2 miles between New Madrid and Helena, 20.5 miles between Helena and the mouth of White River, and 23.9 miles between the Arkansas River and the Louisiana line. In the State of Mississippi crevasses totalled 2.2 miles in length, while in Louisiana crevasses had a total length of 4.7 miles. (See Plate XXXIV.) As a result of the 1874 flood, a board of engineers was created to prepare a flood control and reclamation plan. This board was known as the "Levee Commission" and was composed of General G. K. Warren, Major H. L. Abbot, Captain W. H. H. Ben-yaund, Mr. J. E. Sickels, and Mr. P. O. Hebert. The report of the Levee Commission is found in the Annual Report of the Chief of Engineers for 1875. The board found the existing levee system defective as a result of five principal causes, namely; vicious organization; insufficient grades; poor construction and injudiciously selected cross sections; inadequate arrangements for inspection and guarding; and faulty location. Between 1866 and 1874, caving banks in Louisiana alone had destroyed an aggregate of 107 1/2 miles of levee line.

The Levee Commission prepared two distinct estimates for the protection of the valley by levees. One estimate contemplated merely the closure of the breaks in the existing levee system, and totaled about 8,066,000 cubic yards of earthwork at an approximate cost of about $3,460,000. The Levee Commission pointed out, however, that this plan was unsatisfactory, since caving banks would continue to breach the line. The second estimate made by the Levee Commission contemplated a permanent levee system. The estimate called for about 114,774,000 cubic yards of earthwork at an approximate cost of $45,-910,000. The levee grades proposed by the Levee Commission are shown in Table XXXIII. Nothing was done by the Federal Government to effect the repairs recommended by the Levee Commission.

Between 1874 and 1882, only one serious high water occurred (1881) and all levee lines except those in the St. Francis Basin were repaired. In the year 1878 a board of engineers consisting of Colonel J. G. Barnard, Colonel Z. B. Tower, Lieut. Colonel H. G. Wright, and Majors C. B. Comstock and C. R. Suter, all Corps of Engineers, was appointed to report upon the improvement of the low-water navigation in the Mississippi and Missouri Rivers. This board subsequently received instructions to consider the effect of a permanent levee system
on the Lower Mississippi both for navigation and flood control. The report of this board was submitted in January, 1879, and is contained in the Annual Report of the Chief of Engineers for that year. The board concluded that a complete levee system would be an aid to high-water navigation, but would have little or no influence upon navigation at low stages. The board further remarked that the greatest obstacles both to the improvement of navigation and to the maintenance of levees were the instability of the river and the caving of its banks. The board further concluded that the levee system if undertaken should be matured and developed in connection with navigation improvement.

As has been stated in Chapter I, this report is noteworthy for two reasons: First, the board considered flood control and navigation as parts of the same problem; and second, the board stated that the levee system had no influence on low-water navigation. These two conclusions are significant. Shortly after the creation of the Mississippi River Commission in 1879, Congress adopted a policy of limiting the expenditure of Federal funds for levee construction to such work as was a part of the program for the improvement of navigation. The policy which considered levees as part of the navigation improvement program enabled the Commission to carry on levee construction. The pronouncement of the board of 1878 concerning the influence of levees upon low-water navigation is a clear and correct statement covering a somewhat controversial subject.

The first period of levee development ends with the creation of the Mississippi River Commission. This period witnessed the beginning of the Mississippi River levee system with the construction of the first levee at New Orleans, and the gradual extension of that system through the cooperative efforts of interested landowners. Subsequent extensions took place under successive stages of local, county, and state control. During this period, however, levees were deficient in both height and section. They were built but little above the grade of previous confined floods. The extension of the system increased flood heights and made higher levees necessary. The system was in a process of growth to a grade and section which would restrain a confined flood. This period witnessed the initiation of efforts to standardize levee practice and studies were also made to determine probable flood heights. However, no effective organization as yet existed to plan and execute any project for the complete protection of the lower valley. Nevertheless, during the latter part of the period, national interest was gradually aroused. The swamp acts were an abortive attempt by the United States to solve the flood problem. Exhaustive studies had been made. The time was ripe for action by the National Government.

**THE DEVELOPMENT OF THE LEVEE SYSTEM SINCE 1879.**

The creation of the Mississippi River Commission by the act of June 28, 1879 marked the beginning of a new period of levee development. The Federal Government now appeared as an active agent in the control of the river. The Commission found the local inhabitants already organized into levee districts, empowered by state laws to locate and construct levees for their own protection. These local interests had accomplished something, but their efforts lacked coordination and
no real flood protection had as yet been provided. After the organiza-
tion of the Commission, the local districts provided rights of way for
Government work and also continued the construction of levees outside
the zones of Commission activity.

The Mississippi River Commission submitted its preliminary report
on February 17, 1880 (published in the Annual Report of the Chief of
Engineers for 1881). This report asserted that, during floods, the
levees operated to deepen the channel and enlarge the bed of the river.
It further stated the belief of the Commission that the river had been
better for navigation during the period 1850-1858 than at any subse-
quent time. This period marked the greatest development of the levee
system prior to 1880. The Commission was of the opinion that, while
the levees were not essential for navigation, nevertheless the repair and
maintenance of existing levee lines would hasten channel improvement.
In short, the levees were regarded as desirable though not necessary ad-
juncts to the navigation improvements. As a protection against de-
structive floods, the Commission stated that levees were of course es-
sential.

The views expressed above were not unanimous and two dissenting
Commission members submitted a minority report. This minority
statement called attention to the fact that stage rises were usually fol-
lowed by building up of crossing bars. The creation of abnormally
high flood planes by the construction of a levee system might possibly
increase the amount of this rise in the bottom thus injuring, instead of
improving, navigation. The dissenting members further asserted that
navigation difficulties were occasioned by excessive low-water channel
widths, a condition which levees on top of the river banks could not
affect in any way. They concluded that levees were of very little value
in improving the low-water navigation, although they recommended
the closure of gaps in the existing levee line as a flood control measure.

Further study of the levee question was undertaken by a Committee
on Levees and Outlets which was created by Commission resolution on
July 9, 1880. The findings of this committee appear as an appendix
to the report of the Commission dated January 8, 1881 and is contained
in the Annual Report of the Chief of Engineers for 1881. This com-
mittee voiced the opinion that the existence of outlets together with
the abandonment of local levees had acted to raise the river bed in the
lower reaches of the Alluvial Valley. The levee system was, on the
other hand, considered a direct influence for the improvement of navi-
gation; the prevention of destructive floods; and the elimination of
varying and abnormal local conditions. These findings were substan-
tially in accord with the preliminary report of February 17, 1880. It
is worthy of note that, from the phraseology of its report, the committee
apparently considered outlets and levees as alternative methods of
river control rather than as measures capable of simultaneous utilization
in the same plan for improvement.

The river and harbor bill of March 3, 1881 made the first appropri-
ations for the actual construction of improvement works by the Com-
mision. This bill specifically provided that no funds should be ex-
pended upon levees for any purpose other than that of deepening or
improving the channel of the river. This provision was in effect a
statement of national policy regarding Mississippi River levees. They
were henceforward to be constructed as navigation works rather than works designed for flood protection. All Members of the Commission were agreed that levees tended, in some degree, to increase the scouring and deepening powers of the current. They were not, however, of one opinion as to the extent of this influence on the low-water channel, or as to its value compared with other methods of improvement. These questions were regarded as requiring further observation and study before final conclusions regarding them could be reached.

The year 1882 was characterized by a flood of great severity. A total of 284 crevasses occurred in the levee lines. These crevasses had a combined length of 56.1 miles.

In 1882 the Commission finally adopted a levee policy which was expressed in the project for levee construction outlined in the Annual Commission Report of December 1, 1882. This policy was based on the theory that the confinement of flood waters would periodically flush out the channel thus removing obstructing bars and preventing the formation of new obstructions. The levee project itself may be summarized as follows: The Commission recommended the construction of levee lines with a grade sufficient to confine the most frequent floods. These levees were regarded as a means of securing channel improvement for navigation at a minimum cost. The restraint of abnormal floods would, it was thought, involve a cost disproportionate to the injury which they might cause to the channel. The extent to which the levee grades were to be raised was to be determined by considerations of economy. At certain localities such as outlets and depressions of the bank, where levees would be costly and liable to destruction, a massive section and a grade higher than the greatest floods was recommended. Similar sections and grades were recommended for levees at the heads of the great basins, where the destruction of levees would cause great loss. The Commission further recommended the extension of the levee system upstream rather than downward. This upstream extension would, it was felt, have less tendency to produce a temporary increase in flood heights than would the downstream extension of the lines. The Commission further contemplated the closure of new breaks before the old gaps were repaired. This decision was based on the presumption that the new breaks would inflict new injuries on navigation while the ultimate effects of the older breaks had already been reached. It was furthermore believed that greater channel improvement would be attained by the greatest possible development of the levee lines at existing grades rather than by the construction of short lines of great height and cross section.

Under the policy of 1882, the work of joining unconnected lengths of levees into a continuous line was begun in the fall of 1882 and was continued until the spring of 1883 when operations were suspended by the flood of that year.

After the subsidence of the 1883 high water, levee work was pushed ahead rapidly until the following year when another flood again caused a cessation of work.

The floods of 1882, 1883, and 1884 demonstrated the necessity for raising and strengthening the levee lines below Red River. They effectively disproved the previously held belief that the construction of levees below Red River had been followed by a depression of the flood plane.
The Annual Report of the Mississippi River Commission for 1884 stated that the enlargement of levees below Red River was necessary notwithstanding the fact that navigation was unobstructed in that reach. This statement was based on the belief that the construction of levees as an aid to navigation in the reaches above Red River could not proceed indefinitely without involving damage below. The levees below Red River were thus considered an indirect aid to navigation in the reaches above.

The flood of 1886 prevented any new work during that year and caused numerous small breaks in the west bank levee line within a distance of about 10 miles north of Arkansas City. These breaks were due largely to insufficient levee grades. With these exceptions, however, all Government levees remained intact. The levee system as it existed in 1886 is shown on Plate XXXV.

In the year 1888 the Commission recognized the necessity for higher levee grades but before any active steps could be taken to increase them, the flood of 1890 caused a total of 53 crevasses with an aggregate length of 6.88 miles. The river and harbor act of September 19, 1890, which was passed after this flood, marked a temporary change in policy. This change lay in the omission from the act, of the provision which restricted levee construction to that necessary for navigation improvement. The Commission was thus relieved from the necessity of justifying all levee expenditures as necessary for navigation improvement. The joint resolution of March 3, 1891, however, replaced this restriction upon levee construction.

The flood of 1890 was adopted as the flow line to which levee grades were to be referred. This necessitated the raising of completed sections of the levee line. The Mississippi River Commission Report for 1891 contained the new grades adopted for the several levee fronts. There was a lack of uniformity in these grades which might at first glance connote a lack of study. This was not, however, the case. Levee grades were referred to the actual flood height rather than to the estimated confined height of the flood. The actual height was of course affected by the crevasses. To allow for this effect, the levee grades were raised in the vicinity of the crevasses to elevations deemed adequate to contain the fully confined discharge of the river.

The floods of 1892 and 1893 were similar in many respects. Between Arkansas City and Vicksburg, and below the mouth of Red River, these floods approached the maximum stages recorded before 1892. These great flood heights were the result of the improved levee lines along these reaches. The crevasses in 1892 had an aggregate length of about 2.3 miles. During the 1893 flood, there were but 7 crevasses with an aggregate length of about 1.8 miles. The report of the Commission for the year 1893 announced a policy designed to furnish reasonable protection to the greatest possible area. This policy contemplated the closure of all small breaks as soon as possible after their occurrence in order to maintain the continuity of the maximum possible length of levee line. These lines should be made strong enough to resist ordinary high waters even though they might not furnish absolute protection against the highest floods. The Commission also recognized the fact that the progressive extension of the levee system would inevitably be followed by increased flood heights.
In the year 1897 a destructive flood occurred. At this time the levee lines below Cairo aggregated 1,377 miles in length. The flood caused 37 crevasses, aggregating 8.7 miles in length and flooded an area of 10,667 square miles including backwater areas. The crevasses were all due inherently to low grades and weak cross sections, or to old, imperfect, or uncompleted construction. For reports on this flood by the engineers of the local levee districts, the reader is referred to the Annual Report of the Mississippi River Commission for 1897.

The Annual Report of the Mississippi River Commission for 1899 contains a statement of the principles which governed levee construction at that time. The Commission recognized the obligation of giving a substantially equal degree of security against overflow to each of the basins and districts. The allotment of Government funds for levee construction had been based upon this intention with such modification as economy and financial considerations made advisable. The Commission stated it was reasonable that a large, densely populated and cultivated basin should be afforded greater security than a small, unimproved one.

In 1903 the levee system was again subjected to a flood test. The flood of that year was within 1.57 feet of the highest known stage at Cairo. The volume of this flood was largely increased by the St. Francis, White, Arkansas, Yazoo, and Red Rivers whose flood stages synchronized with that on the main river. Stages were higher at Carrolton than at any previous time. Notwithstanding these great flood heights, there was a very large reduction in the number of crevasses whose aggregate length was 2.2 miles. At Bougere (26 miles above the mouth of Red River), 9,300 feet at the end of the levee line were destroyed, however. The total area submerged, including backwater overflow, was 7,954 square miles.

The extension of the jurisdiction of the Mississippi River Commission for the purposes of levee construction now became necessary. Before the creation of the Commission, the levee lines on the St. Francis front had occasioned considerable trouble. These lines had always been weak and the flood of 1858 had left them in a ruinous condition. Their repair had been slow and the Commission was for some time in doubt as to the wisdom of protecting the St. Francis Basin at all. By 1892, however, the Commission had allotted funds for the construction of levees in the Lower St. Francis District, thus definitely adopting the policy of protecting the St. Francis front. There was some uncertainty, however, as to the ultimate wisdom of this work which deprived the river of a natural flood reservoir basin. Widely divergent views were advanced. The reader is referred to the Annual Report of the Mississippi River Commission for 1895 in which is published papers by Col. Charles R. Suter, and Captain C. McD. Townsend bearing on this subject. Once embarked upon this policy, however, the Commission realized that its jurisdiction, which then terminated at the mouth of the Ohio River, must necessarily be extended to the real head of the Alluvial Valley at Cape Girardeau, Mo. This necessary extension of jurisdiction was granted by the act of June 4, 1906.

With the extension and development of the levee lines, an important problem arose for solution. This was the question of replacement of the increasing yardage lost annually through the destruction or
abandonment of existing levee lines. Since 1882, the levees built by the United States had been planned for a probable effective life of at least 20 years. In practice the average life of these levees exceeded that figure. Many lines constructed by local interests had, however, a much shorter life. The extent of the levee losses due to bank caving, crevasses, etc., is illustrated by the following: during the eleven-year period 1900-1910 (both years, inclusive), 52,225,000 cubic yards were placed in the levees by the United States, and 72,643,000 cubic yards by private interests. During this same period, the total amount of levee lost or abandoned aggregated 26,911,000 cubic yards or about 21 1/4 per cent of the total yardage placed. The Annual Report of the Mississippi River Commission for 1906 contains a discussion of this subject. By 1910, the Commission had gone on record as advocating the protection of levee lines which were themselves justified as aids to navigation. This protection consisted in general of bank revetment to prevent loss by bank caving. As will be seen later in this chapter, however, the cost of bank protection is so great as compared to levee construction that its use to prevent levee setbacks is uneconomic. The proper solution of the problem lies of course in the proper location of levee lines. The probable life of main river levee lines is at present (1931) fixed at 30 years. Revetment is now used only when the retirement of the levee line is impracticable.

A disastrous flood occurred in the year 1912. The high water culminated at Cairo on April 6 and developed a protracted flood through the lower valley extending through April and May. Fifteen crevasses occurred between Cairo and New Orleans. The aggregate length of levee line destroyed was about 6 miles. When this flood occurred, 587 miles of levee were still below the grades adopted by the Mississippi River Commission. In other words, above Vicksburg, 47 per cent of the levees under Mississippi River Commission jurisdiction were below grade, while below Vicksburg, this percentage was only 26 per cent. Only 2 of the 12 major crevasses occurred below Vicksburg. The levees on the St. Francis front suffered heavy damage. In fact, 58 per cent of the levee destruction occurred on this front. This was not surprising as the St. Francis levees were totally inadequate. This flood emphasized the necessity for an increase in levee grade and section. The Annual Report of the Commission for 1912 stated that concrete revetment (for the riverside slope) and sheet piling (for foundation protection) might become necessary parts of levee construction in unfavorable locations.

The following year, 1913, a flood occurred which broke the records established by the flood of the previous year. The levees, badly damaged by the previous flood, were further breached and great damage to the valley resulted. Forty-five crevasses occurred, the aggregate length of which was 5.3 miles.

As a result of the flood of 1913, a new levee grade and section were adopted in 1914 and remained generally in force until superseded by the 1928 levee standards. (See Table XXXI.) The development of the levee section is illustrated by Plate XXXVIII. The 1914 levee dimensions are discussed later in this chapter. The former policy of working upstream from the lower end of the basins was abandoned in 1914. The reasons for this change of policy are explained in the Annual Report of the Commission for that year. This report stated that
the controlling lines from the head of the St. Francis Basin southward were now in such condition as to give protection from moderate floods. Further work in cooperation with states and local interests should therefore, as far as practicable, begin at or near the upper ends of the several basins and progress downstream along sections of essentially permanent alignment.

By the act of March 4, 1913, the Commission's jurisdiction over levee construction was extended to Rock Island, Ill. Above Cape Girardeau the valley of the Mississippi River is generally less than 5 miles in width and nowhere greater than 10 miles in width. Compared with the lower river, the flood heights above Cape Girardeau are low. The upper river levee system consists therefore of lines of comparatively light section and low grade which protect relatively small areas. The numerous small tributaries of the upper river break these lines into a series of short sections which in general tie into the highlands along the banks of tributary streams. These levees will not be discussed here.

In 1915 a disastrous Gulf storm occurred at a time peculiarly favorable for damage to the levees below New Orleans, as the river was at a bankfull stage below that city. This storm raised the water surface in the river more than 5 feet and caused severe wave action. Waves broke over the levees with sufficient volume to carry boats and drift logs completely over them. Wooden wave fences which had been constructed to protect the levees against wave action were completely destroyed. In many places the levees had been faced with concrete for wave protection. Most of this concrete facing was destroyed or damaged. A total of about 18 miles of levee were practically obliterated and about 95 additional miles of levee were more or less damaged. There is no previous record of the coincidence of a hurricane with a high-river stage as the two phenomena usually occur at different seasons. The probability of a recurrence of such a disaster is remote.

A flood occurred in 1916 which culminated on February 4 at Cairo with a maximum stage of 53.2 feet. At all gage stations between Cairo and Sunflower Landing (353 miles below Cairo) the maximum stages were lower than those of the flood of 1913. The White and Arkansas Rivers, however, rose to unprecedented high stages. The peaks of these tributary floods reached the Mississippi River almost simultaneously and near the crest stage on the main stream. Breaks in the Arkansas River levees above the upper limit of Federal construction flooded a large portion of Arkansas and Louisiana. The only crevasse in the Mississippi River levees occurred on the west bank at Buckridge, about 32 miles below Vicksburg. This crevasse reached a maximum width of about 1,800 feet and flooded an area of 818 square miles.

The jurisdiction of the Mississippi River Commission was, by the river and harbor act of July 27, 1916, extended to include the Arkansas River between its mouth and the Lincoln-Jefferson County line with authority for expenditure of funds on this reach. This enlarged the jurisdiction of the Commission to a point about 93 miles above the mouth of the Arkansas River.

Although the flood of 1916 was not severe, its occurrence soon after the disastrous floods of 1912 and 1913 revived public interest in flood control. Federal levee construction had heretofore been limited to work done in the interests of navigation, commerce, and the postal
service. The Mississippi River Commission nevertheless continued to regard the levee lines as necessary, principally for flood protection. There was, moreover, an increasing tendency to consider the levee system as a whole, the destruction of any part of which would jeopardize the safety of the other parts. The trend toward more complete Governmental control of the levee system can be observed in the river and harbor act of March 4, 1915 which directed the Commission to report the amounts expended by local interests for levee construction. The report made pursuant to this act is found in House Document No. 645, 64th Congress, 1st Session. The value of the interests protected by levees was fixed by the report at $994,335,000. From 1882 to 1914 (both years, inclusive), the expenditures made by local interests for flood protection were fixed at $91,106,000. In 1916, testimony given by Members of the Mississippi River Commission before the Congressional Committee on Flood Control contained estimates that the completion of the levee lines below Cape Girardeau would require 155,000,000 cubic yards of earthwork and would cost from $45,000,000 to $50,000,000.

On March 1, 1917 the first flood control act was passed. So far as concerns levees, this act contained three important provisions: First, the construction of levees for flood control purposes was authorized. Second, all levee construction by the Federal Government was made contingent upon local cooperation. Third, the Commission was authorized to expend Federal funds upon tributaries to the extent necessary to protect the upper limits of any Alluvial Valley basin from flood. With reference to the second provision, the law required local interests to contribute toward levee construction, such a proportion of the cost as the Mississippi River Commission might fix, but not less than one-third of the cost. Rights of ways continued to be provided by local interests as in the past. Local interests were also charged with the maintenance of completed works. The Mississippi River Commission had control over the expenditure of contributed funds as well as Federal funds. This act did not, however, deny local interests the right to construct levees for their own protection entirely at their own expense. In the past, expenditures by local interests had exceeded Government expenditures. The act substantially eased the burdens of the local interests.

After the passage of the first flood control act, levee construction along the St. Francis front was pushed to completion. These levees had hitherto been inadequate in grade and section and had not been continuous. The line along the Lower St. Francis Levee District was completed in 1920, and that along the Upper St. Francis Levee District was finished in 1925. The so-called Cypress Creek closure was also completed in 1921. This closure is discussed in detail in Chapter XI and is illustrated on Plate XLI.

The flood of 1920 was moderate both as to stage and duration. The levees constructed by the Federal Government sustained little damage except below New Orleans where two breaks totalling 475 feet in length occurred. Flood water passing through the Cypress Creek gap (then open) overflowed an area of about 687 square miles.

The flood of 1922 caused three crevasses below Vicksburg which totalled 5,800 feet in width. This flood, however, caused apprehension as to the safety of New Orleans and as a result a relief outlet was constructed by the removal of the east bank levees at Pointe a la Hache,
about 45 miles below New Orleans. The Pointe a la Hache relief outlet is described in Chapter XI.

The act of September 22, 1922 extended the jurisdiction of the Mississippi River Commission to the tributaries and outlets of the lower river below Cairo insofar as these tributaries and outlets were affected by the flood waters of the Mississippi. The act of March 4, 1923 further extended Commission jurisdiction over levees to Rock Island, Ill., on the main river. This jurisdiction also included levees along the tributaries which emptied into the main river between Cairo and Rock Island insofar as these tributaries were affected by Mississippi flood waters.

The flood of 1927 was the most disastrous in the history of the river. The extent of the flood and the main crevasses caused by it are shown on Plate XXVII. In addition to the crevasses shown, many breaks occurred in levees outside Commission jurisdiction. An area of about 23,000 square miles was inundated. The total length of levee breached in main river lines was 5.2 miles. Crevasses in levees under Commission jurisdiction on the south bank of the Arkansas aggregated 1.5 miles.

The flood of 1927 resulted in the passage of the second flood control act of May 15, 1928. This act adopted the present flood control plan (contained in House Document No. 90, 70th Congress, 1st Session). The levee system provided for in this plan is shown on Plate LXX. In addition to main river lines, it includes levees on the south banks of the Arkansas and Red Rivers; along both banks of the Atchafalaya River; and guide levees for two floodways in the Boeuf and Atchafalaya Basins respectively. A somewhat smaller floodway is located between Birds Point and New Madrid. In addition, a floodway controlled by a spillway is provided at Bonnet Carre' for the protection of the City of New Orleans. The construction of levees under this plan was initiated at once. On June 30, 1931 the total length of levees below Cape Girardeau was 1,829.5 miles. This includes levees on the tributaries within Commission jurisdiction, Atchafalaya River levees, and levees along the Birds Point-New Madrid and Bonnet Carre' floodways. The experience gained in the flood of 1927 indicated the necessity for a grade and section substantially greater than that of 1914. (See Plate XXXVIII.) Accordingly, new grades and levee sections were adopted in 1928 (the 1928 levee standards are described later in this chapter).

Since the flood of 1927, one flood has occurred, that of 1929. Despite the injury done to the levee system by the flood of 1927, the entire main river levee system below Cape Girardeau successfully withstood the 1929 flood without a break. Similar river heights had never before occurred without crevasses. Complete protection was afforded by the Government works and no flood losses were sustained except in the backwater areas, or areas overflowed by tributary waters outside the limit of the existing Government project. This excellent record was accomplished within two years after the most disastrous flood in river history and only one year after the adoption of the present flood control plan.

A resume' of the growth of the levee system is shown in the following tabulation:
TABLE XXX

LENGTH OF THE MISSISSIPPI RIVER LEVEE SYSTEM BELOW CAPE GIRARDEAU

(Including levee lines on the tributaries and outlets so far as these levee lines are within the jurisdiction of the Mississippi River Commission.)

<table>
<thead>
<tr>
<th>Year</th>
<th>Miles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1880</td>
<td>991</td>
</tr>
<tr>
<td>1890</td>
<td>1,239</td>
</tr>
<tr>
<td>1905</td>
<td>1,439</td>
</tr>
<tr>
<td>1910</td>
<td>1,500</td>
</tr>
<tr>
<td>1920</td>
<td>1,547</td>
</tr>
<tr>
<td>1923</td>
<td>1,555</td>
</tr>
<tr>
<td>1927</td>
<td>1,582</td>
</tr>
<tr>
<td>1931</td>
<td>1,830</td>
</tr>
</tbody>
</table>

THE DESIGN AND LOCATION OF LEVEES.

The essential conditions governing levee construction are: a height adequate to prevent overtopping; a base wide enough for protection against destructive foundation seepage; and a cross section sufficiently massive for security against dangerous seepage through the structure itself. The development of levee types has been largely the result of practical experience.

Levees may in general be grouped in two classes, trapezoidal and banquette levees. (See Plate XXXIX.) In the former type the cross section is a trapezoid, while in the latter type this trapezoidal section is buttressed on its landside slope by an additional section of earth known as a banquette. In general, European practice differs from American practice in the steepness of side slopes and in the width of crown. European levees usually carry roadways and therefore generally have wide crowns and steep side slopes. In the United States this is not always the case. Mississippi River levees have no roads on their crowns. The crowns of Mississippi levees are narrow and their side slopes relatively gentle.

The first Mississippi River levee (the New Orleans levee) apparently followed European custom. Although this levee was probably only about 3 feet high, its crown width was 18 feet and it carried a roadway. By 1735 this levee line had been extended up and downstream for a total distance of about 42 miles. The new levee was said to be weaker than that immediately in front of the city. The levees built between 1735 and 1850 were generally weak and insufficient in section. They were probably all of the trapezoidal type. In planning these lines, no effort seems to have been made to predict the stage of the fully confined river. The construction of the levees was largely in the hands of the local landowners who probably determined the grades and sections to which they were built. There is no evidence of effective supervision by any central authority. The extension of the early levee lines
raised flood heights and necessitated the enlargement of the levees first constructed. The enlargement of these levees merely transferred the points of weakness elsewhere. Little attention was paid to the danger of seepage. Furthermore, the location of the levee lines was frequently faulty. Many levees were too close to the river and were frequently breached by caving banks during low stages. The history of early floods is a repetition of levee failures by overtopping, crevassing, and caving banks.

The passage of the swamp acts in 1849 and 1850 was followed by attempts by the states to standardize the construction of the levees which were to be built as a result of these laws. Levee practice differed in the different states. Louisiana, for example, provided for a crown width equal to one-third the base width, and side slopes of 1 on 2 to 2½. In Mississippi, the chief engineer designated the side slope (usually 1 on 6 for the riverside and 1 on 2½ for the landside). He also fixed the levee grade. Arkansas provided for the following dimensions: crown width equal to height, and base width equal to seven times the height. The elevation of Arkansas levees was to be at least thirty inches above overflow. These efforts accomplished little towards the development of effective levee protection, as the floods from 1858 to 1874 abundantly testified.

One of the earliest proposals for the establishment of levee grades was that of Mr. Charles Ellet. In his report of 1851 (see House Committee Document No. 5, 70th Congress, 1st Session), Ellet advocated outlets and reservoirs as primary means of securing flood control. Pending the opening of the outlets, however, he suggested that levee grades below Old River be raised to a grade at least 6 feet above the highest known floods. Ellet did not believe that this proposed grade would afford protection against the greatest flood which might occur. He merely considered these levees as auxiliaries to the other flood control works proposed by him.

Humphreys and Abbot, in their report on the Delta Survey (1861), selected the flood of 1858 as their plane of reference. Their levee grades (given in detail in their report) called for freeboards varying from 3 to 11 feet above the 1858 flood. Table XXXIII is a comparison between the grades proposed by Humphreys and Abbot and the present standard levee grades.

The history of the subsequent development of the levee cross section is the history of the Mississippi River Commission. Although the Federal Government has never attempted to impose its standards for levee construction upon local interests, the Mississippi River Commission soon standardized Federal levee practice. Furthermore, the Commission has always fixed the grade and cross section of levees built jointly by the United States and by local interests. Local interests have therefore gradually adopted United States practice as standard although privately built levees fail, in the majority of cases, to meet Federal specifications. The development of the levee section between 1882 and 1928 is illustrated on Plate XXXVIII.

In the year 1882 the Mississippi River Commission prescribed the following levee specifications: crown width 8 feet, except where otherwise directed by the engineer; side slopes to be designated by the engineer; borrow pit to be located not less than 20 feet from the riverside toe of
the structure; ground occupied by the levee to be cleared of trees, stumps, and all other perishable material; and trees and stumps to be cut level with the surface of the ground. When the levee was less than 5 feet in height, all stumps were grubbed out. The entire surface of the levee foundation was broken by spade or plow in order to bond with the levee section. A muck ditch 4 feet wide at the top, 2 feet wide wide at the bottom, and 3 feet deep was required. All stumps and roots crossing this ditch were to be removed beyond the base of the levee. The muck ditch was to be located on the riverside of the center line and 3 feet from that line. For details of these specifications, the reader is referred to the Annual Commission Report for 1882. The specifications for the levees on the St. Francis and Yazoo fronts, prepared in 1883, are cited as representative of levee practice at that time. (See Annual Report of the Commission for 1883.) The levee grade was fixed at 1\(\frac{1}{2}\) feet above the high water of 1882. The riverside slope was 1 on 3 and the landside slope was 1 on 2. The width of the crown equalled the height of the levee up to a grade of 8 feet; above this grade the crown width was 8 feet. The muck ditch conformed to the standard requirements.

After the flood of 1882 it became current practice to flatten the landside slope from 1 on 2 (previous practice) to 1 on 3. Following the flood of 1890, it became current practice to add a banquette of moderate height and width at places where the levee crossed old bayous or lake beds.

In 1896 the following standard dimensions were adopted: crown width 8 feet; riverside slope 1 on 3; landside slope 1 on 3 to a point 8 feet below the crown, thence 1 on 10 for a distance of 20 feet (the banquette); thence 1 on 4 to the landside toe of the levee. By these specifications the banquette type was adopted as standard for Government built levees. These specifications, however, proved inadequate for the floods of 1912 and 1913; the principal cause of levee failure in these floods being overtopping. In 1914, therefore, the Mississippi River Commission adopted a new standard. The grade was fixed at 3 feet above the 1912 high water confined. The banquette was from 5 to 8 feet below the levee crown, depending upon the height of the levee. For levees between 10 and 13 feet in height, the width of the banquette was fixed at 20 feet. For levees between 13 and 16 feet, the banquette width was fixed at 30 feet; while for levees exceeding 16 feet in height, the banquette width was fixed at 40 feet. For very high levees, a secondary banquette or wide berm was sometimes provided over very low ground. A description of the 1914 levee section is found in the Annual Report of the Mississippi River Commission for that year.

The flood of 1927 proved the inadequacy of the 1914 levee grades. Following this flood, a new set of levee standards known as the 1928 section was adopted. The 1928 specifications abandoned the banquette type for the trapezoidal levee section. For purposes of levee construction, the alluvial soils of the lower valley were divided into three classes, buckshot (or clay), loam, and sand. The properties of these different soils will be described later in this chapter. The freeboard was fixed at 1-foot above the maximum probable flood. Freeboard will be discussed later in this chapter. The 1928 levee section for levees below Cape Girardeau, Mo., is shown in the following table.
TABLE XXXI
1928 LEVEE SECTIONS—LOWER MISSISSIPPI RIVER

<table>
<thead>
<tr>
<th>Section</th>
<th>Crown Width (feet)</th>
<th>Riverside Slope</th>
<th>Landside Slope to contain seepage line of</th>
<th>Governing Material</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>10</td>
<td>1:3</td>
<td>1:6</td>
<td>75% or more buckshot</td>
</tr>
<tr>
<td>B</td>
<td>10</td>
<td>1:3½</td>
<td>1:6½</td>
<td>Loam</td>
</tr>
<tr>
<td>C</td>
<td>12</td>
<td>1:5</td>
<td>1:8</td>
<td>75% or more sand</td>
</tr>
</tbody>
</table>

Note: Seepage lines are assumed to spring from the riverside slope, 1 foot below the crown of the levee.

The specifications outlined in Table XXXI are the result of more than fifty years practical experience including the experience gained in the greatest flood of record (1927). No less rigorous requirements are regarded as satisfactory for the Lower Mississippi River. For the upper river, however, this very strong section is not necessary. Above Cape Girardeau, the standard levee is of the banquette type built to specifications approximating the 1914 levee section.

In Table XXXII will be found a comparison of the various levee grades adopted by the Mississippi River Commission.

The two objects of levee construction, namely, protection of land and concentration of flood discharge, are most effectively served by locating the levee lines as close as possible to the river bank. The river banks are usually higher than the lands farther back. River bank levees need not, therefore, be as high as levees built further back from the bank. The river bank is, however, exposed to the constant danger of caving and it is therefore usually necessary to set the levee back from the bank. Old cut-off lakes, bayous, sloughs, and outlets are complicating factors in the selection of levee lines. Prior to 1928, Commission policy contemplated the location of levees so as to insure a probable useful life of at least 20 years. The present flood control plan, however, contemplates a levee life of at least 30 years, a change in policy resulting from the increase in levee construction costs. The new policy has necessarily caused local levee realignments, especially at sharp bends in the channel. These realignments frequently throw minor areas outside the levee lines. Whenever practicable, however, the existing levees in front of such areas are permitted to remain intact, thus affording protection for whatever remaining useful life they may have.

As has been stated above, the Commission, between 1900 and 1910, seriously considered the practicability of bank protection as a means of protecting levees against bank caving. As has been shown, bank protection is an uneconomic method of levee protection. Bank protection is now used to protect levees only in exposed locations from which the retirement of the levee line is impracticable.

Experience has developed a standard practice in the preparation of levee foundations. Present specifications require that the entire levee foundation, together with adjacent strips 5 feet wide, be prepared as follows: All trees are cut down and removed. All logs, brush, and debris on the surface of the foundation are also removed. The foundation, to-
gether with the contiguous 5-foot strips, is then thoroughly grubbed and all stumps, roots, buried logs, etc., removed. All tap roots over \(1\frac{1}{2}\) inches in diameter are removed to a depth of 6 feet. An inspection or muck ditch, 6 feet wide at the top, 4 feet wide at the bottom, and 6 feet deep, is dug normally along the center line of the foundation. All logs, decayed vegetable matter, etc., revealed by this ditch are removed and the ditch is then back filled. When sloughs, old pits, ditches, or depressions exist within 100 feet of the base of the levee on the land side, or within 40 feet on the river side, the specifications usually provide that they be filled up to the natural surface of the ground. It may be stated that the removal of cypress stumps is sometimes made the subject of special provisions in the specifications. These stumps are so tenacious that they must generally be removed by blasting, a process which is apt to increase the danger of foundation settlement. The provisions made for the removal of cypress stumps under such conditions are described below.

Levees must frequently be built on locations where the substrata are not strong enough to support their weight. In such a case, subsidence of the levee by compression or movement of the foundation occurs. When the amount of this subsidence is small, work on the levee section may be continued after settlement has ceased with a reasonable hope of permanency. When the subsidence is large or continuous over a long period, however, repair is expensive and difficult. A subsidence is essentially a foundation failure and must be clearly differentiated from a slide or slip in the levee section itself. A slide is a construction fault resulting from the use of poor levee material or the improper placement of material in the levee section. The similarity in the appearances of slides and subsidences is frequently the cause of controversy.

Whether a movement of material in the levee is a slide or a subsidence may usually be determined by careful cross-section measurements, and by examination of the ground within a few hundred feet of the center line of the levee. A subsidence is accompanied by a definite loss in the cross-section area of the levee. The slide, on the other hand, being largely confined to the material above the ground surface, merely causes a change in the shape of the levee cross section with little or no loss in cross-section area. A subsidence is normally accompanied by an appreciable disturbance of the surface of the contiguous ground surface. This ordinarily takes the form of an upheaval or series of upheavals. The evidence of subsidence is particularly apt to be found in the borrow pit if the latter is in the zone of disturbance. The borrow pit constitutes a zone of foundational weakness and upheaval is normally found there. It is good practice to locate the borrow pit outside the zone of probable disturbance whenever subsidence is considered probable.

With the increase in levee grade and section, subsidences have increased in frequency. Settlement is especially likely to occur where the levee line crosses old sloughs, bayous, cypress brakes, and similar depressions in the surface of the ground. Settlement is also likely to occur when the levee foundation contains strata of quicksand. It is better to avoid subsidences rather than to attempt to repair them. To this end present practice contemplates the thorough study of foundations by means of borings taken at fixed intervals along the center lines of proposed levees. Whenever foundation weakness is indicated,
Inspection ditch has been dug; stumps have been removed; and the ground surface has been plowed up.

Cottage Bend Levee near St. Joseph, La., August 16, 1931.
a thorough exploration is made before the levee location is finally fixed. When weak foundations can not be avoided, however, two methods of levee construction present themselves. The first method consists in the continued placement of dirt in the levee section until the foundation is compressed to the point of stabilization. The second consists in the construction of false berms. By the first process the levee is built up until subsidence begins. After subsidence has occurred, work is suspended for a time to allow the foundation to "set". The fill is then continued by the methods least likely to cause a recurrence of the settlement. Should other subsidences take place, this process is repeated until the levee is brought up to grade and section. The second method consists in the construction of a wide false berm or banquette extending outward from the center line of the levee. This false berm must extend far enough and must be heavy enough to act as an effective counterpoise for the weight of the levee itself. Hydraulic methods are particularly suitable for the construction of false berms and wide footings. Hydraulic methods are discussed later in this chapter. As has been mentioned above, the judicious selection of borrow pit locations is a necessary accompaniment of proper foundation treatment. Where weak foundations are encountered, the borrow pits must be so located that they do not further weaken the foundation. Borrow pits are discussed in detail later in this chapter.

The difficulty in removing cypress stumps from the levee foundation has been mentioned above. Where cypress stumps are encountered on weak foundations, levee specifications make special provision for their treatment. These usually limit the use of explosives in clearing to light charges merely sufficient to split the stumps in order to facilitate their removal by grubbing. Heavy charges which blow the stumps out are likely to disturb the foundation and weaken it further. For this reason heavy charges are prohibited. It is now considered good practice to permit cypress stumps to remain, under certain conditions, in the foundation in depressions for which false berms are specified. They are merely cut off not more than 1-foot above the ground. No cypress stumps are allowed to remain on the sloping banks of bayous, nor are any permitted where the net fill fails to cover their tops to a depth of at least 5 feet. Cypress stumps are accorded this special treatment because cypress is a wood which is extremely resistant to decay. Other timber is always removed.

One peculiarity in connection with levee foundations may be noted. The point to which a levee foundation can be loaded is frequently very delicately fixed. Levees built on weak foundations have frequently existed for years without a sign of subsidence. Attempts to enlarge them have usually been followed by subsidences which are frequently more extensive and serious than those which occurred during the original construction.

The levee now under construction at Ward Lake (on the east bank of the Mississippi River about 210 miles below Memphis) is an interesting example of a serious and extensive subsidence. This levee crosses an old lake bottom. The material composing the bottom of this old lake is characteristic of old river channels, sloughs, and lakes in the valley of the Lower Mississippi. It consists of a bluish wet clay of about 40 per cent moisture content with a high percentage of decomposed vegetable matter. The construction of this levee was begun in 1904. The
levee material was placed from a timber trestle built across the lake bed. When the grade of the earthwork reached the top of the trestle, the latter was removed and the balance of the levee material was placed by "haul-in" equipment operating on the fill itself. The increasing weight of the fill caused subsidence which continued until stabilization was ultimately reached. The disturbance in the foundation extended for approximately 600 feet on each side of the center line of the levee. At the toes of the levee, the original ground service was elevated about 10 feet. The levee was finally stabilized and completed in 1910.

The levee grade established by the present flood control plan (act of May 15, 1928) necessitated the raising of this levee by about 8 feet. This was to be accomplished by riverside enlargement with some modification of the landside slope (including filling in over the banquette of the existing levee). Work began in May, 1931. Standard gage railroad "haul-in" equipment was used. On August 2, 1931 a settlement occurred after about 17 feet of fill had been placed. The material in the levee section moved downward and outward toward the river approximately along the riverside slope of the original levee. As additional material was placed, this movement continued. The ground surface on the river side, about 40 feet from the toe of the levee, was upheaved about 15 feet and sloped outward from this point in irregular waves. At some points this disturbance extended outward for about 1,000 feet from the center line of the levee. The average distance from the center line of the levee to the riverside limit of the zone of visible disturbance was about 800 feet. Stakes placed to mark the riverside toe of the levee were moved toward the river by distances varying from about 35 feet to about 159 feet. By January 7, 1932 the subsidence had stabilized throughout a portion of the levee line. Borings taken throughout this stabilized reach indicated that, where the unstable foundation strata were relatively thin, the new fill had completely squeezed out the soft wet clay and was resting on a firm, sandy foundation. Where the unstable, underlying strata were thick, boring gave little information since the compression of the wet clay strata squeezed out water which saturated the new fill. Where the unstable foundation strata were very thick, the levee fill did not apparently stabilize until the upheaval in the zone of disturbance on the riverside of the levee became sufficiently extensive to act as a counterbalance for the weight of the levee.

It will be noted that the subsidence was confined to the riverside of the levee. No disturbance was noted on the landside.

In general, it is not possible to establish with precision the flow line of the flood against which the levee is designed to afford protection. For this reason, and to prevent overtopping by waves, the levee is provided with freeboard. A justifiable freeboard above an assumed flood flow line is a complex function of the frequency, height, and duration of the flood; and also of the levee cross section. Experience indicates that a freeboard of at least 3 feet should be used when the levee design is based on the maximum flood of record. When the levee grade is based on the computed maximum probable flood, a freeboard greater than 1-foot is unwarranted for levees of the type now standard on the Lower Mississippi River. Excessive freeboards materially increase construction costs. Hence, freeboards should be held at the minimum consistent with security and reasonable cost of maintenance during flood. It is possible
to raise an inadequate levee grade by emergency topping but this is expensive and has its limitations. The present minimum freeboard of 1-foot allowed by levee specifications considers the maximum probable flow against which the present structure affords protection. The history of the river, so far as known for the past two hundred years, shows no flood approximating this theoretical maximum. A levee designed to afford 1-foot freeboard against this flood will contain with more than 3 feet of freeboard any flood approximating the normal. The present freeboard requirements, therefore, constitute a reasonable compromise between extreme economy and extreme structural stability.

The following table furnishes a comparison between the levee grades selected from time to time by the Mississippi River Commission and those now in force. This table is an apt illustration of the increase in the elevation of flood flow lines resulting from the increasingly complete confinement of floods by levee lines.

**TABLE XXXII**

**STANDARD LEVEE GRADES (1881 to 1928)**

(Referred to gage zeros at stations listed)

<table>
<thead>
<tr>
<th>Locality</th>
<th>Miles below Cairo</th>
<th>Levee Grades (Expressed in feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1881</td>
<td>1883</td>
</tr>
<tr>
<td>Cape Girardeau</td>
<td>54.5</td>
<td></td>
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<td>Cairo</td>
<td>0</td>
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</tr>
<tr>
<td>Columbus</td>
<td>21.8</td>
<td>47.10</td>
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<tr>
<td>New Madrid</td>
<td>71.0</td>
<td>39.53</td>
</tr>
<tr>
<td>Cottonwood Point</td>
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<td>38.69</td>
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<tr>
<td>Fulton</td>
<td>175.4</td>
<td>37.15</td>
</tr>
<tr>
<td>Memphis</td>
<td>227.0</td>
<td>41.80</td>
</tr>
<tr>
<td>Mhoon Landing</td>
<td>273.2</td>
<td>49.49</td>
</tr>
<tr>
<td>Helena</td>
<td>307.1</td>
<td>43.65</td>
</tr>
<tr>
<td>Sunflower Landing</td>
<td>353.7</td>
<td>40.40</td>
</tr>
<tr>
<td>Mouth of White River</td>
<td>391.7</td>
<td>49.0</td>
</tr>
<tr>
<td>Arkansas City</td>
<td>436.7</td>
<td>49.6</td>
</tr>
<tr>
<td>Greenville</td>
<td>480.2</td>
<td>45.0</td>
</tr>
<tr>
<td>Lake Providence</td>
<td>542.0</td>
<td>41.9</td>
</tr>
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<td>Vicksburg</td>
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</tr>
<tr>
<td>St. Joseph</td>
<td>662.4</td>
<td>46.4</td>
</tr>
<tr>
<td>Natchez</td>
<td>705.7</td>
<td>47.8</td>
</tr>
<tr>
<td>Red River Landing</td>
<td>772.6</td>
<td>47.8</td>
</tr>
<tr>
<td>Bayou Sara</td>
<td>809.9</td>
<td>49.0</td>
</tr>
<tr>
<td>Baton Rouge</td>
<td>841.0</td>
<td>37.6</td>
</tr>
<tr>
<td>Plaquemine</td>
<td>861.0</td>
<td>33.2</td>
</tr>
<tr>
<td>Donaldsonville</td>
<td>892.8</td>
<td>30.9</td>
</tr>
<tr>
<td>College Point</td>
<td>911.3</td>
<td>25.20</td>
</tr>
<tr>
<td>Carrollton</td>
<td>964.5</td>
<td>18.1</td>
</tr>
<tr>
<td>Ft. Jackson</td>
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<td>10.00</td>
</tr>
</tbody>
</table>

* Above Cairo.

Perhaps the most striking illustration of the development of levees is furnished by comparison of the grades adopted in 1928 with those estimated by Humphreys and Abbot in 1861, and by the Levee Commission in 1875, as being sufficiently high for complete security. The following table illustrates this comparison.
### Table XXXIII

**Comparison of Levee Grades Recommended by Humphreys and Abbot (1861), and the Levee Commission (1875), with Those Adopted in 1928**

(Expressed in feet and referred to zeros of present gages at stations listed)

<table>
<thead>
<tr>
<th>Locality</th>
<th>Miles below Cairo</th>
<th>Grade Line Recommended by Humphreys and Abbot, 1861</th>
<th>Grade Line Recommended by Levee Commission, 1875</th>
<th>1928 Grade Line</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cairo</td>
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<td>53.6</td>
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<tr>
<td>Columbus</td>
<td>21.8</td>
<td>46.7</td>
<td>46.7</td>
<td>54.0</td>
</tr>
<tr>
<td>New Madrid</td>
<td>71.0</td>
<td>38.8</td>
<td>38.8</td>
<td>52.5</td>
</tr>
<tr>
<td>Cottonwood Point</td>
<td>124.5</td>
<td>42.9</td>
<td>42.9</td>
<td>49.5</td>
</tr>
<tr>
<td>Fulton</td>
<td>175.4</td>
<td>40.9</td>
<td>40.9</td>
<td>50.5</td>
</tr>
<tr>
<td>Memphis</td>
<td>227.0</td>
<td>40.0</td>
<td>40.0</td>
<td>54.5</td>
</tr>
<tr>
<td>Mhoon Landing</td>
<td>273.2</td>
<td>45.5</td>
<td>45.5</td>
<td>53.0</td>
</tr>
<tr>
<td>Helena</td>
<td>307.1</td>
<td>50.6</td>
<td>50.6</td>
<td>66.5</td>
</tr>
<tr>
<td>Sunflower Landing</td>
<td>353.7</td>
<td>50.6</td>
<td>50.6</td>
<td>61.0</td>
</tr>
<tr>
<td>Mouth of White River</td>
<td>391.7</td>
<td>55.2</td>
<td>55.2</td>
<td>65.0</td>
</tr>
<tr>
<td>Arkansas City</td>
<td>438.7</td>
<td>54.0</td>
<td>54.0</td>
<td>63.5</td>
</tr>
<tr>
<td>Greenville</td>
<td>480.2</td>
<td>50.3</td>
<td>50.3</td>
<td>59.5</td>
</tr>
<tr>
<td>Lake Providence</td>
<td>542.0</td>
<td>51.3</td>
<td>51.3</td>
<td>57.5</td>
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<tr>
<td>Vicksburg</td>
<td>601.8</td>
<td>51.5</td>
<td>51.5</td>
<td>61.0</td>
</tr>
<tr>
<td>St. Joseph</td>
<td>662.4</td>
<td>47.8</td>
<td>47.8</td>
<td>58.0</td>
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<tr>
<td>Natchez</td>
<td>705.7</td>
<td>52.4</td>
<td>52.4</td>
<td>61.0</td>
</tr>
<tr>
<td>Red River Landing</td>
<td>772.6</td>
<td>50.3</td>
<td>51.3</td>
<td>60.5</td>
</tr>
<tr>
<td>Bayou Sara</td>
<td>806.9</td>
<td>41.4</td>
<td>43.4</td>
<td>54.0</td>
</tr>
<tr>
<td>Baton Rouge</td>
<td>841.0</td>
<td>38.2</td>
<td>39.2</td>
<td>50.0</td>
</tr>
<tr>
<td>Plaquemine</td>
<td>861.0</td>
<td>33.3</td>
<td>35.3</td>
<td>44.5</td>
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<tr>
<td>Donaldsonville</td>
<td>892.8</td>
<td>29.9</td>
<td>31.9</td>
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<td>College Point</td>
<td>911.3</td>
<td>25.3</td>
<td>27.2</td>
<td>34.5</td>
</tr>
<tr>
<td>Carrollton</td>
<td>964.5</td>
<td>18.5</td>
<td>20.2</td>
<td>25.2</td>
</tr>
</tbody>
</table>

In early levee construction, the borrow pits from which the levee material was taken were frequently placed on the landside of the levee. This was, however, a direct invitation to foundation seepage, a danger which increased as the flood plane rose with the extension of the levee system. As levee practice became standardized, borrow pit practice followed suit. Borrow pits are now habitually placed on the riverside of the levee and precautions are taken to minimize the danger of foundation seepage from them. They are separated from the levee by a wide berm and are dug to relatively shallow depths only. Landside borrow pits are avoided even at the expense of long hauls to the levee site from distant riverside pits. When landside borrow pits are unavoidable, their use is surrounded by rigorous specifications. The rigid enforcement of these specifications is essential.

Differences in pit specifications arise from differences in subsoil. Subsoils found above the mouth of Red River are much less satisfactory for levee foundations than those found below Red River. Standard specifications for levees above Red River call for a berm 40 feet wide between the riverside levee toe and the inner lip of the borrow pit. The inner slope of the pit is dug not steeper than 1 on 2 to a depth of 3 feet and the slope of the pit bottom toward the river is not greater than 1 on 50. Below Red River the berm is also 40 feet wide but the pit bottom slope toward the river may be as steep as 1 on 10. Above the Missouri River the levee berm is usually about 20 feet wide and the pit is deeper than on the lower river. Landside borrow pits above Red
River must be located not less than 100 feet from the landside levee toe, while below Red River this minimum distance is reduced to 80 feet. The landside borrow pits must not be more than 3 feet deep at the side nearest the levee and its bottom must slope away from the levee on a grade not steeper than 1 on 100.

To prevent destructive high-water currents through riverside borrow pits, traverses are ordinarily left in them at stated intervals. These traverses are prisms of natural earth, extending perpendicularly across the pits. They are 14 feet wide at their tops and their side slopes are 1 on 2. These traverses are cut to permit ordinary drainage, otherwise the standing water ponded by them might prove a serious menace to the levee foundations.

Levee requirements call for clean earth free from objectionable matter. This means that the borrow pits must be cleared ahead of the levee construction. Before construction of the levee, the character of the borrow pit soil is investigated by borings. The locations of new levee lines are determined only after due consideration of the locations of possible borrow pits and the character of the soils.

Borrow pit specifications do not, however, constitute the sole preventive measure against foundation seepage. As has been stated above, the entire foundation site is cleared and an inspection or muck ditch is dug. Buried logs, stumps or roots which are brought to light by it are of course removed. When back filled, the muck ditch serves to interrupt the continuity of porous foundational strata. Proper back fill is therefore important and must be done carefully, and with good material.

Objections to the muck ditch are sometimes advanced. These are usually based on grounds of increased danger of seepage and subsidence. It is occasionally claimed that the muck ditch raises the hydraulic gradient in the levee section by permitting the upward transmission of hydrostatic pressure from the foundation. This fear is apparently ungrounded. No case is known where such a rise in hydraulic gradient has occurred. The increased danger of subsidence is a more important objection. The present massive levee section has greatly increased foundation pressure. By cutting foundation strata, the muck ditch decreases its bearing power under the zone of maximum pressure. Whatever the merits of this contention may be, the muck ditch is still considered an essential part of levee design and is omitted only under local conditions of an exceptional nature. The removal from the foundation of logs, stumps and other decayed vegetable matter and the interception of foundation drainage are so important to the safety of the structure that they outweigh any possible disadvantages of muck ditch construction.

Practical considerations demand that the levee be built of such material as may be locally available. Different classes of levee material are frequently found on a levee site as the borrow pits extend through strata of different soils. It is seldom, therefore, that a levee of any appreciable length is composed exclusively of one class of material. In general the soils available for Lower Mississippi Valley levee construction fall into three classes; clay, loam, and sand. The clay is of the blue or black variety and is usually mixed with sand or other impurities. It is locally known as "gumbo" or "buckshot". The term "loam" includes the larger portion of the available levee material. It
is very light, compacts somewhat slowly, and is easily eroded unless protected by a covering of vegetable growth. Loam is, however, the predominant material used for levee construction on the Mississippi River and, in spite of its deficiency, it gives comparatively satisfactory results. Sand, when sufficiently coarse, is a fairly satisfactory levee material. On the Lower Mississippi River, however, it is likely to be too fine for very good results. As has been stated above, the 1928 levee standards prescribe three different cross sections for the three different classes of levee material.

After the levee embankment is built to the proper height and dimensions, it is smoothly dressed to slope and the entire surface is planted with living sods of Bermuda or some other acceptable grass which will best meet the climatic conditions. The specifications require that the sods be not less than 4 inches square; be placed not more than 18 inches from center to center; and be covered with 1 to 2 inches of earth as directed by the contracting officer. The seeding of the levee slopes may in some cases be required in lieu of sodding.

CONSTRUCTION METHODS.

The site, type, and dimensions of a levee and its borrow pit having been determined, the engineering problems involved in its construction are relatively simple. The governing condition in levee construction is the economic placement of yardage in the levee section.

The most primitive method of levee construction is digging and placement by man power. On the Lower Mississippi River the first levees were built by this method, wheelbarrows being used for the transportation of dirt. The levees were small; labor was plentiful; and the outlay for equipment was small. The method was well adapted for use by the plantation owners who constructed many of the early levees. Gradually a more or less standard procedure developed. Plank runways supported by three-legged timber supports extended on approximately even grades from the borrow pit to the lower section. Borrow pits were on the riverside, the landside, or on both sides, as convenience dictated. The economic limit of haul was about 75 feet. Much of this work was done by "station-men" or laborers paid by the yard. An expert "station-man" could move from 10 to 12 cubic yards per day at a cost varying from 25 to 30 cents per yard. Levee construction by wheelbarrow has, however, long since passed from general use on the Lower Mississippi.

With the growth of the levee section more economic means of moving earth became necessary. Scrapers and wagons then appeared. Both slips and wheel scrapers were used. The economic limit of haul with scrapers approximated about 150 feet. The wheel scrapers had a capacity of about 11 cubic feet. Slips were of course smaller. In operation a snatch team and a plow (with team) were provided for every four or five scrapers. The plows turned over the surface of the levee foundation and loosened the dirt in the borrow pit. Snatch teams were needed for loading the scrapers and for heavy pulling. Under average conditions a scraper outfit could move from 35 to 40 cubic yards per team-day. Scrapers are no longer used for levee construction on the Lower Mississippi River. The present massive levee section with its correspondingly
Closure of Morganza Crevasse about 25 miles below Old River. Work done by local interests.

1890
Refuge Setback 12 miles below Greenville, Mississippi. Tractors and 7-yard wagons placing material. August 30, 1931.

long haul has rendered them uneconomic. Occasionally scrapers are used for dressing the levee crown and slopes but they have no other use.

In construction by dump wagon, elevating graders were generally employed for loading. At first these graders were hauled by teams but later mechanical traction was employed. With the development of mechanical propulsion the use of animal drawn wagons has declined to the point where in the present day they are usually found only on occasional minor jobs of a size insufficient to justify the mobilization of mechanical equipment. Present day mechanized equipment consists of tractor drawn dump wagons ordinarily loaded by draglines. The dump wagons are of from 6 to 10-yards capacity and are frequently of the crawler type. Dump trucks equipped with removable treads for wet weather service are also used to some extent. In construction by this method, auxiliary equipment is required to dress the levee slopes. Bulldozers are usually very satisfactory for this purpose. Dump wagons and trucks are particularly useful when hauls are long and borrow pits irregular. They give good service under dry conditions but are ill-adapted to wet weather work. The output of a dump wagon outfit depends upon character of levee material, length of haul, weather, and efficiency of organization. It is therefore not possible to state a "normal" or "average" yardage per unit of equipment. In this connection, however, the reader is referred to an article entitled "Constructing the Thirty-five-Mile New Madrid Floodway Levee" written by Mr. T. T. Knappen, Senior Engineer, U. S. Engineer Department. This article appears in the Engineering News-Record for April 16, 1931. Mr. Knappen there describes an organization which averaged about 15,000 cubic yards per 8-yard dump wagon per month working double shifts.

The need for efficient levee building methods led the Mississippi River Commission in 1892 to initiate experiments toward the development of improved types of equipment. Early experiments included the equipment of the Dredge Pal-Ute with a high speed engine operating special earth handling apparatus. This equipment proved to be impracticable. Attention was then turned to hydraulic methods, and in 1895 the cutter head Dredge Ram successfully built a short section of levee in the vicinity of Carrollton (New Orleans). Hydraulic levee construction by the Ram in 1896 was not so successful, however. In 1898 attempts by levee contractors to use specially designed levee building machines at Longwood (100 miles above Vicksburg) and at Plaquemine (20 miles below Baton Rouge) were both failures. In the years 1900-1, however, especially designed machines were successfully used in the Lafourche Levee District (west bank between Donaldsonville and New Orleans). These machines operated revolving cranes and moved along the berm on the riverside of the levee. In 1907 a levee machine similar to a revolving crane was built and successfully operated in the Memphis District. This machine is described in the Annual Report of the Chief of Engineers for 1908. In 1908 a similar machine was purchased.

The floods of 1912 and 1913 were followed by the adoption of the 1914 levee section which was much more massive than that previously considered standard. Larger levee building equipment became necessary. Two general types were developed, the dragline and the so-called tower machine. The reader is again referred to the article by Mr. T. T. Knappen entitled "Constructing the Thirty-five-Mile New Madrid

The dragline was at first best adapted to work below Red River where the narrow berm and deep borrow pits required a shorter boom than did work above Red River. The increasing use of the dragline, however, resulted in the development of larger machines. Draglines which are capable of operating 6-yard buckets over a 175-foot radius are now in use on the lower river.

The tower machine is essentially a slack cableway supported by a “head” tower and a “tail” tower. The head tower is usually from 100 to 135 feet in height and is mounted on a platform which also carries the operating machinery. The whole is self-propelled, usually on a self-laid track. The tail tower is from 25 to 40 feet in height and is usually supported by a self-propelled tractor mount. In operation the head tower is located on the landside of the levee under construction, while the tail tower is placed directly opposite on the riverside of the borrow pit. The slack cableway is thus suspended across the levee and borrow pit. The cableway supports a wheeled carriage from which the bucket is suspended. The bucket is run by gravity from the head tower to the tail tower. At the end of its run it is “let go” into the pit and is pulled toward the head tower in a manner similar to the operation of the dragline bucket. As the berm of the borrow pit is approached, the bucket is raised by the cableway and thus cuts to a slope approximating that allowed by specifications for the bottom of the borrow pit. The bucket crosses the berm and is dumped in the levee section. It then runs back to the riverside of the borrow pit and repeats the operation. As with other types of equipment, the yardage which a levee machine can handle depends largely on individual circumstances. A levee machine with a 110-foot head tower and a 10-yard bucket can, however, average as much as 8,000 yards per double shift day under favorable conditions.

Auxiliary equipment (scrapers, bulldozers, etc.) is needed to dress levees built by tower machines and draglines.

The majority of the draglines and tower machines in use on the lower river were built for the 1914 levee standard. The adoption of the 1928 levee section rendered most of these old machines obsolescent since they were too small for the new levee section.

It is now common practice for draglines to work in multiple. One dragline operates on the berm and places material in the levee section. Machines in the pit place material for the first machine to rehandle. Tower machines also have been successfully used in pairs, and even in groups of three, on large levee sections. One tower machine builds the first “lift” in the levee section by digging in the borrow pit next to the berm. The head tower of the second machine operates on the embankment built by the first machine and the third machine (when used) operates on the embankment built by the second one. The additional elevation thus afforded the head towers permits the extension of the height of the cableway by an amount sufficient to permit the buckets to dig to the extreme width of the borrow pit.
LEVEE CONSTRUCTION BY DRAGLINE
Morrison-Picayuneville Levee about 25 miles below New Orleans.
June 15, 1931
LEVEE CONSTRUCTION BY TOWER MACHINE

Head tower of machine on Bedford-Bayou Vidal Levee 6 miles below Vicksburg. Note levee fill at extreme left. October 10, 1931.
The use of dredges for levee construction on the lower river is comparatively recent. Sand is the material best adapted to hydraulic levee construction but loam and clay soils can also be used. Before hydraulic levee construction begins, the muck ditch must be excavated and two parallel retaining side dikes must be built at or near the toes of the levee. This is all done by dry-land methods. The fill between the side dikes is made by the hydraulic method. After all material has been placed, the structure is permitted to settle and dry when necessary. When sufficiently dry, it is shaped to grade and section by dry-land methods. One danger of hydraulic levee construction lies in the segregation of "fines". If the dredge discharge is "ponded", the heavy material is deposited close to the mouth of the discharge pipe and a graduation of material in increasing degrees of fineness occurs as the distance from the discharge pipe increases. Ultimately a point is reached where the material deposited is so fine that it constitutes a horizontal layer of weakness through the levee section. This segregation of "fines" occurs in all alluvial soils of the lower valley except sand. To prevent the occurrence of this segregation in the levee, present specifications provide for free outlets of waste water. Ponding is prohibited. The intervals between outlet sluices in the side dikes varies from 1,000 to 2,000 feet according to the size of the dredge in operation. No obstructions to free flow are permitted in outlets or at any other points in the levee section, except that transverse dikes are constructed across the levee section immediately below each outlet. When the end of the discharge pipe is within from 500 to 250 feet of the transverse dike (distance depending upon the size of dredge used), the dike is breached. The sluice-way above it is closed, and the next sluice-way below is put into operation. Hydraulic levee construction is not confined to levee sites within reach of natural waterways. Artificial flotation ditches are frequently dug by the dredges themselves. The dredge normally used for levee construction is a box (non self-propelled) cutter head dredge equipped with spuds. For discussions of levee construction by hydraulic methods, the reader is referred to an article by Major S. C. Godfrey, Corps of Engineers, entitled "Dredging as an Aid to Levee Construction" and an article by Major E. C. Kelton, Corps of Engineers, entitled "Hydraulic Levee Building on Lower Mississippi" (Manufacturers Record, June 20, 1929).

The clamshell dredge is not adapted to the lower river levee construction. This dredge developes its greatest value when working in material which drains easily and assumes a net slope at least as steep as that required in the finished levee section. The soils of the lower valley do not in general possess these characteristics. Buckshot is especially objectionable for levee construction by clamshell. It retains water for a long time and, when wet, slides and flows easily. The present levee cross section is not adapted to construction by clamshell dredge since material destined for the landslide slope must usually be rehandled. To reduce the amount of this rehandling to a minimum, the flotation ditch is normally placed as close as possible to the riverside toe of the levee with resultant danger of sliding and subsidence during construction. Great difficulty is encountered in combating and correcting these section and foundation failures. Once sliding has occurred, the internal friction in the levee section is reduced and resistance to further movement is lessened. Thus works of correction are
often futile and always expensive. Small, incipient slides join to produce slides and subsidences of great extent. In practice it has been found necessary to cut out entirely those sections showing any tendency to slide and to rebuild them entirely with new, dry material. This procedure is very expensive.

Attempts have been made to adapt the belt conveyor to levee construction but they have never been successful on the Lower Mississippi River. The attempts have included both wet and dry methods. The operating machinery is likely to break down, the equipment is heavy, and usually hard to move, and can not handle wet, lumpy material economically. One such unsuccessful attempt was made by a levee contractor operating on the west bank of the Mississippi River about 10 miles below Vicksburg in 1929-1930. His plant was a dredge equipped with a bucket operating on a 100-foot dragline boom. This bucket discharged into a hopper which fed a conveyor system which carried the material into the levee section. The operation of the plant was entirely unsuccessful. The greatest difficulty was apparently encountered in the hopper which could not be made to feed properly on to the first belt conveyor.

Industrial railways have been successfully used for levee construction. In one such set-up, the railway tracks were laid perpendicular to the center line of the levee and the dump cars were loaded by a small dragline operating in the borrow pit. The dump cars dumped on the levee foundation and the material was placed by a second dragline which shaped up the levee section to grade. In another set-up, the tracks were raised and the dirt was dumped directly into the levee section. Although industrial railways are not widely used, they are capable of satisfactory and economical long-haul work.

The method of placement of earth in the levee section is important. The ideal levee building specifications should require that the levee embankment be started full out to the slope stakes and carried regularly up in horizontal layers to gross fill. It is, however, impracticable for most types of levee building equipment to operate in this manner. These ideal specifications are used in practice only for "haul-in" by wagons. With this equipment, the levee is built in layers from 3 to 5 feet thick depending upon the capacity and size of the equipment.

Clean earth, free from all foreign matter, is used in the construction of the embankment. No earth which sloughs, or shows a tendency to slough, is placed in the embankment except when hydraulic methods are used. Great care is taken to avoid the creation of planes of weakness. To this end, measures are taken to secure proper bonding between the levee section and the foundation (by plowing up the foundation). Whenever any portion of the uncompleted levee is unduly compacted from any cause whatever, the compacted surface is thoroughly broken up before fill is placed upon, or against, it. Provision is made for shrinkage after completion of the levee. The allowance made for shrinkage varies with the manner of construction. For hydraulic methods, the allowance is 8 per cent of the net fill. For construction by layers, the shrinkage allowance is fixed at 15 per cent. For other methods, including dragline and tower machines, the shrinkage allowance is 25 per cent. This allowance is so disposed that the levee dimensions will not be reduced by shrinkage below net requirements.
LEVEE CONSTRUCTION BY TOWER MACHINE
Bedford-Bayou Vidal Levee 6 miles below Vicksburg. Bucket dumping in levee section. Tailtower and borrow pit not shown. October 10, 1931.
Cottage Bend near St. Joseph, Louisiana. View from levee section showing riverside borrow pit and tail tower. Mississippi River shown in background. February 26, 1931.
The adoption of the 1928 levee section has necessitated the enlargement of the greater part of the existing levee line. Enlargement of an existing levee and its conversion from the banquette to the trapezoidal section involves no problems essentially different from those of levee construction in the first instance. The same methods and the same equipment are used. In the enlargement of a levee, the old levee crown and slope are of course plowed up to insure proper bonding with the new earthwork. The enlargement is habitually made on the riverside, as landslide enlargement is likely to result in a plane of weakness between the old and new earthwork nearly parallel to the seepage line through the levee. As a rule no muck ditch is constructed in enlargement work.

The specifications covering levee construction are rigid and they are of necessity rigidly enforced. These levee specifications are the result of the entire experience of United States Government engineers and of the local levee districts. They can not be safely relaxed. The increasing height, size, and cost of levees prohibits laxness in construction methods.

ECONOMICS OF LEVEE CONSTRUCTION.

The cost of levee construction is a complex function of many factors among which are: accessibility of job; length of working season; anticipated weather and flood hazards; total volume of work and time limit for completion; nature of material; length of haul; levee and borrow pit specifications; and availability of equipment. General economic conditions also effect costs. Periods of general prosperity are signalized by increased construction costs while periods of depression decrease them.

Table XXXIV below indicates the changes in unit costs of levee construction from about 1860 to the present time so far as these costs can be determined from available data.

<table>
<thead>
<tr>
<th>TABLE XXXIV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Costs of Levee Construction 1860 to 1931</td>
</tr>
<tr>
<td>Cost per Cubic Yard</td>
</tr>
<tr>
<td>Estimate contained in report of Delta Survey, 1861 ......................... 20 cents</td>
</tr>
<tr>
<td>Estimate contained in report of Gen. A. A. Humphreys, 1865 ............. 40 cents</td>
</tr>
<tr>
<td>Estimate of the Levee Commission, 1875 ..................................... 40 cents to 43 cents</td>
</tr>
<tr>
<td>Unit cost of levee construction under jurisdiction of the Mississippi River Commission—from organization of the Commission to June 30, 1928 ................. approximately 23 cents*</td>
</tr>
<tr>
<td>Cost of levee construction under jurisdiction of Mississippi River Commission—July 1, 1928 to December 31, 1931 ......................... 24.5 cents*</td>
</tr>
</tbody>
</table>

* Including overhead.
In considering Table XXXIV, the reader is struck by the wide difference between the estimate of the Delta Survey (1861) and those of Humphreys (1865) and the Levee Commission (1875). It is not known whether these estimates are comparable. If they were representative of actual contemporary construction costs, the difference between the Delta Survey estimate and later estimates becomes noteworthy. This wide difference may be due in part to the existence of slave labor in the south at the time of the Delta Survey.

Table XXXIV also indicates that the cost of levee construction between the passage of the present flood control act and December 31, 1931 was about 1.5 cents per yard higher than it had previously been. When, however, we consider the increased construction difficulties introduced by the adoption of the 1928 levee section with its increased height and width, and also with its increased borrow pit width, it is apparent that present day costs compare favorably with those of the past. The improvements in levee construction equipment and construction methods have more than counteracted the general rise in construction costs during the last sixty years.

### TABLE XXXV

Average Unit Costs of Levee Construction by United States Since Passage of Act of May 15, 1928 (July 1, 1928 to Dec. 31, 1931)

<table>
<thead>
<tr>
<th>Fiscal Year</th>
<th>Cost per Cubic Yard</th>
</tr>
</thead>
<tbody>
<tr>
<td>1929</td>
<td>31.7 cents</td>
</tr>
<tr>
<td>1930</td>
<td>26.7 cents</td>
</tr>
<tr>
<td>1931</td>
<td>24.8 cents</td>
</tr>
<tr>
<td>1932 (first half of year)</td>
<td>19.8 cents</td>
</tr>
</tbody>
</table>

Total Yardage—201,085,000 Cubic Yards.
Average Cost per Yard for Period July 1, 1928 to Dec. 31, 1931 - 24.5 cents.

It will be noted that the total yardage placed by the United States between July 1, 1928 and December 31, 1931 aggregates 201,085,000 cubic yards. This includes levee construction on certain lower river tributaries as well as on the main river. The total yardage placed by the United States in the same area before the passage of the present flood control act aggregated only 328,342,000 cubic yards. This indicates the vigor with which the present flood control project is being pushed.

Unit levee costs decrease as the lower reaches of the river are approached. This is due to changes in climate, river regimen, and levee specifications. As one travels from Cape Girardeau towards the mouth of the river, the climate becomes increasingly favorable for construction operations, the severity of the winter weather moderates progressively, and the amount of frost decreases. The regimen of the river also improves as it approaches its mouth. Flood warnings at the upper end of the Alluvial Valley are short as compared with those afforded levee contractors in the lower reaches. The rate of bank caving decreases progressively towards the south. This bank caving rate affects the distances to which levees must be set back for safety. In the lower reaches the short distances by which levees are set back makes for low
unit set-back costs. In fact, many of these "set-backs" are built by "construction turn-over", i.e., by use of the material in the old levee for the construction of the new line. This method is relatively cheap as compared with entirely new construction. The narrow, deep borrow pits called for by specifications for levees below Red River also make for cheapness of construction. Above Red River, however, borrow pits are wide and shallow; and the use of construction turn-over is less common; conditions which increase construction costs.

Table XXXV is an illustration of the effects of the national economic situation upon levee construction costs. The United States is now (December 31, 1931) undergoing a period of depression which may be said to have begun with the break in the stock market during October, 1929. The fiscal year 1929 (which ended June 30, 1929) was a period of high prices and apparent prosperity immediately preceding the depression. Levee costs for that year were therefore high. Costs for the year 1931 show the effects of the depression. The fiscal year 1931 was an excellent year for levee construction. The experience of contractors during that fiscal year coupled with the increasingly apparent effects of the national depression is resulting in materially lowered levee costs for the present fiscal year (1932). It is not to be assumed, however, that the downward trend of unit construction costs is entirely the result of the present period of economic depression. The impetus given to levee construction by the passage of the flood control act has already manifested its effect in a very perceptible improvement in construction methods and equipment. The reduction in unit costs is in part the result of this improvement. It appears improbable therefore that average construction costs will again reach the peak of 1929 during the construction period of the present project.

Table XXXVI below illustrates the comparative costs of the different methods of levee construction. This cost is based on a total of 131,401,000 cubic yards of levee constructed or placed under contract between July 1, 1929 and December 31, 1931.

<table>
<thead>
<tr>
<th>Method</th>
<th>Cost Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Haul-in</td>
<td>1.229</td>
</tr>
<tr>
<td>Dragline</td>
<td>.865</td>
</tr>
<tr>
<td>Tower and dragline</td>
<td>.871</td>
</tr>
<tr>
<td>Tower</td>
<td>.978</td>
</tr>
<tr>
<td>Hydraulic</td>
<td>1.223</td>
</tr>
<tr>
<td>Average</td>
<td>1.000</td>
</tr>
</tbody>
</table>

In the above table, "haul-in" includes construction by dump wagons, trucks, industrial railways, etc. The term "tower and dragline" refers to construction by combinations of tower machine and dragline equipment. The unit cost of any given construction method is not, however, a true index of the economy of the method. Each method is peculiarly adapted to operation under certain specific conditions. Under these conditions its use is less expensive than any other method. Irrespective
of comparative costs, therefore, each method has an economic use. The
cwide opportunity given to contractors for competitive bidding upon
Lower Mississippi levee work is a powerful influence toward the selec-
tion of the appropriate economical construction method for each par-
ticular levee job.

In itself, a levee has no value. It is of value only insofar as it
protects valuable land and property from flood damage. This potential
flood damage expressed in money is the value of the flood protection
afforded by any given levee line. In the final analysis, the justification
for a levee system must be found in a comparison between the cost of
the system and the value of the flood protection it affords. The deter-
mination of levee sections and grades is therefore an economic as well
as an engineering problem. The solution of this problem rests on: first,
a determination of the flood menace (flood heights and frequencies);
second, a determination of the potential flood damages; and third, an
estimate of levee construction and maintenance costs. To be econom-
ically justified, the cost of levee construction and maintenance must be
less than the value of the protection which it offers.

This determination of the economics of the problem applies not only
to the levee system as a whole, but also to realignment of existing levee
lines. These local realignments usually take the form of levee "set-
backs" made to avoid breaching by bank caving. Set-backs may be
avoided by the use of bank protection. On the Mississippi River below
Cairo, the first cost of bank protection construction is approximately
$325,000 per mile and the annual maintenance cost is at least 5 per cent
of the first cost. On this basis, the capitalized cost of protecting exist-
ing levee lines by means of bank protection is estimated to be not less
than $650,000 per mile. Based on present day cost figures, it is esti-
ated that the average capitalized cost of providing levee protection on
the Lower Mississippi by the "set-back" method is not more than
$250,000 per mile. This estimate is based on an assumed life of thirty
years for a levee not protected by bank protection. It includes not only
the cost of original construction and of ordinary maintenance; but also
the cost of relocation and reconstruction every thirty years.

It is thus apparent that the cost of levee preservation by bank pro-
tection is normally much greater than the cost of a levee set-back. Fur-
thermore, a levee line which is maintained in its position by bank pro-
tection, is subject to a double attack; attack upon the bank protection
itself and attack upon the levee proper. Levee set-back is therefore the
normal procedure. Bank protection is economically justified only when
the land value of the area which would be sacrificed by a levee set-back
is very large, or where for some reason a set-back is impracticable. In
this connection, the reader is referred to an article by Brig. Gen. T. H.
Jackson entitled "Flood Control on Alluvial Rivers" as published in the
Engineering News-Record, January 22, 1931.

LEVEES AND FLOOD STAGES.

The three principal causes of levee failure on the Lower Mississippi
River have been overtopping, bank caving, and seepage. Minor causes
of trouble are erosion and wave wash. Overtopping is obviously the
result of the miscalculation of possible flood heights. Proper levee
location and bank protection (where justified) provide against caving
LEVEE TOPPED WITH SANDBAGS
bank failures. Therefore, if satisfactory grades and locations be assumed, seepage is the main factor influencing levee design. Levee failures are evidenced by breaks in the levee lines. These breaks are known as crevasses.

Once a levee has been overtopped, its failure is usually assured. A well sodded levee built of a soil (such as seasoned buckshot) which is resistant to erosion may resist for a long time a limited depth of flow across its crown. Ultimately, however, it must fail. It can be saved only by an early fall in flood stages. When a loam or sand levee is overtopped, it is doomed. The grade of a levee line can be raised to a limited extent by "topping" it, usually with sand bags. Where the levee crown is of standard width, the maximum amount by which its grade can be raised varies from 3 to 5 feet. If sand bags are not available, a plank fence backed with earth is fairly effective for topping. When sand bags are used, care must be taken that the borrow pits from which they are filled are so located that the levee is not endangered thereby. The presence of seepage water behind the levee frequently makes the filling of sand bags difficult. Sometimes the borrow is made from the landside levee slope. This should, however, be done only in extreme emergency as it dangerously weakens the levee section. The safeguard against overtopping lies in proper specifications for freeboard. As has been stated above, the levees on the Lower Mississippi are planned for a freeboard of 1-foot above the maximum probable flood. These specifications provide a freeboard of more than 3 feet against any flood approaching the average.

Crevasses caused by bank caving occur during low water as well as during flood. No effective emergency measures exist for the closure of such breaches during flood. As has been said above, proper location and bank protection (where justified) are the correct preventive measures.

Wave wash may sometimes be dangerous. On the lower reaches of the river, permanent wave fences are erected in exposed locations to protect the levee line. Protection has also occasionally been provided by facing riverside slopes with concrete. The most severe wave action of record was that caused by the Gulf storm of September 29, 1915. This storm is described earlier in this chapter. Wave action may be dangerous even though its erosive force is not great. When the levee freeboard is insufficient, breaking waves may saturate the crown and landside slopes thereby materially increasing saturation hazards. Wave fences and topping are tolerably effective emergency methods of wave protection.

Erosion of the levee section occurs along exposed lines and at salient levee angles. Fortunately the retired locations of controlling levee lines and the existence of vegetation in front of them results in flood velocities along their front so slow that erosion dangers are usually non-existent. Erosion is prevented by facing the riverside levee slope with a non-erodible covering, such as concrete, or by spur dikes. The latter method is the normal one. Concrete facing has been used in only a few instances. The spur dikes project out from the main levee line to a distance sufficient to deflect the erosive currents away from the line. The extreme ends of these dikes are usually protected against erosion by concrete or riprap paving. Such protection is not always necessary, how-
ever. For example, heavy woods growing around the outer end of such a dike may so reduce the flow of flood waters past it that no erosion can occur.

The danger of seepage during flood has been the most powerful consideration governing the design of the levee section. Theoretically the levee section must be sufficiently massive entirely to contain the seepage line from its point of entry on the riverside slope until it passes into the foundation under the levee. In other words, there should be no seepage outcrop on the landside slope. As a matter of fact, however, the construction of a levee so massive that no seepage outcrop appears on its landside slope during extreme and prolonged floods is economically impracticable, and is unnecessary from the standpoint of safety. As will be shown later in this chapter, seepage outcrops frequently occur without serious danger to the stability of the levee. The adoption of the banquette levee section was an attempt to produce a structure entirely safe against seepage (see Plates XXXVIII and XXXIX). In practice, however, the banquette section was not entirely satisfactory. Seepage lines often rose higher than had been anticipated with the result that two seepage outcrops frequently appeared on the landside, the lower one on the rear banquette slope and the upper one above the banquette. This upper outcrop was in a location difficult to drain. Experience with the banquette levee (section of 1914) during the 1927 flood has led to the readoption of the trapezoidal section (section of 1928).

Experiments have from time to time been undertaken to determine the effect of seepage on lower river levees constructed under various specifications and from various classes of material. Two sets of experiments, those of 1898 and those of 1929, are noteworthy. The 1898 observations were made on banquette levees built with an assumed 3-foot freeboard. The observations were, however, taken during flood conditions when actual freeboards were 2 feet or less. While the flood of 1898 was comparatively short, it is possible that in some instances the investigations showed saturation conditions somewhat worse than those for which the levees were designed. (See Plate XL.) The methods used in these investigations were crude. Vertical holes were bored at measured distances in the landside section of the levee from the toe to the crown. The distances below the surface at which saturated earth was encountered were recorded and plotted to scale. No effort to determine the hydraulic gradient was made. The saturation lines deduced from these observations were probably somewhat raised by capillarity. The tests of 1929 followed the same form of procedure as those of 1898 except that the field methods were improved over those used in the earlier investigations. The hydraulic gradient was observed. The 1929 flood, which was characterized by a high-water period of unusual duration, furnished an excellent opportunity for observation. Borings were made at thirty-nine different points selected for variation in levee design and in construction materials. The levee sections investigated included the old banquette type, modifications of the banquette type, and the present standard section. At the selected points, borings were made on lines normal to the levee axis. The depths at which moist and saturated earth were encountered were observed and
TREATMENT OF LEVEE ON LAND SIDE TO PREVENT FAILURE DUE TO SEEPAGE

Mississippi River above New Orleans, April 22, 1927

Note drainage ditches at land side toe of levee.
recorded. Pipes fitted with perforated points were inserted into the borings and hydraulic gradients were determined by water column measurements. At stated time intervals, these observations were repeated along lines of borings parallel to the first sets and at short intervals up and downstream from them. The usual saturation line, as revealed by these field studies, was steepest near the river slope of the levee, but rapidly flattened under the middle of the levee section. Under favorable conditions it passed into the foundation within the levee section. However, after prolonged high water, or when the levee was rain soaked, the seepage line almost invariably outcropped on the landside slope. In some cases the seepage line entered the foundation within the levee section, turned upward, reentered the section, and outcropped on the rear slope. This condition usually resulted from standing drainage water on the landside toe of the levee. The outcrop of water above the banquette (on levees of the banquette type) was frequently observed.

The Waterways Experiment Station at Vicksburg has undertaken a study of seepage and saturation phenomena. These experiments have not (December 31, 1931) been completed. They will doubtless, however, add materially to the knowledge on this subject. The results of the 1929 experiment vary widely and can not be regarded as conclusive in any sense. The summarized findings of these tests are, however, given in Table XXXVII below. These results can be regarded only as preliminary in their nature, their value remains to be determined by future and more exhaustive experiments.

**TABLE XXXVII**

Seepage Tests 1929 (Preliminary Results)

<table>
<thead>
<tr>
<th>Class of Levee Material</th>
<th>Slope of Seepage Line</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buckshot (clay)</td>
<td>1:7 to 7.7</td>
</tr>
<tr>
<td>Loam</td>
<td>1:6.5 to 7.4</td>
</tr>
<tr>
<td>Sand</td>
<td>1:8.4 to 10.6</td>
</tr>
</tbody>
</table>

It is interesting to compare these results with the present standard levee sections (see Table XXI). In each case the observed seepage slope is flatter than that provided for by specifications. The present levee grades will afford a freeboard of probably more than 3 feet against any flood approximating the average. This levee section will (for an "average" flood) probably contain everywhere the seepage lines indicated above. The duration of the "maximum probable flood" will be insufficient to raise seepage lines to dangerous points. The entire observed history of the Mississippi River supports this view. The existing standard specifications may therefore be considered adequate.

It is, however, necessary to say a few words here to correct any possible erroneous impression which might be drawn from this discussion. In the Alluvial Valley of the Lower Mississippi River the presence of water behind the levees is almost universal during periods of sustained high water. In confined areas the drainage problem becomes
a serious one. The extent to which seepage can occur through and under the levee without levee failure would seem remarkable to one who had not seen it. The discussion of seepage here must, therefore, be understood to refer only to seepage of sufficient volume to constitute a positive danger to the levee structure.

Seepage failures may be grouped under two heads, failures in the levee section proper and foundation failures. Section failures may be due to a variety of causes. The natural percolation of flood waters through the levee section may be hastened by openings in the levee section itself. These openings may be due to burrowing animals (moles, ground hogs, muskrats, or crawfish); shrinkage fissures; or voids due to the decay of vegetable matter. Animal holes and shrinkage fissures can be eliminated only by proper maintenance. The danger from the decay of vegetable matter has been eliminated by proper specifications and careful inspection during levee construction. It now exists in old structures only.

Under the effect of a sustained high water, a levee may "slide", that is, a large section from near the crown may slide off on an inclined plane. This is, of course, due to failure of saturated material in the structure to support its superimposed burden. Slides usually occur on the landside of the levee although riverside slides occasionally take place. These latter occur usually on a falling river following a period of sustained high water. Surface saturation due to breaking waves or to prolonged rains materially increases the danger of sliding. It will be recalled that levee enlargements are habitually made on the riversides of the existing structures. The planes between old and new earthwork in landside enlargements are so located as to increase the danger of slides during flood.

A levee rarely reaches a condition of dangerous saturation without the occurrence of more or less copious seepage. So long as the seep water remains clear, the levee is fairly safe. When, however, it attains sufficient force to wash out particles of soil, the levee is threatened. Before this danger point is reached protective measures are indicated. These consist of treatment of either levee slope and occasionally of both. River slope treatment usually consists in the placement of a blanket of impervious earth, preferably clay, placed by grab buckets from barges. Landside protection is provided by mats of brush or willow placed on the landside slope and weighted down in place by sand bags. Riverside protection is obviously superior to landside protection and should be employed wherever possible. Since it depends on floating plant, however, it is practical only when the latter may be maneuvered along the levee. When this is impossible, landside treatment must be used. In any case, however, it is essential that drainage from the levee section be facilitated. The provision of proper drainage, to carry the water away from the levee and the areas immediately behind it, is the first principle of levee protection during floods. In all landside treatment care must be taken not to interrupt this free drainage. Brush or willows must not be placed so thickly on the landside slope that they form an impervious mat. The object of this brush covering is to permit the water to flow off freely but at the same time to hold the levee material
FLOOD FIGHTING AT LAKEPORT, ARKANSAS 17 MILES BELOW GREENVILLE, MISS. APRIL 22, 1927.

Note: Levee is topped with board fence and sandbags.
Land side slope is being protected against seepage.
in place. Enough sand bags should be used to hold the brush in place and no more. Overweighting a seeping levee is likely to do more harm than good.

The so-called Fulton slide well illustrates the proper use of emergency protection against seepage. The measures taken there to prevent a crevasse were typical of successful flood fighting measures. The Fulton slide occurred during the flood of 1922 on the left bank of the river about two miles below Arkansas City. The levee at this point was trapezoidal in section, 26 feet high, with a crown of 8 feet and side slopes of 1 on 3. It was built of clay (buckshot). The structure had previously been smaller but had been enlarged to these dimensions by use of a landside borrow pit. The lip of this pit was about 30 feet from the toe of the levee. On April 18, 1922 the flood waters were within 2 feet of the crown. At six a.m. on this date, a levee patrolman reported a longitudinal crack 150 feet long and in the center line of the crown, on the landside of which the levee had settled by about 6 inches. Prompt emergency measures were necessary. A timber bulkhead parallel to the levee was constructed in the borrow pit. Inside this bulkhead a layer of willow poles was laid, care being taken not to obstruct drainage. Upon the willows a berm of sand bags was constructed to stop the sliding at the levee toe and to provide a heavy counterweight on the landside foundation. Meanwhile settlement in the levee section continued and by noon it had reached a maximum of about 7 feet. The rate of settlement was, however, constantly decreasing as a state of equilibrium was approached. The next step therefore was to prevent failure in the reduced section on the riverside of the fissure. A layer of light, bushy willows was now placed on the landside of the levee above the slide, care being taken to facilitate drainage. These willows were held in place by a layer of sand bags which was as light as practicable since the overloading of the weakened levee would have been extremely hazardous. The river slope of the levee was then covered with a blanket of earth placed from barges. These measures were completed in about thirty hours, by which time the slide had ceased. At the point of maximum subsidence, the settlement amounted to 9 feet measured vertically from the levee crown. Protection of a more permanent nature was then afforded by means of the construction of a bulkhead (and back fill) on the levee slope. The emergency measures taken to save this levee followed in an orderly manner. First, the sliding in the levee section was stabilized and foundational movement was prevented. Second, the reduced levee section was fortified by treatment both on the landside and riverside. Third, the section was increased by a bulkhead on the river slope. At all times extreme care was taken to facilitate drainage. Had drainage been interrupted at any time, the levee would probably have crevassed.

The most common manifestation of concentrated foundation seepage is the sand boil. This phenomenon is caused by subsurface percolation which frequently takes place along well-defined lines of weakness such as decayed, buried vegetation. The water passing through such a foundational channel finally comes to the surface behind the levee, usually in the form of a small, clear spring. So long as this water remains clear, little danger is to be apprehended. Usually, however, it soon begins to carry up soil particles thus indicating foundational
erosion. It then increases in size, depositing about itself a roughly circular ring of sand somewhat similar to the crater of a small geyser. The unchecked continuance of such a sand boil ultimately causes levee failure by subsidence and crevasse. When the sand boil is at or near the landside levee toe, this subsidence is very rapid. A large volume of water bursts forth, the levee sinks and breaks, sometimes with a violence almost explosive. Such a phenomenon is locally known as a "blow out". It is possible for a blow out to occur within the levee section as did in fact occur at the Mound Landing crevasse in 1927 (described later in this chapter). The present rigid levee specifications, however, insure against the existence of concentrated percolation lines in the levee itself. Blow outs are therefore generally evidences of foundation failures. The distance from the landslide toe of the levee to the sand boil may vary from zero to several hundred feet, those in close proximity to the levee being, of course, the more dangerous. Sand boils may occur either singly or in numbers. In certain locations they are of frequent and numerous occurrence during every season of prolonged high water. They usually vary in size from very small orifices in the ground surface to fissures 18 inches in diameter. The largest one of record occurred near Vaucluse, Ark. (about 8 miles below Greenville, Miss.) during the flood of 1929. Sand boils have frequently occurred in the vicinity of Vaucluse. In 1922 a very large one occurred during the flood of that year. The sand boil which occurred in 1929 built up a crater about 15 feet wide across the top. The water bubbling up in it could be heard for a distance of more than 1,000 feet. It was, however, successfully controlled. The treatment of a sand boil consists in ringing it with sand bags to such an elevation that active flow is stopped or checked to such a point that it ceases to be dangerous. With the checking of the active flow, erosion in the foundation also ceases and, so long as the sand boil remains in this condition, there is no further danger. When sand boils occur in large numbers, the affected area may be inclosed in a small sub-levee whose ends are joined to the landslide slope of the main levee, and which is high enough to permit water to rise to the point where hydrostatic head through the foundation is equalized on both sides. This may be attained without foundational erosion merely by siphoning water over the main levee. Provision must of course be made for draining these areas after the flood.

Foundational seepage is frequently prevented by placing an overload on the ground surface in immediate proximity to the levee. This usually takes the form of a wide, low berm on the landside. In isolated cases sheet piling has been successfully used to intercept foundational seepage. Its heavy cost, however, prevents the general use of sheet piling on the river.

The rate of seepage through and under levees varies with local soils and conditions. The theory has been advanced that under any given conditions, seepage varies inversely with the silt content of the flood waters. This is based upon the assumption that seepage water deposits silt in porous strata thereby reducing their porosity. Observation of sand levees bears out this theory. A new sand levee seeps copiously at first, but after two or three periods of high water, seepage is much reduced
Sand boil in its initial stages.
Near New Roads, Louisiana.
May 15, 1927.

Sand boil near New Roads, Louisiana.
May 10, 1927.
Land side slope of levee in foreground.
by the deposit of silt in the riverside slope. Whether this phenomenon has any appreciable effect on foundational seepage can not be stated here since the widely varying character of alluvial soils and the activity of the river in cutting its banks render definite conclusions impossible.

Contrary to usual experience, an excellent opportunity was recently afforded near Greenville, Miss., to study the nature and effects of foundational seepage. At a point about three miles south of Greenville, serious trouble from this cause had been experienced in east bank levees for years. A crevasse which occurred here in 1903 resulted in the deposit of a thick stratum of coarse sand in a roughly semicircular area behind the levee line. This crevasse had been closed by a setback loop which was protected against foundation seepage by a series of sub-levees at its landside. In 1913, bank caving necessitated the relocation of the upper end of this loop. The 1913 setback was located across the semicircular sand deposit described above. This sand was excavated along the levee line to the original ground surface and was thrown back into a berm on the landside of the new setback line. This berm appears to have been of considerable effect in preventing foundational seepage. Near the junction of the 1903 and 1913 setbacks, however, the levee for a distance of about 300 feet was protected neither by berm nor by sub-levees. Subsequent investigation developed that the foundation here consisted of several layers of relatively impervious material varying in thickness from 5 to 7 feet. The underlying strata were composed of sand of a fineness approaching quicksand and extending to an undetermined depth. During the flood of 1927 the water remained above 50 feet on the Greenville gage for nine days. However, the section of levee under consideration held. In 1929, however, the stage remained above 50 feet on this gage for a period of twenty-one days. Under this extended water pressure, a partial failure occurred in the levee. The hydrostatic head in the sub-stratum of fine sand apparently gradually built up until it blew up the surface soil at the toe of the landside levee slope over an area of about 4 feet wide by 7 feet long. This was immediately followed by a settlement in a zone 30 feet wide extending entirely across the levee. A diminution of the volume of flow was immediately apparent and emergency measures to hold the levee were successful. After the flood, investigation revealed that a seepage cavity extended through the levee foundation. On the riverside this cavity was about 7 feet below the natural ground surface. It sloped upward to the rear until it came to the surface behind the landside levee toe. At the riverside toe of the levee its cross-section area was about 20 square feet. The water rushing through the cavity under a head of approximately 21 feet would have inevitably crevassed the levee under ordinary conditions. Fortunately, however, the levee was constructed of a fine sandy soil which was unable to support its own weight above the flowing stream. Its prompt collapse along the entire length of the foundational cavity effectively stopped the water and saved the levee.

The magnitude which crevasses on the Lower Mississippi River may attain is best illustrated by a description of the Mound crevasse of 1927. This was the greatest crevasse of record on the Mississippi River. It occurred on the east bank at a point almost due east of Ar-
kansas City and about 3\(\frac{1}{2}\) miles upstream from that town. The Mound Landing levee (which crevassed) had been originally built in 1867 and had subsequently been enlarged three times on its riverside. There is no evidence of planes of weakness in the levee resulting from these enlargements. The levee was built of a light, sandy loam and was, in 1927, fully up to existing standard specifications. It was approximately 20 feet high, its crown was 8 feet, its riverside slope 1 on 3, and its landside slope 1 on 3 to the banquette which was 8 feet below the crown. The banquette was 40 feet wide with a top slope of 1 on 10 and a rear slope of 1 on 5. The break occurred at an angle in the levee which had been protected from flood water erosion by a spur dike projecting downstream from the apex of the angle. A ferry landing was located immediately upstream from this dike. Ramps were provided for vehicular traffic across the levee at this point. The levee crown had been raised by sand bags but a rise during the twenty-four hours immediately preceding the break so elevated the water surface that waves broke over this topping, thoroughly saturating the landside slope which had been previously soaked by an extraordinarily heavy rainfall. About twenty-four hours before the break occurred, an inspection was made and seepage was noted in the rear banquette slope and on the slope above the banquette, where small streams of water were coming out of the levee surface with considerable velocity. Despite energetic emergency measures, the levee failed about 6:30 a.m. on April 21. A witness observed a considerable stream of water coming through the levee at the junction of the rear slope with the top of the banquette. This was accompanied by subsidence in the levee crown followed by a violent "blow out" in the levee section itself above the banquette and immediately downstream of the spur dike. Within six hours the break had widened to about 1,000 feet. It continued to widen at a gradually decreasing rate until it reached an extreme width of 3,047 feet on May 7. Observations on this latter date indicated a discharge of about 450,000 cubic feet per second through the crevasse. At the original break, scour exceeded 100 feet below the original foundational surface. Some discussion has occurred as to the immediate cause of this crevasse. Certainly it was evidenced by a blow out within the levee section itself. It has been claimed that the use of the levee as a ferry landing had weakened the structure. Considering the saturated condition of the levee itself, this theory has some weight. The levee was built of a relatively inferior material which had for days stood in a thoroughly saturated condition with a freeboard considerably less than that for which it was designed. Whatever the immediate cause of the failure, the underlying reason was seepage. Had it been possible to prevent this excessive seepage, the crevasse would probably have not occurred.

Repeated crevasses have been known to occur in the same locality. Bonnet Carre' on the east bank of the Mississippi River about 33 miles above New Orleans (and now the site of the Bonnet Carre' floodway) has been the location of several crevasses. Crevasses occurred there in 1849, 1859, 1871, and 1874. The last crevasse was allowed to remain open until 1883 when it was closed by cooperative construction on the part of the Mississippi River Commission and local interests.
SAND BOIL AT VACULUSE, ARKANSAS 8 MILES BELOW GREENVILLE FLOOD OF 1922.

Exact location of sand boil crater indicated by a ring of sand bags in left center.
TABLE XXXVIII

PRINCIPAL CREVASSES ON THE LOWER MISSISSIPPI RIVER 1890 TO 1931
(Both Dates, Inclusive)

<table>
<thead>
<tr>
<th>Crevasse</th>
<th>Date</th>
<th>Miles below Cairo</th>
<th>Discharge cu. ft. per second</th>
<th>Width</th>
<th>Cause</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nita</td>
<td>Mar. 14, 1890</td>
<td>899 L</td>
<td>402,600</td>
<td>2,892</td>
<td>Rice flume in levee.</td>
</tr>
<tr>
<td>Upper Morganza</td>
<td>Apr. 21, 1890</td>
<td>788 R</td>
<td>31,600</td>
<td>700</td>
<td>Overtopped.</td>
</tr>
<tr>
<td>New Morganza</td>
<td>Apr. 22, 1890</td>
<td>789 R</td>
<td>146,000</td>
<td>2,543</td>
<td>Overtopped.</td>
</tr>
<tr>
<td>Fanny Riche</td>
<td>Apr. 21, 1890</td>
<td>794 R</td>
<td>27,100</td>
<td>845</td>
<td>Waves from steamboat.</td>
</tr>
<tr>
<td>Lobdell</td>
<td>Apr. 22, 1890</td>
<td>816 R</td>
<td>58,900</td>
<td>1,081</td>
<td>Blow out.</td>
</tr>
<tr>
<td>Ames</td>
<td>Mar. 15, 1891</td>
<td>961 R</td>
<td>91,000</td>
<td>1,065</td>
<td>Rice flume in levee.</td>
</tr>
<tr>
<td>Belmont</td>
<td>June 12, 1892</td>
<td>908 L</td>
<td>139,800</td>
<td>1,427</td>
<td>Crawfish hole.</td>
</tr>
<tr>
<td>Sarpy</td>
<td>Juno 13, 1892</td>
<td>937 L</td>
<td>115,900</td>
<td>1,280</td>
<td>Crawfish hole.</td>
</tr>
<tr>
<td>Lakeport</td>
<td>May 11, 1893</td>
<td>497 R</td>
<td>85,200</td>
<td>1,259</td>
<td>Not reported.</td>
</tr>
<tr>
<td>Keigers</td>
<td>May 14, 1893</td>
<td>505 R</td>
<td>47,100</td>
<td>774</td>
<td>Not reported.</td>
</tr>
<tr>
<td>Grand Lake</td>
<td>May 15, 1893</td>
<td>508 R</td>
<td>41,400</td>
<td>980</td>
<td>Not reported.</td>
</tr>
<tr>
<td>Wyllys</td>
<td>May 23, 1893</td>
<td>545 R</td>
<td>230,200</td>
<td>2,950</td>
<td>Not reported.</td>
</tr>
</tbody>
</table>

No major crevasses between 1893 and 1897.

| Flower Lake   | Apr. 4, 1897   | 292 L             | 73,000                       | 2,020 | Sloughing of landside slope.       |
| Williamson    | Apr. 4, 1897   | 309 R             | 39,600                       | 1,320 | Swift current cut levee.           |
| Hubbard       | Apr. 4, 1897   | 312 R             | 28,200                       | 1,130 | Slough of levee.                   |
| Modoc         | Mar. 22, 1897  | 336 R             | 23,400                       | 1,055 | Overtopped.                        |
| Schoolhouse Bayou | Mar. 20, 1897 | 343 R             | 37,200                       | 3,100 | Cut by levee board.                |
| Stop          | Mar. 30, 1897  | 454 L             | 46,500                       | 1,520 | Sloughing of levee.                |
| Deerfield     | Mar. 28, 1897  | 493 L             | 40,000                       | 1,240 | Waves rolling over levee.          |
| Shipland      | Apr. 21, 1897  | 548 L             | 146,800                      | 1,565 | Wave wash and sloughing.           |
| Biggs         | Apr. 16, 1897  | 604 R             | 146,800                      | 1,565 | Wave wash and sloughing.           |

No major crevasses between 1897 and 1903.

| LaGrange      | Mar. 27, 1903  | 480 L             | recorded                     | 3,875 | Unknown.                           |
| Hollybrook    | Apr. 3, 1903   | 542 R             | "                           | 3,000 | Abatement of vigilance.           |
| Albemarle     | Mar. 27, 1903  | 569 L             | "                           | 1,100 | Deficient grade and section.      |

No major crevasses between 1903 and 1912.

| Reelfoot      | Apr. 5, 1912   | 41 L              | 20,500                       | 5,488 | Overtopping.                       |
| Golden Lake   | Apr. 10, 1912  | 190 R             | 77,200                       | 3,785 | Overtopping.                       |
| St. Clair     | Apr. 6, 1912   | 221 R             | 222,200                      | 2,172 | Structural weakness.              |
| Wyanoke       | Apr. 6, 1912   | 241 R             | 124,300                      | 2,411 | Structural weakness.              |
| Modoc         | Apr. 6, 1912   | 327 R             | 104,200                      | 2,480 | Structural weakness.              |
| Ferguson      | Apr. 15, 1912  | 334 R             | 85,500                       | 1,300 | Overtopped.                       |
| Knowlton      | Apr. 15, 1912  | 305 R             | 38,300                       | 716   | Caving bank.                      |
| Lake Beulah   | Apr. 17, 1912  | 404 L             | 208,100                      | 2,400 | Sand boils.                       |
| Panther Forest| Apr. 12, 1912  | 452 R             | 101,000                      | 1,870 | Sloughing landside enlargement.   |
| Upper St. Francis | Apr. 2-6, 1913 | 4 R | 40 above to 7 R | 2,816 | Overtopped by flood. |
### TABLE XXXVIII—Continued

**PRINCIPAL CREVASSES ON THE LOWER MISSISSIPPI RIVER 1890 TO 1931**

(Both Dates, Inclusive)

<table>
<thead>
<tr>
<th>Crevasse</th>
<th>Date</th>
<th>Miles below Cairo</th>
<th>Discharge cu. ft. per second</th>
<th>Width</th>
<th>Cause</th>
</tr>
</thead>
<tbody>
<tr>
<td>Medleys (29 breaks)</td>
<td>Apr. 1913</td>
<td>28 R</td>
<td>6,200</td>
<td>6,200</td>
<td>Overtopped by flood.</td>
</tr>
<tr>
<td>Wilson (2 breaks)</td>
<td>Apr. 9, 1913</td>
<td>190 R</td>
<td>2,815</td>
<td>2,815</td>
<td>Overtopped by flood.</td>
</tr>
<tr>
<td>Random Shot</td>
<td>Apr. 9, 1913</td>
<td>185 R</td>
<td>1,656</td>
<td>1,656</td>
<td>Overtopped by flood.</td>
</tr>
<tr>
<td>Graves Bayou</td>
<td>Apr. 8, 1913</td>
<td>250 R</td>
<td>3,120</td>
<td>3,120</td>
<td>Overtopped by flood.</td>
</tr>
<tr>
<td>Skipwith</td>
<td>Apr. 21, 1913</td>
<td>530 L</td>
<td>3,610</td>
<td>3,610</td>
<td>Blow out.</td>
</tr>
<tr>
<td>Lake St. John</td>
<td>Apr. 27, 1913</td>
<td>684 R</td>
<td>2,700</td>
<td>2,700</td>
<td>Blow out.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No major crevasses between 1913 and 1916.</td>
</tr>
<tr>
<td>Buckridge</td>
<td>Feb. 15, 1916</td>
<td>632 R</td>
<td>116,800</td>
<td>1,030</td>
<td>Sloughing of insufficient section.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No major crevasses between 1916 and 1922.</td>
</tr>
<tr>
<td>Weecama</td>
<td>Apr. 26, 1922</td>
<td>683 R</td>
<td>286,000</td>
<td>3,069</td>
<td>Landslide slough.</td>
</tr>
<tr>
<td>Poydras</td>
<td>Apr. 27, 1922</td>
<td>985 L</td>
<td>450,000</td>
<td>1,100</td>
<td>Caving bank.</td>
</tr>
<tr>
<td>Myrtle Grove</td>
<td>Apr. 22, 1922</td>
<td>1008 R</td>
<td>1,000</td>
<td>1,000</td>
<td>Muskrat hole.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No major crevasses between 1922 and 1927.</td>
</tr>
<tr>
<td>Dorena</td>
<td>Apr. 16, 1927</td>
<td>33 R</td>
<td>Not recorded</td>
<td>3,240</td>
<td>Overtopping.</td>
</tr>
<tr>
<td>Whitehall</td>
<td>Apr. 15, 1927</td>
<td>278 R</td>
<td>2,400</td>
<td>2,400</td>
<td>Caving banks.</td>
</tr>
<tr>
<td>Knowlton</td>
<td>Apr. 20, 1927</td>
<td>366 R</td>
<td>6,042</td>
<td>6,042</td>
<td>Overtopping.</td>
</tr>
<tr>
<td>Laconia</td>
<td>Mar. 29, 1927</td>
<td>376 R</td>
<td>1,225</td>
<td>1,225</td>
<td>Caving banks (during low stage).</td>
</tr>
<tr>
<td>Mound Landing</td>
<td>Apr. 21, 1927</td>
<td>433 L</td>
<td>3,047</td>
<td>3,047</td>
<td>Seepage.</td>
</tr>
<tr>
<td>Cabin Teele</td>
<td>May 3, 1927</td>
<td>586 R</td>
<td>Not recorded</td>
<td>2,400</td>
<td>Overtopping.</td>
</tr>
<tr>
<td>Winter Quarters</td>
<td>May 4, 1927</td>
<td>659 R</td>
<td>1,000</td>
<td>1,000</td>
<td>Sliding (seepage).</td>
</tr>
<tr>
<td>Glasscock</td>
<td>May 1, 1927</td>
<td>732 R</td>
<td>1,230</td>
<td>1,230</td>
<td>Overtopping.</td>
</tr>
<tr>
<td>Brabston</td>
<td>May 1, 1927</td>
<td>734 R</td>
<td>1,082</td>
<td>1,082</td>
<td>Overtopping.</td>
</tr>
<tr>
<td>Bougere No. 1</td>
<td>May 1, 1927</td>
<td>747 R</td>
<td>991</td>
<td>991</td>
<td>Overtopping.</td>
</tr>
<tr>
<td>Bougere No. 2</td>
<td>May 1, 1927</td>
<td>748 R</td>
<td>772</td>
<td>772</td>
<td>Overtopping.</td>
</tr>
<tr>
<td>Junior</td>
<td>Apr. 23, 1927</td>
<td>1010 R</td>
<td>1,010</td>
<td>1,010</td>
<td>Levee rammed by ship.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>No crevasses since 1927.</td>
</tr>
</tbody>
</table>

In the past, attempts have frequently been made to close active crevasses. After a levee has once broken, however, nothing can normally be done until the flood plane behind the levee has built up to such a point that the slope through the break is very flat and currents are correspondingly gentle. Small breaks in low levees can then be closed without great difficulty. The Annual Mississippi River Commission Report for 1890 describes a method of making such a closure. This same annual report describes methods of checking the widening of a crevasse.
The Beulah Lake crevasse was the last large crevasse to be closed successfully. This crevasse occurred in 1912 on the east bank of the Mississippi River opposite the mouth of the Arkansas River. It was caused by foundational failure and reached the width of 2,400 feet. After the flood of 1912, the closure of the crevasse was undertaken but the first rise in 1913 found the work still uncompleted. An emergency sand bag loop was therefore put up to close the remaining gap. This loop failed on January 5, 1913, causing a breach 993 feet wide. This new crevasse was closed before the second rise of 1913 occurred. The Yazoo and Mississippi Valley Railroad built a trestle across the gap and dumped 48,000 cubic yards of riprap between February 6 and April 8. By July 1 the riprap had been covered by 145,000 cubic yards of earth. The second rise of 1913 came against this rock fill and considerable seepage through it occurred at first. This seepage rapidly diminished, however, when the dumping of earth on the fill was begun. The maximum head of water against this emergency fill occurred on April 24 and amounted to about 19 feet.

The closure of large crevasses is usually not justified economically. It is utterly impossible to undertake closure operations until the areas behind the crevasses have been flooded to a depth sufficient to materially reduce the velocity of the current through the crevasse. When this point is reached, most, if not all, of the destruction caused by the crevasse has already occurred. The expenditure of large sums in expensive temporary closure construction is then unjustified. It is therefore necessary that the utmost precautions be taken to prevent crevasses. During floods, depots for tools and materials must be established at various places, and the levee lines must be patrolled day and night. All incipient faults in the levee must be repaired as soon as noted. In general, the care and maintenance of a completed levee line (including flood fighting) rests with the local interests protected by it. Maintenance usually consists of the clearing of the levees of weeds and brush and keeping them properly sodded.

Stock grazing on dry levees is not harmful if permitted under restrictions. Hogs should not be permitted on the levees, however. Fences may be constructed and maintained along the boundary line of the right of way, but permanent fences are not permitted on any portion of the levee embankment. Portable cross fences of approved design may, however, be used. Where highways are placed on any portion of the levee embankment or upon the levee right of way, they must be so constructed and drained that they will not become a menace to the safety of the levee line. The local interests have full, legal authority to protect the lines. It may, however, be noted that alteration or destruction of these lines is against Federal law (act approved March 3, 1899, section 14).
CHAPTER VII.

DREDGING ON THE LOWER MISSISSIPPI RIVER.

INTRODUCTION.

Dredging may be defined as the removal of subaqueous material and its disposal according to the requirements of the work in hand. The earliest dredging tools were drags which were drawn along the bottom of the stream or pool under improvement. The term "dredging", in its broadest sense, includes subaqueous excavation by land plant as well as floating plant. This chapter will, however, be confined to subaqueous excavation by floating plant.

The excavation of pools for man's use, and of channels for his ships, probably dates back to the dawn of recorded history. In connection with their extensive maritime enterprises and naval operations, the Phoenicians, Greeks, and Romans improved natural harbors and constructed artificial ones. They probably carried on dredging operations although, so far as is known, no records of such work remain. The first known use of dredging machinery is believed to have taken place in Holland.

In America, attempts to deepen the channel of the Mississippi were made as early as 1726 when the French colonists undertook operations with harrows dragged across the bars at the river mouths. The earliest use of dredges in this country was, however, probably incident to the construction of canals such as the James River Canal of 1785, the Dismal Swamp Canal of 1794, the Merrimac River Canal of 1792, and the Carondelet Canal (Louisiana) of 1794.

The earliest practical application of the steam engine in dredging was made in England in 1796. The first dredge operated by a double action, high pressure steam engine appears to have been constructed in the United States by Oliver Evans in 1801.

The types of dredges which have been used on the Mississippi River are classified as follows:

(a) Mechanical agitators which stir up the mud on the bottom so that it is carried away by the current.

(b) Scrapers which scrape the material from the tops of shallow bars into deeper water.

(c) Dipper dredges which operate similarly to steam shovels on land.

(d) Grapple or bucket dredges which raise the material from the bottom by means of buckets or containers on lift lines.

(e) Ladder dredges which are equipped with endless chains of buckets designed for subaqueous excavation.
(f) Hydraulic jet dredges which play water jets under high pressure on the bars thus washing the material from them into deeper water.

(g) Hydraulic pipeline dredges which remove material from the bottom by suction and deliver it through long discharge pipes. Hydraulic pipeline dredges on the Mississippi River are of two types, cutter head dredges and dust pan dredges.

DREDGING OPERATIONS PRIOR TO THE ORGANIZATION OF THE MISSISSIPPI RIVER COMMISSION.

The first attempt on the part of the Federal Government to improve the channel of the Mississippi was made in 1837. In that year, Captain A. Talcott, Corps of Engineers, made an extended survey of the Passes with a view to improving the navigation channel to the sea. Talcott recommended a plan for dredging by buckets. In 1839 his plan was actually put into operation but lack of funds made substantial progress impossible. In 1852 the sum of $75,000 was appropriated by the United States for the establishment and maintenance of a navigable channel through the Passes. A board of Engineer Officers was appointed by the War Department to study the problem. This board recommended as follows:

(1) That an attempt be made to deepen the channel by agitating or stirring up the silt on the river bottom.

(2) That resort be made to dredging by buckets if plan (1) failed.

(3) That jetties be constructed in case both plans (1) and (2) failed.

(4) That lateral outlets be closed if necessary.

The board further stated that, if all these measures failed, a ship canal would be necessary. The War Department approved the recommendation for dredging by agitation and a contract was accordingly made with the New Orleans Towboat Company for deepening the Southwest Pass to eighteen feet. This contract was successfully prosecuted for a time, and a depth of 18 feet was obtained in 1853. The failure of Congress to appropriate further funds caused the abandonment of the work, however. By 1856 the results of this work were entirely obliterated.

In 1856 a contract for the improvement of Southwest Pass and Pass a l’Outre was made with Messrs. Craig and Rightor. The specifications called for channels 20 feet deep to be maintained for 4 ½ years. The contractors first attempted the use of jetties, and actually built two jetties. These structures were a failure, however, and attempts to improve the Passes by this means were abandoned for the time being. (See Chapter IX.) The contractors then resorted to scrapers and agitators; dredging by buckets; and blasting the mud lumps. The contract was later revised to call for a depth of 18 feet only and the contractors succeeded in opening channels of this depth through both Passes.
They, however, failed to meet the specification requiring maintenance for 4½ years. The work was again opened to competition and another contract let. The new contractors, however, failed to execute the contract and the War Department was forced to undertake the work direct. Under the plan employed, harrows and scrapers were dragged seaward along the bottom of the channel. By this means a depth of 18 feet was successfully maintained across the bars for a period of one year at a cost of $60,000.

The Civil War caused the discontinuance of Federal improvement operations and extensive work was not resumed until 1867 when dredging operations were recommenced. In 1868-9 the steam propelled dredge Essayons was constructed for dredging in Pass a l'Outre. This dredge was a double ended boat, carrying an excavating screw 14 feet in diameter operated by a double engine at a rate of 60 r.p.m. This screw projected about two feet below the keel of the boat. Its effectiveness was increased by a scraper attached to the upstream end of the boat on either side of the keel. In dredging, the boat was moved downstream with the screw revolving and the scraper in position. The Essayons was intended to establish and maintain a 20-foot channel. While its operation was at first disappointing, it succeeded in deepening the channel from eleven to seventeen feet in two months of dredging. The chief weakness of the dredge seems to have been the propeller blades which required frequent repair at the cost of considerable time.

Shortly after the construction of the Essayons, the dredge McAlester was built for use in Southwest Pass. This boat was equipped with two conical screws, 20 feet long and 5 feet in diameter at their bases. These screws were so set that their points came together under the boat's bow and their bases were so separated that they covered a dredging width of about 20 feet. The axes of the screws were horizontal and the flanges tapered from about 12 inches wide at the base to about 6 inches at the point. When these screws were in operation, it was very difficult to steer the boat. The mud was readily plowed up from the river bottom but was not sufficiently broken up to be readily carried away by the current.

Simultaneously with the construction of the Essayons and McAlester, an appropriation was made for the construction and operation of two scraper dredges on the Upper Mississippi between St. Paul and the mouth of the Illinois River. The scraping device used on these boats consisted of a heavy frame attached to the bow of the boat and carrying heavy cross bars to which were attached six steel cutters or buckets. In operation, the dredge went to the upper side of the bar, lowered its scraper, and backed slowly downstream scraping the sand down into the deep water below the bar. This operation was repeated until the desired depth was obtained. Two side-wheel steamers, the Montana and Oatfrey, were thus equipped. These boats were successful in maintaining navigation depths of from 4 to 4½ feet. They deepened bars from 8 to 18 inches, usually in a very few hours work. Steamboat owners announced that as a result of this work, their packets were able to make regular trips without interruption during the low-water season, a condition of affairs never before known within the memory of pilots of 35 years experience. The steamboat owners
operated a similar scraper boat between Keokuk and St. Louis at their own expense. Scraping operations continued for several years at a cost of about $20,000 per year per dredge, but as the relief was only temporary and had to be repeated annually, this method of channel improvement was finally superseded on the Upper Mississippi by fixed contraction works.

At an early date navigation difficulties were experienced at the mouth of Red River. Shreves Cut-off was made by the Engineer Department in 1831 in the interest of navigation (for description of Shreves Cut-off see Chapters II and X). The cut-off, however, failed in its purpose and the channel (known as Old River) between the Mississippi and Red Rivers subsequently shoaled to such an extent that navigation was seriously hindered. In the year 1877 the State of Louisiana undertook to deepen Old River by lashing together two tugs and a steamer and driving them through the shallow places. This method of channel maintenance was unsatisfactory and shortly thereafter the Federal Government undertook the improvement of Old River for navigation.

The use of hydraulic dredges on the Mississippi River originated in 1877-1878 with the operations of the dredge G. W. R. Bayley in South Pass. This dredge was built in Pittsburgh for the Eads Company which was then engaged in the improvement of the mouths of the Mississippi River. (See Chapters I, IX, and X.) The Bayley was a side-wheel steamboat equipped with a wrought iron suction pipe 27 inches in diameter and jointed to permit a deflection of 30 degrees in any direction. The lower end of the suction was curved and flattened out to a width of 4 feet and was armed with a steel scraper designed to cut to a depth of 8 inches. When not in use, the suction pipe was lifted to a position above water. The discharge pipe was 30 inches in diameter and was so arranged that the spoil could be delivered to barges or placed in bins aboard the dredge. These bins had a total capacity of 512 cubic yards. They could be filled in seven minutes operating time. The capacity of the dredge (delivering the dredged material directly overboard) was approximately 1,000 cubic yards per hour. For a more detailed description of this dredge, the reader is referred to “Dredges and Dredging on the Mississippi River” by Mr. J. A. Ockerson published in the Transactions of the American Society of Civil Engineers, 1898 (No. 838).

DREDGING OPERATIONS SINCE THE ORGANIZATION OF THE MISSISSIPPI RIVER COMMISSION.

With the organization of the Mississippi River Commission in 1879, the development of a definite dredging policy may be said to have begun. In its first report (March 6, 1880) the Commission advocated the use of permeable contraction works for channel improvement and contemplated dredging as an auxiliary to these works to be undertaken in exceptional cases only. The first attempts to deepen the channel, therefore, consisted primarily of contraction works (see Chapter IX) and the development of dredges suited to work on the river was slow. Nevertheless, the need for efficient dredges became increasingly apparent and progressive improvements were forthcoming.
After the cessation of scraping operations in the upper river, several experiments in channel excavation were made above Cairo. In the year 1881 water jets were used successfully on Horsetail Bar near St. Louis. The work was done by four pile drivers equipped with water jets, and lashed with their heads together so that the jets were brought into juxtaposition. In operation, the pile drivers were pulled back and forth across the bar by anchors and steam windlass while the jets were working to full capacity. These experiments are described in the Annual Report of the Chief of Engineers for 1882.

In 1883 an Osgood dipper dredge with six dump scows was employed for dredging in the river above the mouth of the Illinois. The plant was first used on Howard's Bar near LaGrange, Missouri, and opened a channel 1,000 feet long, 110 feet wide, and 5 feet deep in about a week. A part of the dredged material was removed by scows but the major part was dropped beside the cut. This dredge was later used at other places in the Upper Mississippi. The Osgood dipper dredge gave good results under favorable conditions but was too slow for opening temporary channels through bars, consequently in the year 1887 the Engineer Office at Rock Island entered a contract for the construction of an hydraulic dredge equipped with decked flat boats to support 500 feet of discharge pipe. This dredge was named the Success. Its suction head was provided with a water jet to agitate the dredged material. The suction pipe was supported by a double pontoon which pivoted on a triangular base attached to the dredge itself thus permitting the suction head to swing in a semicircle. The maximum width of cut was about 90 feet. The dredge was equipped with spuds. The discharge pipe was supported by flat boats equipped with frames which gave it a slope of about 1 on 100. The dredge was maneuvered by two head lines anchored upstream; being moved and kept on line by swinging first on one spud and then on the other. For a description of the Success, the reader is referred to the Annual Report of the Chief of Engineers for 1887.

In 1888 two ladder dredges were procured for use in Vicksburg Harbor and at the mouth of Red River respectively. These dredges were named the Menge and the Pah-Ute. The Menge could dredge approximately 4,000 cubic yards in 10 working hours while the Pah-Ute had about half this capacity. These dredges generally delivered the dredged material to dump scows which were towed to the dumping ground.

By the year 1891 it had become plainly evident that the contraction works experimentally constructed on the Lower Mississippi at Plum Point and Lake Providence reaches were not successful (see Chapter IX). The Commission therefore appointed a Committee of two from its membership to report upon the practicability of the use of dredges for the maintenance of low-water navigation; and to prepare a project for the construction of a large dredge. Col. Charles R. Suter, Corps of Engineers, and Mr. Henry Flad were the members of this committee. Their report was rendered in 1892 and is contained in the Annual Report of the Mississippi River Commission for 1893. The committee concluded that dredging afforded the only practicable means of low-water channel improvement, and recommended the construction of an experimental dredge for this work.
As a result of this report an experimental non self-propelled hydraulic dredge was built in Memphis in 1893 under the immediate supervision of Mr. Henry Flad. For a description of this dredge (later called the Alpha) and also of the dredges Beta, Gamma, Delta, Epsilon, and Zeta; the reader is referred to a paper by Mr. J. A. Ockerson, Member, Mississippi River Commission, entitled "Dredges and Dredging on the Mississippi River". This paper is published in the Transactions of the American Society of Civil Engineers for 1898 (No. 838).

The experimental dredge was equipped with two suction pipes 30 inches in diameter located in the bow and stern respectively. The bow pipe carried a straight suction head equipped with jet agitators, while the stern pipe was equipped with a curved drag suction. Each suction pipe was operated by a separate pump. The discharge pipe was 30 inches in diameter and was made up of sections 33 feet long connected by strong rubber joints and iron coupling bars. An air chamber on each side of the discharge pipe gave it sufficient buoyancy to float when loaded with 10 per cent or less of sand. The dredge was operated by two head lines attached to hydraulic piles set above the bar to be excavated. In operation, the dredge was pulled upstream on its lines at a rate which varied with the depth of material dredged but which averaged from 60 to 75 feet per hour. When one cut was finished, the dredge was dropped down to the lower end of the bar, and another cut was made parallel to the first. This process was repeated until a channel of the required width and depth was secured.

In the spring of 1895 the drag-suction pipe and pump were removed, some changes were made in the superstructure, and the dredge named the Alpha. The Alpha which now carried only a straight suction with jet agitators was used during the low-water periods from 1895 to 1898, inclusive. In 1900 it was retired from service.

While the experimental dredge was under construction at Memphis in 1893, the dredge Ram was constructed by the Bucyrus Steam Shovel and Dredge Company. This dredge was built for use in Old River (between the Mississippi and Red River). The Ram was a self-propelled stern-wheel craft with a maximum capacity of about 300 cubic yards per hour delivered through 300 feet of pipe. The suction and discharge pipes were 15 inches in diameter. The suction pipe was equipped with a cutter head to break up the material to be dredged. Although the cutter head had been in use for some time, this dredge was the first to be so equipped on the Mississippi River. In operation, the Ram was swung on a stern spud by lines to kedge anchors cast on either side of the bow.

The dredge Ram has been operated more or less continuously since her construction not only in Old River but elsewhere in the lower tributaries and even on the main stem of the Mississippi. This dredge is now being retired from service.

While the Alpha and the Ram were in operation on the lower river, dredging methods were under development on the upper river. In 1896 a jet dredge was constructed for work between St. Louis and Cairo. This dredge known as dredge No. 1 was equipped with four 12-inch discharge pipes with flattened nozzles through which a stream of water was forced against the bar. The bar material was thus stirred up and washed into deep water. The pump capacity of this dredge
was 20,000 gallons per minute. In operation, the dredge attacked the bar from the upstream side and moved downstream until the lower pool was reached. This operation was repeated until the desired depth was obtained. On short reefs the jet dredge appeared to be fairly efficient, but on bars of considerable width the accumulation of sand in front of the jets became too great to be moved economically. Mr. Ockerson describes this dredge in his paper, "Dredges and Dredging on the Mississippi River".

The experimental dredge built by the Mississippi River Commission in 1893 proved the value of dredging for the temporary improvement of the low-water channel, and the Mississippi River Commission proceeded to acquire several dredges for channel maintenance below Cairo. In August, 1894 inquiries were directed to interested parties inviting suggestions as to the design of a new dredge whose capacity was fixed at 1,600 cubic yards per hour. Three firms were ultimately requested to prepare detailed plans and specifications. These plans were opened in December, 1894 and the contract for the construction of a non self-propelled dredge, which was to be known as the Beta, was awarded to the American Hydraulic Dredging Company of Chicago, Ill. The Beta was completed in January, 1896. The tests indicated a capacity of 4,920 cubic yards per hour instead of 1,600 cubic yards per hour called for by specifications. This was due to the terms of the contract which provided a large bonus for increased capacity. The draft of the vessel was, however, 6\(\frac{2}{3}\) feet instead of 4\(\frac{1}{2}\) feet as called for by the specifications. This fault was corrected about two years later by widening the hull from 40 feet to 58 feet beam. The hull was constructed of steel and was 214 feet long. The mechanical equipment consisted of two independent units complete from suction to end of discharge pipe. The two suction pipes were each 33\(\frac{3}{4}\) inches in diameter. Each suction pipe was divided near the forward end of the hull into three smaller sections, each 19\(\frac{1}{2}\) inches in diameter. These three sections were framed together and could be raised or lowered together in one piece through a depth of 20 feet. In all, therefore, the dredge was provided with six intakes each of which had a cutter head. Each pump discharged into a 33-inch discharge pipe line 1,000 feet in length and provided with deflecting baffle plates at its end. The pipes were made up in 50 and 100-foot sections, each section being supported by a steel ponton. The tests and early operations of the dredge developed a number of minor defects. At first all the cutter heads worked in the same direction which had a tendency to pull the dredge sidewise out of the cut. This was remedied by modifying one set of cutter heads so that the two sets worked in opposite directions. After two years operation the cutter heads were replaced by two suction heads equipped with jet agitators. With these modifications, the Beta became the first dredge of the so-called "dust pan" type; a type developed primarily for channel dredging on the Mississippi River. The difference between the Beta and later dust pan dredge designs lay in the fact that it had two suction, capable of independent operation whereas, on later types, (except the Kappa and the Flud) the suction operate as single units. The Beta was in service until 1921 when it was converted into a cutter head dredge and renamed the Omega.

The third dredge to be constructed by the Mississippi River Com-
mission for channel dredging was the non self-propelled dust pan dredge *Gamma* which was built in 1897 by the Bucyrus Steam Shovel and Dredging Company of South Milwaukee, Wisconsin. The capacity of this dredge was fixed by specification at 800 cubic yards per hour through a discharge line 1,000 feet long. The *Gamma* proved satisfactory both in capacity and economy of operation. The hull was of steel; 138 feet long, 38-foot beam, and its draft was about 4.5 feet. The suction was placed in a well, 32½ feet long in the bow of the dredge. The *Gamma* had two intake pipes hinged to permit lowering to the depth of 15 feet. Each pipe terminated in a suction head 8 feet wide. The two suction heads were framed together to form a single unit with an overall width of 19 feet. Each suction head was provided with nine 2½-inch jet agitators. The *Gamma* had a stern discharge pipe 34 inches in diameter and 1,000 feet long. This pipe was made up in 50-foot sections each of which was supported by two cylindrical floats. In 1914 the *Gamma* was remodeled into a twin screw tunnel type craft. This necessitated lengthening the hull by 27½ feet.

By 1925 the *Gamma* had reached the point when substantial repairs were necessary. The hull and dredging equipment needed replacement although the propelling machinery was still in good condition. The dredge was therefore again remodeled. The old suction head was replaced by a new one, 30 feet in width and capable of working to a depth of 18 feet. Agitation was provided by sixty 1½-inch jets. The *Gamma* as remodeled gave satisfactory results. It was kept in service until 1929 when it sank in the river between Cairo and New Madrid.

The non self-propelled dust pan dredge *Delta* was built by the New York Dredging Company in 1897. The hull was of steel; 175 feet long, 38-foot beam, and its draft was about five feet. At the bow, the suction was inclosed by a fender making the overall length 199 feet. The intake pipe was 34 inches in diameter and branched into two pipes each 24⅞ inches in diameter which turned to the right and left square along the outside of the bow and then turned forward again, each branch then separating into two pipes 17½ inches in diameter. There were therefore four intake pipes connected with the suction head. The pipes and suction head were framed together and operated as one unit. Instead of having jet agitators, the *Delta* was provided with a mechanical cutter for breaking up the material to be dredged. This cutter was placed at the outer end of the suction head. It consisted of a steel shaft upon which 22 cast steel wheel cutters were mounted. This shaft was connected to the cutter engine by means of sprockets and chains. These mechanical cutters proved unsatisfactory and were later replaced by jet agitators. Like the other channel dredges, the *Delta* was a stern discharge dredge. The pipe was 1,000 feet long constructed in 50-foot sections connected by rubber sleeves provided with a baffle plate at the end. The pipe was supported by ponton floats on each side. These floats were U-shaped in section and sustained the pipes in yokes. The *Delta* was kept in service until the present year (1931). It is now being dismantled.

In the year 1897 the construction of the two non self-propelled dredges *Epsilon* and *Zeta* was begun by the Springfield Boiler and Manufacturing Company of Springfield, Illinois. These dredges were ready
DREDGE GAMMA
Dredging crossing of Island 35
August, 1929
for service in the spring of 1898. The stipulated capacity of each of the two dredges was 1,000 cubic yards of ordinary river sand per hour dredged from a maximum depth of 15 feet and delivered through 1,000 feet of floating pipe. The dredges were of the dust pan type and were alike throughout except that the Epsilon was equipped with water jet agitators while the Zeta carried mechanical agitators. Their hulls were of steel; 157 feet long, 40-foot beam, and had working drafts of about 4 feet. The suction operated in wells in the bows. The design of the suction on these dredges differed somewhat from that of the Gamma. Two suction pipes ran forward from the pump separating until they were 12 feet apart at the bow and suitably hinged to permit dredging to a maximum depth of 15 feet. Each suction pipe terminated in a suction head ten feet wide. Both suction were framed together and operated as a single suction 20 feet wide overall. The Epsilon was provided with jet agitators. The Zeta, on the other hand, had a mechanical agitator consisting of a vertical scraper or harrow attached to the front end of the suction head and given an up and down motion by means of a bell crank. This mechanical agitator proved unsatisfactory and was later replaced by a jet agitator. The discharge pipes of these dredges were 32 inches in diameter and were built in 50-foot lengths. The discharge pipes were supported by pontons 8½ feet wide. Each pipe section and its supporting ponton were built as a single unit. The dredge Epsilon was dismantled in 1927. The Zeta is still in service.

The recommendations of the dredging committee (Suter and Flad) contemplated self-propelled dredges but up to the year 1899 no self-propelled dredges were constructed. In that year the construction of a self-propelled dredge was undertaken. This new dredge was known as the Iota. It was built by the Springfield Boiler and Manufacturing Company and was completed in 1900. Its hull was of steel; 192 feet long, 44-foot beam, and 4-foot draft. It was equipped with two 24-inch suction pipes each terminating in a separate dust pan suction head and capable of working to a maximum depth of 18 feet. The two suction heads were connected so as to operate as a single unit but this connection was not rigid. This lack of rigidity at first caused much loss of time through breakdown. In 1902, however, the heads were rigidly joined. The overall suction head width was 23 feet. The agitators were of the jet type. The dredge was a side-wheel craft with paddle wheels set further forward than was usual for this method of propulsion. This design, however, facilitated swinging the head of the dredge. The dredge was constructed for stern discharge. The discharge pipe was 32½ inches in diameter made up in 50-foot lengths, each supported on a scow shaped ponton.

The Iota has been used for channel maintenance ever since its construction. It has been improved and altered from time to time. It is still in operation.

The two dredges Kappa and Henry Flad were sister dredges of the self-propelled dust pan type. They were built by the Bucyrus Steam Shovel and Dredge Company and were completed in 1901. Their hulls were of steel; 192 feet in length, 44-foot beam, and their draft was 5 feet. Each dredge had two 24-inch suction pipes, each of which terminated in a suction head capable of independent operation. The overall width of port and starboard suctions together was 23 feet, 6 inches. These
independent suctions were designed to facilitate operation by permitting the two suctions to operate at different levels. The suctions of these two dredges proved unsatisfactory however, and in 1902 the dredges were modified so that their suctions operated as single units. The dredges were equipped with jet agitators and carried 32-inch stern discharge lines 500 feet long made up of sections 50 feet long. The discharge lines were supported by scow shaped steel pontons. The dredges were side-wheelers of the same general pattern as the Iota but their propelling engines were more powerful. The Kappa continued in service until 1929. In that year it was loaned to the Kansas City District in the Upper Mississippi Valley Division. While in that district it sank in the Missouri River in 1930.

The Henry Flad was continued in service until 1930 when it was was completely remodeled. At present this dredge is considered to be practically new.

The dredge B. M. Harrod was constructed in 1907 by the Springfield Boiler and Manufacturing Company of Grafton, Ill. It is a self-propelled dust pan dredge equipped with a single pump outfit. The hull is of steel 210 feet in length, the beam is 44 feet, and the draft when loaded is 5 to 6½ feet. The dredge is of the same general type as the Kappa or the Flad but is larger than either of those dredges. The suction head can be lowered to a maximum depth of 20 feet. Its overall width is 32 feet. The suction was originally designed for dredging either up or downstream. Downstream dredging did not prove successful, however, and the dredge is now habitually used for dredging upstream only. The Harrod was provided with double rows of jet agitators, the upper row being designed to prevent caving sand from closing the intake or burying the head when dredging against a deep face. The dredge is a side-wheeler. Its size and draft sometimes make it unwieldy over a shallow crossing.

In recent years Memphis harbor has been shoaling badly (see Chapter II). In 1917 a project was adopted for the maintenance of this harbor. This project has required dredging after practically every high-water season since its adoption. Between 1916 and 1921 dust pan dredges were used for this purpose and, while they succeeded in keeping the harbor open, their operation was very expensive. This was due to the fact that the pumps of dust pan dredges are not adapted to the high lift and long discharge line necessary in harbor dredging. For harbor dredging and similar purposes, therefore, the old dust pan dredge Beta was converted into a nonself-propelled cutter head dredge of standard design. After this conversion this dredge was renamed the Omega.

The dredge Taber was built in 1912 for the Little Rock District. The dredge is self-propelled, being equipped with a stern wheel. Its hull was constructed of steel. The overall dimensions were: length, 206 feet; beam, 44 feet, 4 inches; and draft, 4 feet. It was used on the Arkansas River and at New Orleans, and was later assigned to the engineer district at Norfolk, Va. In 1923 the Taber was transferred back to the Mississippi River Commission. The Taber was originally a dust pan dredge but, after her transfer to the Mississippi River Commission, it was converted into a cutter head dredge. The Taber has a capacity of from 500 to 1,000 cubic yards per hour through a 24-inch
discharge pipe (later reduced to 20 inches diameter). The suction pipe was 22 inches in diameter. In 1930 the Taber was transferred to the Louisville Engineer District on the Ohio River.

In addition to the dredges described above, three other dredges have been employed on the tributary streams of the Lower Mississippi, the hydraulic dredges Waterway and McGregor and a dipper dredge known as No. 68. The dredge Waterway was built in 1912 by the Dubuque Boat and Boiler Company. It is a self-propelled (stern wheel) cutter head dredge with a 16-inch suction pipe. The McGregor was originally a sister dredge of the Taber. It was built at the same time as that dredge and was assigned to the same engineer district. Later the McGregor was converted from a dust pan dredge into a cutter head dredge; and still later it was reconverted into a dust pan dredge. It has been transferred away from the Lower Mississippi Valley Division of the Engineer Department. Dredge No. 68 was built in 1896. It is a dipper dredge capable of working to a depth of 18 feet. This dredge has been used for operations on the White and Arkansas Rivers.

At present the following dredges are in service on the Lower Mississippi River; the Zeta, the Iota, the Henry Flad, the B. M. Harrod, and the Omega. Two additional dust pan dredges (the Ockerson and the Potter) are now under construction by the Dravo Contracting Company of Pittsburgh, Pa., and are scheduled for completion in August, 1932. These two new dredges have steel hulls; 214 feet long, 46-foot beam, and 5-foot, 3 inches-draft. They are self-propelled, twin screw, tunnel boats. They have 32-foot suction heads capable of dredging to a maximum depth of 20 feet. Discharge is through the sides of the hulls.

The act of January 21, 1927 placed the upper limit of seagoing navigation on the Lower Mississippi at Baton Rouge and adopted a project for the maintenance of a navigation channel 35 feet deep between Baton Rouge and New Orleans. Navigation improvement in this reach was placed under the jurisdiction of the First New Orleans District of the Engineer Department. The maintenance of a channel for river navigation was thus automatically limited to the reach of the lower river above Baton Rouge. Recently navigation improvement between New Orleans and the Head of the Passes has also been placed under the First New Orleans District. The lower limit of Commission jurisdiction over navigation improvement is, therefore, fixed at Baton Rouge. The jurisdiction of the Commission for flood control below Baton Rouge remained unchanged, however.

The reorganization of the Mississippi River Commission following the passage of the present flood control act (act of May 15, 1928) was marked by a change in policy as to navigation improvement on the lower river. It will be recalled that the Commission in 1896 undertook the maintenance of a navigation channel by dredging only. Between that date and 1928 the only regulating works constructed on the lower river were designed to close objectionable chutes and secondary channels. These works are described in Chapter IX. In 1928, however, a policy of river regulation by the systematic construction of contraction works was adopted. (See Chapter IX.) The completion of the contraction program will require several years during which channel dredging will still be the main means of navigation improve-
ment. As the contraction program progresses, however, the need for dredging should become progressively less and less. With the successful completion of this program, the need for dredging should be reduced to a minimum. Dredging on the lower river will never be entirely unnecessary, however. During abnormal periods of extreme low water, some dredging will no doubt be needed to a limited extent to supplement the contraction works. Moreover, a limited amount of dredging will probably be necessary in reaches the improvement of which, by contractions, is not economically justified.

A word concerning the organization of the dredging force is of interest. Prior to the creation of the Mississippi River Commission, dredging operations were under the charge of the local engineer offices. This practice continued until the year 1896 when responsibility for channel dredging on the lower river was transferred to the Office of the Secretary, Mississippi River Commission. The adoption by the Commission of a policy of channel maintenance by dredging was thus followed by the centralization of all dredging activities under one head. Meanwhile dredging on the lower tributaries was performed by local District Offices of the Engineer Department not under Commission jurisdiction. By 1918 dredging operations had become so extensive that a Dredging District was organized with headquarters at Memphis, and having jurisdiction over all dredging in the main river below Cairo. As a part of the reorganization of the Mississippi River Commission in 1928, the Dredging District was abolished and the responsibility for dredging was again placed upon the District Engineers. This is the organization at the present time. The reorganization of the Engineer Department in 1929 (and the consequent creation of the Lower Mississippi Valley Division) has simplified the dredging organization. The Office of the Division Engineer, Lower Mississippi Valley Division and of the President, Mississippi River Commission, are the same. The Memphis and Vicksburg Districts operating under the Mississippi River Commission also have jurisdiction over the tributaries in their capacities as districts of the Lower Mississippi Valley Division. The present organization has therefore a flexibility not possessed by the Dredging District. At the present time the dredges Zeta, Iota, Flad, Harrod, and Omega belong to the Memphis District and the dredges Ockerson and Potter are under construction for that district. This concentration of dredging plant in the Memphis District is due to the fact that the problem of channel dredging lies almost entirely within that district.

THE DEVELOPMENT OF THE DUST PAN SUCTION HEAD DREDGES.

We have seen that dredging operations on the Lower Mississippi were first undertaken by means of scrapers and agitators. We have also seen that various other dredging methods were used either on the lower river or the upper river during the years preceding the adoption of hydraulic dredging on the Lower Mississippi. The subsequent history of the lower river dredging is largely the history of the development of the dust pan type hydraulic dredge.

The G. W. R. Bayley and the Success have been described earlier in this chapter. These two dredges were forerunners of a type of suction dredge which was ultimately to develop into the present day "dust pan"
dredge. The experimental dredge (later the Alpha) built by the Mississippi River Commission in 1893 was the first step in that development. As has been stated earlier in this chapter, this dredge was equipped with two suction, a bow suction and a stern, or drag suction. This dredge furnished therefore a direct comparison of the relative merits of the two types of suction. Experience proved the superiority of the bow suction and when the dredge was remodeled in 1895 the stern suction was discarded.

The requirements of lower river channel dredging call for a dredge capable of dredging a wide cut. The dredge Beta (built in 1896) was built to meet that need. This dredge was in fact two separate dredging plants mounted on a single hull. Each plant unit operated three cutter heads framed together to operate as a unit. The cutter heads failed to operate satisfactorily and were replaced by suction heads equipped with jet agitators. As thus rebuilt the Beta became the first dust pan dredge on the Lower Mississippi.

The dredge Gamma (built in 1897) was an improvement on the Beta. The Gamma had two suction heads equipped with jet agitators. These two suction heads were, however, framed together and operated as a single unit. The Delta (built in 1898) was a new departure in dust pan dredge design in that, instead of being equipped with jet agitators, its suction head was armed with a series of rotary cutters. These cutters proved unsatisfactory and were replaced by jet agitators in 1901-2. The last effort to substitute mechanical agitation for jet agitation was made on the dredge Zeta (built in 1898). The Epsilon and Zeta were twin dredges except that the former was equipped with jet agitators while the latter carried a vertical scraper on the front of the suction head. A direct comparison of the methods of agitation was thus afforded by these two dredges. The jets proved to be greatly superior to the scraper and the Zeta was equipped with jets in 1899. Since that year jets have been considered the standard method of agitation for dust pan dredges.

The Iota (built in 1901) was a step forward in lower river dredging practice. This dredge was the first self-propelled dust pan dredge to make its appearance on the lower river. The non self-propelled dredges hitherto constructed required the use of auxiliary plant such as towboats and pile drivers which the adoption of a self-propelled design now rendered unnecessary. The Iota possessed a further advantage in its superior mobility. A great factor in the usefulness of a channel dredge lies in its mobility: The dredge must be capable not only of efficient operation but also of speedy transfer from point to point as the situation changes.

The Iota, however, possessed one weakness. The two sections of its suction head were connected to operate as a single unit but this connection was not rigid. This caused much loss of time due to breakdown and a rigid coupling was provided in 1902.

The sister dredges Kappa and Henry Flad (built in 1901) were self-propelled. These dredges were peculiar in that each dredge possessed two suction heads capable of independent motion. They were not, however, equipped with duplicate pumping plants as was the dredge Beta. This duplicate suction head design was intended to overcome a difficulty experienced in the practical use of dust pan dredges.
When a dredge starts a cut across a reef, one side of the suction head frequently rests upon the reef while the other side does not. This is especially likely to be the case when the axis of the reef lies at an acute angle with the axis of the proposed cut, or when the dredge lies at an angle to the cut. When the suction head operates under this condition, one side is likely to become clogged while there is a free flow of water through the other. The suction head design of the Kappa and the Flad was intended to overcome this difficulty by permitting dredging at unequal depths under the port and starboard sides of the dredges. Operation, however, failed to demonstrate the practical superiority of the design and in 1902 the two dredges were modified so their suction heads operated as single units.

The dredge B. M. Harrod (built in 1907) was designed for both upstream and downstream dredging. It was anticipated that this design would be capable of both rapid and economical dredging since in operation no time would be lost dropping back from the upper to the lower end of the dredged cut. In practice, however, the Harrod did not give good results when dredging downstream and, as now operated, it dredges upstream only. The jet agitators on the Harrod were, however, an innovation in design. The previously built dredges had been equipped with one row which was placed below the suction intake. The Harrod was, however, equipped with an additional row of jets placed above the intake. This upper row was very successful in preventing clogging in the suction intake when the dredge was working against a deep face. These double lines of jets have become standard on dust pan dredges.

Thus there has been progressively developed a powerful suction head, equipped with jet agitators and designed to operate upstream. The latest developments in suction head design are illustrated in the dredges Potter and Ockerson now nearing completion. These suction heads are 32 feet wide and capable of dredging to a maximum depth of 20 feet. They are equipped with double rows of 1½-inch jet agitators. The design of these suction heads is illustrated on Plate XLIII.

THE DEVELOPMENT OF DISCHARGE PIPES FOR DUST PAN DREDGES.

A progressive development in the design and operation of discharge pipes has accompanied the development of the dust pan dredge. This development is illustrated on Plates XLII and XLIII. Discharge pipe design was at first governed by the assumption that it was necessary to deliver the dredged material into the pool below the bar across which a channel was being dredged. This of course necessitated a very long discharge pipe. The discharge pipe of the experimental dredge (later the Alpha) was 30 inches in diameter and 1,000 feet long. It was made up in 33-foot sections coupled together by rubber joints and iron coupling bars. Buoyancy was provided by two air chambers set above the pipe so that the pipe itself was entirely submerged during operation. In practice, the buoyancy thus provided was not enough and additional flotation was provided by fastening wooden barrels along the line. No baffles were provided for the discharge end of the line and the whip of the discharge is said to have caused the line to swing back and forth, and to break at the joints between sections.
The submerged pipe had certain obvious disadvantages, among which may be mentioned difficulty of coupling up and the impossibility of adequate inspection while the dredge was operating. An effort was made to effect improvements in the construction of the discharge pipes for the dredge Beta (built in 1896). It will be recalled that the Beta possessed two complete dredging units complete from suction head to discharge pipe. The discharge pipes were each 1,000 feet long and 33 inches in diameter. Pipes were made up in 50 and 100-foot sections which were coupled up in operation so as to break joints in the two lines. Flotation was provided by pontons whose bottoms were rounded and whose decks were deeply recessed to receive the pipe sections. This represented a distinct advance in discharge pipe design. The pipe itself was supported above the water and the ponton was a separate unit. This feature facilitated maintenance and lessened the danger of the pontons being filled with water as a result of discharge pipe leaks. The design of the pontons themselves was faulty, however. They lacked sufficient buoyancy and turned over easily, defects which necessitated the addition of supplemental floats. The baffle plate made its appearance on these discharge pipes. These plates were rigid, but were each equipped with a moveable leaf (between the baffle and the discharge pipe) which afforded a method of partially controlling the deflection of the pipe lines.

The defects discovered in the discharge lines of the Beta influenced the design of the discharge line of the Gamma (built in 1897). This line was 34 inches in diameter and 1,000 feet long; and was made up in 50-foot sections. Each section was supported by two cylindrical floats. These floats were not adequate and additional flotation was later provided by supplemental cylinders. The discharge pipe of the Delta (built in 1897) was similar in dimensions and section lengths to that of the Gamma. It was also supported by side floats, although these were somewhat different in design than those of the Gamma.

The discharge pipes of the dredges Epsilon and Zeta (built in 1898) were patterned somewhat after the design of the discharge pipe of the Beta. The supporting pontons were, however, designed for greater stability against overturning. The design was faulty in one important point, however. The pipe section and ponton were built in a single unit which possessed the disadvantage that leaks in the discharge pipe might fill the supporting ponton with water.

The discharge line of the dredge Iota (built in 1900) was a definite advance in discharge pipe design. This pipe was 32\(\frac{1}{4}\) inches in diameter made up in 50-foot sections, each of which was supported entirely above the water line by a double ended ponton. Each ponton carried a turntable track upon which the pipe supports travelled. Thus the ponton could be turned to head upstream and lashed in that position even though the discharge line was at a considerable angle to the direction of the current. The discharge pipe was equipped with a pivoted baffle which was controlled by a bridle operated by a capstan mounted on the last barge in the line. This permitted adjustment in the angle of deflection of discharge from the pipe; did away with whip; and controlled the direction of the discharge pipe. The length of the discharge pipe was 500 feet whereas previous pipes had been 1,000 feet long. The practice of discharging into the pool below had been found by experience
to be unnecessary. A shorter pipe which discharged outside the limits of the dredged cut gave satisfactory results provided the dumping ground was so selected that the current carried the dredged material away from, rather than to, the dredged cut. The dredge Iota may be said to have been equipped with the first pipe line of present day standard design. This design may be summarized as a pipe supported entirely above the water by pontons and equipped with a pivoted baffle. A pipe of this type has no whip and is capable of discharging in a direction at a considerable angle to the direction of the current. The design of the discharge pipes of the Kappa and Henry Flad (built in 1901) was quite similar to that of the Iota.

With the construction of the dredge Harrod (1907), still another departure was made in discharge pipe design. The pipe-line sections were each 100 feet long and were supported by round, saucer shaped floats with slightly crowned tops recessed to receive the pipe. This design possesses certain advantages in that it requires no turntable on the ponton and permits a slight reduction in the elevation of the pipe above the water surface thus reducing the discharge head against which the dredge pump must work.

The discharge pipes proposed for the dredges Potter and Ockerson are typical of present approved practice. These pipes are each 32 inches in diameter and 500 feet long. They are made up in 50-foot sections each of which is supported by a scow type ponton 40 feet long and 18 feet wide. Each ponton is equipped with a turntable. Pipe sections are connected by flange joints provided with heavy rubber washers (see Plate XLIII).

Hitherto, Lower Mississippi dust pan dredges have been built for stern discharge. The Potter and Ockerson are, however, designed for side discharge from either side. This design is considered fully as efficient as the stern discharge design and is a simpler and lighter type of construction.

RADIO COMMUNICATION.

The radio has proved of great value in dredging. Since 1923, dredges and auxiliary craft in the Dredging District (and later in the Memphis Engineer District) have been equipped with radios. In addition, a radio station was established at district headquarters. Radio has also proved to be of great value in the collection and publication of navigation information.

DREDGING FOR CHANNEL IMPROVEMENT.

The needs of navigation are the sole reason for channel dredging. Dredged cuts must therefore be located primarily with regard to their practicability for barge traffic which now constitutes by far the greater proportion of river navigation. The small, easily maneuvered packets which formerly constituted the bulk of river navigation were able to use channels impracticable for the heavy, unwieldy barge tows of today. The development of river navigation has, therefore, changed dredging requirements and has considerably increased the amount of dredging to be done.
It is impossible to fix definitely the stages at which channel dredging must be undertaken in any given reach of the lower river. As has been stated in Chapter IV, stage rises are characterized by sedimentation in the channel crossings. After periods of high water, the falling stages tend to erode the high crossing bars which the flood has built up, but this erosion does not keep pace with the rate of stage decline and a point is ultimately reached where the difficulties of navigation may be even greater than those experienced during the lower stages which occur later, and after the current has completed its scouring action. Good navigation conditions can not, therefore, always be expected to prevail whenever the stages are moderate. For example, after the flood of 1927, the depths over a great number of crossings in the Memphis District were temporarily less than 9 feet although the stages were some 10 feet higher than those at which dredging normally commenced. On the other hand, periods of protracted low water usually lower the critical stage at which dredging must be undertaken. Experience has shown, however, that, below certain critical stages, channel difficulties may normally be expected. In the Memphis District this critical stage is about 18 feet on the Cairo gage. In the Vicksburg District the critical stage is about 20 feet on the Vicksburg gage.

A rigid surveillance of the channel throughout the low-water season is maintained by inspection and survey boats. In the Memphis District, where practically all of the dredging is done, the channel is divided into reaches of suitable length and a survey boat is assigned to each reach for continuous service during the low-water season. When, upon inspection, a crossing is found to be in danger of shoaling, a detailed hydrographic survey is made. This survey, when plotted as a map, shows the configuration of the bar and indicates whether dredging will be required.

So far as navigation requirements will permit, a dredged cut should be so located that it will tend to maintain itself. In other words, it should be so placed that it will be subjected to the maximum scouring effect of the current at low water. The cut should, therefore, coincide as nearly as possible with the natural channel which the river will take across the bar when low stages are reached. The prediction as to the location of this natural low-water channel is sometimes quite difficult since the thalweg over a crossing shifts widely with stage fluctuations. The successful prediction of the thalweg location depends not only upon theoretical knowledge but also to a large extent upon practical experience at the crossing in question.

Other considerations being equal, a location should be selected which requires a minimum amount of dredging. This condition would seemingly be satisfied by the selection of the location giving the shortest length of cut over the bar. The slope through such a cut will be steeper than that through a longer cut through the same bar. The velocity of discharge (and consequent scour) will therefore be greatest through the short cut. It is, however, a mistake to locate a cut purely on this basis. A short cut athwart the direction of the low-water current will not only be difficult for navigation, but will also silt up despite the relatively steep slope through it. Such a channel will be more difficult and expensive to maintain than a longer channel which coincides with the natural direction of low-water discharge.
The location of the spoil bank has a definite bearing upon the value of the new channel. The hydrographic survey of the crossing to be dredged sometimes indicates alternative channels either of which the low-water current may ultimately follow. After one of these channels has been selected for dredging, the other channel should, if possible, be used as a dumping ground for the discharge from the dredge. This closure of the alternate channel by spoil will concentrate the low-water discharge down the dredged cut with a resultant increased scour.

The operation of a dust pan dredge is as follows: The dredge first proceeds to the upstream end of the bar and jets down two hydraulic piles fifty or more feet apart and above the upper end of the proposed cut. To these piles are attached two hauling cables which pass around hauling drums located on the dredge just aft of the suction. These hauling cables are crossed in front of the dredge, that is, the cable from the starboard hauling drum is attached to the port pile while the port cable is attached to the starboard pile. This rig provides for ease of maneuver when the dredge comes up on its hauling cables, and prevents it from being pulled sidewise in the cut. After setting the piles, the dredge drops downstream by slacking away on its hauling cables until it reaches the lower end of the proposed cut. This maneuver must be carried out with great care. If the cables are not properly paid out, their weight as they lie on the river bottom will tend to pull the dredge off its course. When the dredge is in position at the lower side of the bar, the suction head is lowered to the desired depth and held in place by the hoisting frame and tackle. The main pump and jet agitators are then started and the dredge pulls forward on its hauling cables fast enough to keep the suction head up against the face of the cut. During operation, soundings are periodically taken from a sounding stage forward of the suction head and off the starboard and port bow. The rate of forward progress depends upon the depth of the cut and the character of the bottom. The suction heads on the present channel dredges are capable of working against a face from 12 to 16 feet in height. The rate of upstream progress may vary from less than a foot to about eight feet per minute depending upon character of the bottom and the depth of the cut. The dust pan dredge is probably capable of best work when dredging against a face between 4 and 8 feet high. When dredging against a low face the percentage of solids pumped is very low. When, on the other hand, the face of the cut is very high, there is danger that caving will bury the suction head; or else that large quantities of material will pass around and over the suction head to be deposited in the dredged cut a short distance astern. It is not to be supposed that all the material from the cut normally passes through the discharge pipe of the dredge. A very large proportion of it is stirred up and is carried in suspension by the current into the pool below the bar. The amount of the material so moved depends upon local conditions. The ability to open up quickly a cut through a bar constitutes a great advantage of the dust pan dredge over other types for channel dredging. Under favorable conditions, the scour of the current through the initial cuts rapidly enlarges them and materially assists the dredge in opening a navigation channel.

After completing 8 to 10 cuts, the dredge relocates the hydraulic piles, drops back, and begins a second series of cuts parallel to the first
The location of the dredge with respect to previous cuts is determined by sounding, by ranges (indicated by floats or flags), and by sextant observations made upon previously located landmarks. In the Memphis Engineer District an instrument man is placed aboard each dredge. He periodically plots the location of the dredge upon a large scale plat of the bar being dredged. The master is thus able continuously to check the location and course and to insure against its leaving undredged areas in the channel. The position of the end of the discharge pipe may also be plotted thus controlling with accuracy the deposit of dredged material in spoil banks.

The problems involved in the selection of dredged cuts are best illustrated by examples. The following examples are believed to be typical. Dredging done at Mac Towhead and Baskett Bar (266 miles below Cairo) during the low-water season of 1930 is illustrated on Plate XLIV (A) and (B). This crossing is in the middle of an eleven mile reach which has a fairly uniform slope averaging 0.63-foot per mile. On July 25, 1930 the thalweg made an easy connection between the pool above the crossing and that below. (Line AB Plate XLIV (A).) On this date, however, the river stood at 1.5 feet on the Mhoon Landing gage (273 miles below Cairo) and was falling. Experience showed that as the stage approached low water, the thalweg swung over toward a location shown approximately by the line CD on Plate XLIV (A). This thalweg position across Baskett Bar made a "square" turn in the river channel, an extremely undesirable condition so far as navigation was concerned. An attempt was therefore made to keep the channel open along the axis AB and, since the crossing was beginning to shoal up, the dredge Zeta began work at this point. In 23 days the Zeta pumped for a total of 210.3 hours. Nevertheless the effective yardage moved was only 95,895 cubic yards, although the capacity of the dredge was approximately 1,000 cubic yards per hour. In other words, the current refilled the channel during this period at approximately one-half the rate of dredging. The survey of August 26, 27, 1930 showed that project dimensions still existed in this channel but the dredged cut was deteriorating rapidly and the river was meanwhile developing a channel on the axis CD. Between August 29 and September 1 the Harrod dredged for 60 hours along that axis and during this period moved 102,035 cubic yards of material. The Harrod's capacity is from 1,500 to 2,000 cubic yards per hour. It is thus apparent that the current did not hinder the work although it may not have helped materially. This new dredged channel is shown on Plate XLIV (B). Although it was not an easy channel for navigation, no further trouble from shoaling occurred during the season. A survey was made the following summer (June 3-8, 1931) after the winter rise had crested and subsided. This survey is shown on Plate XLIV (B). The return of the channel to the vicinity of the axis AB is apparent.

Dredging at Ashport Bar (155 miles below Cairo) during the low-water season of 1928 is illustrated on Plate XLV (A) and (B). This bar is located in Plum Point reach which has always been characterized by navigation difficulties. Early attempts to improve this reach by training works are described in Chapter IX. The present project for the improvement of this reach is also discussed in Chapter IX. Ashport Bar is located near the foot of an eight-mile reach which is almost straight. Between Ashport Bar and Ashport Towhead there is a long bar
in the center of the river separating two pools which overlap by a distance of about three miles. The low-water slope of this crossing is relatively very steep (approximately one-foot per mile). A survey of this crossing on August 29, 1928 showed two crossings along the axes AB and CD respectively. So long as the stage did not fall below 13 feet on the Fulton, Tennessee, gage both crossings were navigable. However, by October 1 the Fulton gage fell to 10 feet and the lower channel (CD) filled up about 3 feet on the average. Experience showed that the low-water crossing channel tended to the approximate position of the axis AB and accordingly it was decided to dredge on that line. On October 2 the dredge Gamma began work and dredged for a total of 100 hours. A total of 177,445 cubic yards of material was moved. As the hourly capacity of the dredge was approximately 1,600 cubic yards per hour, it appears that the current aided the operation by scour in the dredged cut. It will be noted that the lower end of the cut was splayed outward to aid navigation. Although this crossing proved somewhat difficult for navigation, no further maintenance was needed. The survey of November-December, 1929 shows conditions about one year later. During the period intervening between October, 1928 and November, 1929 an unusually long period of high water occurred (flood of 1929) and the lower crossing channel had reopened.

Dredging operations at the foot of Island 35 (198 miles below Cairo) in 1928 are illustrated on Plate XLVI(A), (B) and (C). The channel through the reach of Island 35 has always been difficult for navigation. Attempts were made in 1925 to improve this reach by the construction of sand dams. These attempts were unsuccessful. They are discussed in Chapter IX. The present plan for the improvement of this reach by means of contractions is also described in Chapter IX. Before 1927 the main river channel lay to the west of Island 35 but after the flood of that year it was found that from 60 to 70 per cent of the river discharge was passing through the chute channel east of the island. Since the end of the 1927 low-water season, navigation has been forced to follow this chute channel. During the spring of 1928 experimental dredging was undertaken at the bar at the foot of the island. This dredging was begun by the dredge Gamma at about mid-stage, when the bar was considerably above mean low water, and continued intermittently from March until the middle of June. The survey of June 14 showed a channel which was below the low-water plane throughout practically its entire length and which was satisfactory for navigation at the existing river stages. It was hoped that this experimental dredging would serve to guide the current into the development of a low-water channel across the bar along the axis of the dredged cut thus avoiding a "square" turn in the channel. However, early in July a rise occurred which was followed by a sharp decline in stage. As a result of this stage fluctuation the dredged channel was obliterated as was shown by the survey of July 21-23, 1928. Simultaneously with this fall the channel showed a tendency to shift over toward the foot of Island 35. Between August 4 and October 17 the dredge Gamma made a series of short cuts at this point which were successful in opening a navigation channel.

In general, experience on the Lower Mississippi River has demonstrated that dredging can not be considered as a method of perma-
nent channel improvement. A program of navigation improvement based exclusively upon dredging means a constant expenditure of funds for dredging operations. This outlay becomes progressively greater as navigation increases. Dredging is properly an auxiliary to assist permanent training works in their mission of opening a channel. It is expected that the extension and development of the series of training works now under construction in the upper reaches of the lower river will render unnecessary the extensive dredging program which has heretofore been in effect. Until these training works are substantially advanced towards completion, however, the present dredging program must continue. The progress of the training works should nevertheless progressively decrease the load on the dredges. It is, however, doubtful whether the point will ever be reached when dredging will be entirely unnecessary on the lower river.
CHAPTER VIII.
BANK PROTECTION.

INTRODUCTION.

This chapter is confined to a discussion of bank protection methods in use on the Lower Mississippi River. It is not to be considered a treatise on bank protection as applied to all rivers. Underlying principles are discussed only insofar as is necessary to an understanding of the text. As here used, the term "bank protection" includes all works designed to defend the river banks against attack by the current. It does not include protection against attack from the landside. Thus, for example, bank protection works furnish no security against failures caused by interrupted drainage along the levee battures behind the bank. Bank protection works should, however, be so constructed that bank drainage is facilitated.

An erroneous but fairly widespread impression exists among the uninformed that bank protection need cover the upper bank only. Many faulty bank protection schemes spring from this delusion. This idea is probably based upon observation of wave action, an agency fundamentally different from the current action of an alluvial river. It will be recalled that the river current attacks the bank mainly by undercutting below the water surface, thus causing caving. Proposals for the protection of the upper bank by thick mats of vegetation or by flimsy, permeable structures designed to encourage silting are, therefore, valueless since they make no provision for adequate protection below the water surface.

A properly designed bank protection work fulfills two requirements: First, it affords protection against subaqueous scour, and second, the structure is of reasonable permanence. Works which do not satisfy these primary requirements are of little value in the general case.

At this point it is well to summarize briefly the discussions contained elsewhere in this volume on the subject of bank caving. Although the bank may occasionally cave in straight reaches, caving is found principally in bends. The rate of bank caving normally varies directly with the degree of the curvature of the bend. At low water, the point of maximum attack is usually at, or near, the head of the bend, but as the stage rises the point of attack moves downstream, and, at extreme high water, it is usually at, or near, the lower end of the bend. Low-water caving is normally uniform and fairly free from eddy action. High-water caving, on the other hand, is usually accompanied by pronounced eddy action. During high stages the water pressure holds much insecure bank material in place. When the stage falls, this pressure is removed; and large, irregular sections of bank frequently break off and fall into the river. High-water caving is therefore not uniform and usually results in the formation of pockets in the bank line. When the concave bank is held by protective works, the river tends to deepen its channel in lieu of cutting away the bank. This channel deepening may be relatively great. Bank protection works must be so planned that this deepening of the channel will not ultimately destroy their usefulness.
The name "bank protection" effectually describes its fundamental purpose. The immediate or direct purposes of bank protection may, however, differ widely. Some of the more important are as follows:

a. The prevention of cut-offs—Bank protection is usually placed at the necks of deep bends to prevent caving and eventual cut-offs.

b. Navigation improvement—Bank protection is often placed on the bank opposite contraction works to prevent the undue widening of the channel by bank erosion.

c. The improvement of river harbors.

d. Levee protection—When economically justified, bank protection is used to protect levees from bank caving. Bank protection is, however, normally much more expensive than a levee set-back. The use of bank protection to save levees is therefore restricted to special cases such as the safeguarding of levees which protect centers of wealth and population. (This is discussed in Chapter VI.)

In addition, all bank protection serves one useful purpose. Since it prevents caving, it reduces the volume of the silt load which the river must carry.

Bank protection works on the Lower Mississippi may be divided into two classes; "non-continuous works" and "revetment". Non-continuous works are those designed to deflect the erosive current away from the protected bank or to reduce its velocity to such a point that capacity to scour is lost. Revetment consists of a continuous covering which protects the bank from current attack. Many types of non-continuous protection works have been used on the lower river. These types include screens of wire or light timber; abattis built of poles and brush; permeable cribs or retards; submerged spur dikes; and sunken groins. All have proved inadequate and have been discarded. Up to the present time, revetment is the only satisfactory type of bank protection yet developed for use on the Lower Mississippi. In discussing non-continuous works, it is necessary to distinguish between bank protection works and the contraction works described in Chapter IX. Although contraction works protect the river bank and encourage silting, their primary purpose is channel alignment. Discussion of dikes in the present chapter is confined to those works intended primarily for protection of the banks against erosion.

Revetment consists of a flexible, continuous pavement laid along the bank below the water surface together with paving on the upper bank. Revetment extends from the top to the bottom of the bank. It should also extend for some distance beyond the toe of the bank slope.

As pertains to subaqueous protection, revetment may be divided into two general subclasses: the mattress constructed of lumber, willow, cane, or brush; and the concrete mat. Mattresses of the first class must
be sunk by ballasting. This class of revetment consists of the following types, all of which have been used on the Lower Mississippi:

a. Mattress made of fish pole cane and threaded on wire.
b. Mattress of brush woven together on poles or wire.
c. Board mattress.
d. Framed mattress of willows or brush secured between frames.
e. "Fascines" or mattresses made up of willow "fascines" sewed securely together.

Concrete mats have been constructed in the three following types:

a. Monolithic concrete slabs.
b. Articulated concrete mattress; consisting of reinforced concrete blocks cast in articulated units which are united into large mats by clamps and cables.
c. Slab mattress; consisting of large concrete slabs laid along the bank either butt to butt or overlapping at the edges.

Of the first subclass given above, the framed and fascine mattresses only proved sufficiently satisfactory to justify their continued use. They are now the two standard types of willow mattress on the Lower Mississippi. The cane, the woven, and the lumber mattresses all proved inadequate and have been discarded for general use. During 1931, however, the lumber mattress was given another experimental trial in the Memphis Engineer District. This will be discussed later in this chapter. Of the concrete mat types, the monolithic slab proved unsatisfactory and has been abandoned. The slab mat and the articulated concrete mat are at present standard types on the lower river.

Upper bank paving may be either riprap or concrete. The concrete paving may be a monolithic slab laid in the dry, an articulated concrete mat, or a slab mat. The concrete mats used for upper bank paving are the same as those used for subaqueous revetment, the only difference being in the methods of placement.

The construction of bank revetment involves four distinct operations. These operations, given in the order in which they occur, are: bank clearing; bank grading; the placement of the subaqueous revetment; and paving the upper bank. Bank clearing includes not only the bank itself, but also a strip behind the bank and beyond the top of the revetment. In this operation all trees, stumps, fences, etc., which are likely to impede the work or to cause snags in the river, are removed. The purpose of bank grading is obviously to prepare the bank for the placement of the upper bank paving. The extension of this grading operation below the water surface is, however, of great value in the proper placement of the subaqueous revetment.

NON-CONTINUOUS BANK PROTECTION.

The screen dike used on the Lower Mississippi is illustrated on Plate XLVII. It consisted of a permeable screen of timber or brush hung from buoys and anchored to the river bottom by anchor piles or
cribs. To prevent scour a sill mattress about 2½ feet thick and about 60 feet wide constructed of willows and weighted with rock was laid on the river bottom beneath the screen. A variation of this construction was the wire mesh screen hung from piles, the meshes being about 3 feet square. Screen dikes were built in 1881 for the protection of Memphis Harbor. They were also used in 1881-82 in an attempt to stop serious bank caving at Cowpen Neck immediately above Natchez. In operation, the screen silted up on the river bank above the low-water line almost immediately after construction and would not rise into position when the river stage subsequently rose. Below low water, the screen operated well until high water destroyed or carried away the buoy floats. Then the force of the current forced the screen to the bottom where it was covered up with silt and remained permanently on the bottom.

In 1881 wire mesh screens were used experimentally in Plum Point reach (168 miles below Cairo). These screens were hung so that their bottom edges were about at low water. It was anticipated that the wire mesh would mat up with drift to the point of practical impermeability. The experiment was, however, a failure. The piles which supported the screens washed out and the structure collapsed.

In practice screen dikes proved utterly inadequate as a method of bank protection. Beyond the instances described above, no attempt has been made to use them on the Lower Mississippi. For a discussion of screen dikes, the reader is referred to Occasional Papers No. 41, Engineer School, U. S. Army, 1910; and to the Annual Report of the Mississippi River Commission for 1882.

The abattis dike was a permeable structure constructed of poles and brush, and designed to present an inclined surface to the river current. The abattis is illustrated on Plate XLVIII. While it was designed to encourage siltation, and thus indirectly to serve as bank protection, its primary purpose was for contraction (closure of chute channels, etc.). It is therefore discussed in Chapter IX. Suffice it to say here, that as a bank protection work, the abattis proved practically valueless. Its use has been abandoned on the lower river.

The submerged spur dike, as used on the Lower Mississippi, normally consisted of successive layers of cribs, each 6 or more feet high, ballasted with rock and resting on a foundation mattress designed to prevent scour under or around the outer end of the structure. The top of the dike usually extended on a slope of 1 on 3 from the low-water line on the bank to the outer end of the structure. Spacing between successive dikes varied from a minimum of about 300 feet to a maximum of about 1,200 feet. The cribs were built of willows, brush, and timber tied together with iron tie rods. A description of this type of dike is contained in the pamphlet "Bank Protection, Mississippi River" (Government Printing Office, 1922). Crib spur dikes were used on the lower river between 1884 and 1907. They were built in the harbors of New Orleans, Greenville, Helena, Memphis, and Columbus. They were also used in Giles Bend above Natchez harbor. Cribs and other forms of spur dikes have been constructed to break up large destructive eddies. Where failures occur in the revetment covering a bank, small but violent eddies are set up. These eddies cut pockets in the bank line which frequently enlarge rapidly. They have occasionally become so
destructive that it has been necessary to construct dikes across them to break up their action. The use of submerged cribs for this purpose has been abandoned in favor of other and more successful types of permeable pile dikes.

Submerged spur dikes proved a failure as a means of bank protection on the Lower Mississippi. In spite of the protection afforded by their foundation mats, they were frequently subjected to heavy attack from serious eddies which developed at their outer ends. In addition, they were often flanked out by the high-water current at their shore ends and thus became detached from the bank line. The spacing between dikes was usually found to be too great and scour occurred along the bank line between them thus necessitating additional protection by intermediate cribs or by revetment. The use of this type of bank protection has been abandoned.

The permeable crib dike or retard resembled the abattis in that it was primarily intended as a contraction work. It is therefore discussed in Chapter IX. So far as bank protection is concerned, however, the retard has proven unsatisfactory. It is not now being used for that purpose.

The groin has long been used for the improvement of rivers and several types have been patented. In 1930 experiments were undertaken in the Memphis Engineer District to ascertain whether a type of groin could be developed which would be effective as a bank protection work. If these experiments proved successful, it was hoped that a relatively inexpensive type of groin might be substituted at favorable locations for the expensive revetment now standard on the lower river. In these experiments, the groins were built normal to the river bank line with their tops flush with the bank surface. They extended from the upper bank to the thalweg. Spacing between groins varied from 50 to 300 feet. Two types were constructed, a riprap groin and a line of interlocking sheet piling. The riprap groin was placed in a trench 5 to 6 feet deep. The piling was driven to a depth of about 14 feet. These experiments gave no grounds for hope that sunken groins would prove useful for bank protection. Where the bank was subjected to heavy caving, great construction difficulties were encountered. In slowly caving bends, construction was not difficult but the structures offered little or no protection to the bank line. Groins must be considered valueless for bank protection on the lower river.

THE CANE MATTRESS.

Although the early attempts to develop bank protection methods were confined largely to non-continuous types (cribs, etc.), efforts were also made to construct a satisfactory revetment. The first real effort to use revetment on the Lower Mississippi was made in New Orleans harbor in 1878. Curiously enough, the condition which these mats were intended to rectify was caused primarily by deposits rather than scour. In the bend below the foot of Canal Street, the piling under the wharves was causing heavy deposits of sediment during high water. These deposits frequently slid out into deeper water during succeeding low stages thereby seriously injuring the wharves. It was thought that
these masses of mud would be stabilized if the lower portion of the river bank were revetted against scour. Accordingly, cane mattresses were sunk. These cane mattresses were constructed of "fish-pole cane" woven on strands of galvanized wire into mats from 1 to 2 inches thick, 200 feet long, and 24 feet wide. Each mat was laid on the deck of a barge and was ballasted with sand bags or worn out boiler tubes. One edge of the mat was anchored to a line of piles, and sinking was accomplished by pulling the barge out from under the anchored mat. By June, 1879, some 18,900 square yards of bank had been covered by these mats. Contracts were let for the continuation of this work but progress was unsatisfactory. The mats did not prevent the mud slides and the injury to the wharves continued. The work was abandoned in 1882. The cane mattress had proved too weak for effective protection. It was not again used. For a discussion of these mats, the reader is referred to the Annual Report of the Chief of Engineers for 1879.

THE WOVEN MATTRESS.

The so-called woven mattress was much stronger than the cane mattress described above. The woven mattress was first developed on the Upper Mississippi in the St. Louis Engineer District. In 1882 its use was attempted on the lower river. Woven mattresses were placed in the Cairo-Memphis reach, notably at Ashport and Fletchers Bend in Plum Point reach; and at Hopefield Bend above Memphis. They were also sunk in Memphis Harbor, at Mayersville Island (10 miles above Lake Providence), and at Delta Point (opposite Vicksburg). The woven mattress is described in detail in the pamphlet "Bank Protection, Mississippi River" (Government Printing Office, 1922). The mattress was constructed on barges or floating on the water; it was then ballasted with rock and sunk in place. An early type consisted of a base of No. 8 galvanized wire net with meshes 3 feet square. On this net was placed a layer of brush laid with stems normal to the river current. The brush was fastened to the wire net by poles 5 feet apart, laid over and across the brush. The butt of each pole was thrust under a strand of the net and its top was secured to the net by a wire lashing. This method of construction had one serious defect. The poles were fastened at the ends only, and consequently bowed upward. Much of the brush was therefore insecurely fastened and was displaced while sinking. A later type of woven mat consisted of brush woven over and under poles in basket work fashion. Woven mats possessed little strength and attempts were made to reinforce them heavily with wire and tie rods. This served little purpose, however, since the mattress itself was too flimsy and weak to develop the full strength of the reinforcement.

The woven mat was lacking in strength, flexibility, and compactness. Its permeability was so great that appreciable scour occurred through it. This type was, after thorough trial, abandoned. It has been superseded by the frame and fascine type.

THE BOARD MATTRESS.

The board mattress is used extensively on the Upper Mississippi and on the Missouri. It has also had a limited use on the lower river
THE FRAMED MATTRESS
Building the mattress, Nine Mile Point (above New Orleans)
September 19, 1929
in the form of foot mats for permeable pile dikes (contraction works). As normally constructed, the board mattress used as a footing for a pile dike, consists of 1-inch by 4-inch boards woven together. It is usually built on barges, launched, and moored directly over the area which it is to protect. The shore end is then loaded with stone ballast and sunk. Sinking is then continued from the shore end progressively to the outer end of the mattress. Stone for sinking is placed from barges.

In 1900 an attempt was made to use a board mattress for revetment at Bondurant Chute (50 miles above Natchez). This mattress was not woven. It consisted of 1-inch by 12-inch boards laid closely together and secured by cross stringers of 2-inch by 4-inch timber laid at intervals of 4 feet and extending entirely across the mattress. Strips of 2-inch by 4-inch timber were laid at 10-foot intervals at right angles to the stringers so as to form pockets to prevent the ballast from shifting. This mattress was built on shore and was towed into position for sinking. In slack water, no great difficulty was experienced in the sinking operation. In a swift current, however, the mattress became unmanageable and efforts to sink it were entirely unsuccessful. This type of board mattress has not since been used on the Lower Mississippi. For a description of this mattress, the reader is referred to the Annual Report of the Mississippi River Commission for 1901, or to the pamphlet "Bank Protection, Mississippi River".

In 1931 two sections of woven board mattress were sunk for bank protection at Keyes Point (in Plum Point reach, about 160 miles below Cairo). This mattress consisted of 1-inch by 4-inch rough lumber woven together. Closing boards were placed so as to cover the interstices in the weaving. Board cribbing was constructed on the borders of the mattress to provide additional strength and to hold the ballast in place. The whole was bound together with wire. This mattress has not yet (December 31, 1931) been in place long enough to permit an opinion to be voiced as to its usefulness.

THE FRAMED MATTRESS.

Framed mattresses were first used in Memphis Harbor between 1878 and 1882. Descriptions of these early mattresses are found in the Annual Reports of the Chief of Engineers for 1879 and 1888. The first mattresses were small, light affairs, but experience has indicated improvements in design and construction methods which have resulted in the present standard type (see Plate XLIX). For a detailed description of this mattress, the reader is referred to the pamphlet "Bank Protection, Mississippi River" (Government Printing Office, 1922).

The framed mattress is essentially a three-ply willow mat, 16 inches thick, made up in rectangular units or sections, each 150 feet long by 100 feet wide. When sunk in place, these units lie with their longer dimensions parallel to the bank line. The mattress unit is built on sloping ways inclined toward the water surface to facilitate launching. Normally these ways are located on the river banks or on islands convenient to the willow supply. Occasionally, however, they are placed on barges. The first operation in the fabrication of the mattress is the construction of the mattress frames. Each frame extends entirely across
the width of the mattress (100 feet) and consists of an upper and lower chord which are connected by vertical stanchions. Each chord consists of two parallel members made up of timbers spliced end to end so as to break joints. The vertical stanchions are 32 inches long and fit between the members of the upper and lower chords. The stanchions are spaced 5 feet apart except at the ends of the frame where the spacing is reduced to 3 feet, 6 inches. In construction the stanchions are fastened to the lower chord. The upper chord or cap is fitted down over the stanchions after the willows have been placed. Each frame is strengthened by cables spiked to it.

When the frames are ready, the lower chords are placed on the ways. These chords are parallel to each other and spaced 10 feet apart except at the ends of the mattress where the spacing is reduced to 8 feet. Upon and across these lower chords is laid a single layer of willow poles from 3 to 6 inches through the butts and arranged so as to breaks joints. These willows are nailed to the lower chords of the frames. Upon this layer and at right angles to it (i.e., parallel to the frame) is placed a layer of willow brush, of such thickness that, when completed, the entire mattress will be 16 inches thick. Over this is placed a third layer of willows parallel to and in all respects similar to the first layer. The caps of the frames are now fitted down over the stanchions and the mattress is compressed by jacks specially designed for the purpose. The upper chord is then nailed and tree-nailed to the stanchions. The mattress is then launched and, when afloat, is stiffened by longitudinal strengthening poles placed on top of the frames and perpendicular to them. These poles and the upper chords of the frames form together wide, shallow pockets for the sinking rock.

After construction the sections are made up into tows (sometimes as large as ten or more sections) and are towed to the site where they are to be sunk. Before sinking, the sections are made up into large mattresses by splicing them together side by side. Units are spliced together by connecting the upper chords of abutting frames by means of double fish plates of timber 11 feet long. The strengthening cables along the frames are also joined by clips. Each strip of mattress is continuous throughout the entire width of the revetment (i.e., dimension athwart the current). Mattress strips are sunk with downstream overlaps of 10 to 15 feet between strips. Usually the mattress sections are spliced together in strips on the site where they are to be sunk, but sometimes under favorable conditions the strips are made up elsewhere and then towed into place. This method is useful where special conditions require the sinking operation to be carried out rapidly.

The sinking operation is illustrated on Plate L. Several hundred feet upstream of the point of sinking, a mooring barge is anchored broadside to the current. To this barge a line of "sinking" or "string-out" barges are moored by long cables. These sinking barges lie end to end athwart the current and serve as a working platform for handling and lowering the mattress. The mattress, which is to be sunk, is moored by "sinking" cables to the downstream side of the line of "string-out" barges. A series of "toggle" cables are also run from the mooring barges to the upstream edge of the mattress. These cables pass beneath the "string-out" barges and are secured by pins to the
Ballasting the mattress
THE FASCINE MATTRESS

Constructing Fascine Mattress at Sunflower Landing
47 miles below Helena, Ark., December, 1927
mattress in such a manner that they can be released after the mattress has been sunk. These toggle cables serve to hold the mattress in place during the sinking operation. The shore end of the mattress is anchored by cables run to deadmen buried in the upper bank.

The mattress is now ready for sinking. One stone barge is placed at the outer end of the mattress and a second one at the shore end of the downstream side. Runways are laid over the mattress and the entire strip is loaded to the point of sinking. At the upstream edge of the mattress, this loading is continued until that edge is in a sinking condition, being held up only by the lines to the "string-out" barges. These lines are then released and the stone (or "cast-off") barge loads the outer end until it also sinks. The cast-off barge is then moved inward toward the shore. As it moves, the mattress is sunk by ballast cast off from the barge. When the mattress is fully sunk, the toggle cables are released and the sinking plant moves upstream into position for the next strip of mattress. Strips vary in length (athwart the current) from about 300 to 600 feet, according to the depth of the channel. In sinking a mattress, the shore end should be placed as low down the bank as possible. Willows rot rapidly when exposed to alternate wetting and drying. The mattress therefore deteriorates rapidly at the water line. To prevent this rapid decay, the upper bank paving should be carried as far down the bank as possible. The latest revetment specifications (1931) are designed to minimize this rotting at the water line. They provide that no willow mat shall be laid more than 2 feet above mean low water. Mats used above this point must be made of concrete.

THE FASCINE MATTRESS.

The fascine mattress (Plate LI) was evolved from attempts to improve the weak and permeable woven mattress which has been described earlier in this chapter. The fascine type was designed to insure strength and compactness without too great a sacrifice of flexibility. This mattress was first used at Daniels Point, Arkansas (154 miles below Cairo) in 1893. It was immediately thereafter adopted as standard for the river between Cairo and Vicksburg. In 1925 the zone of its use was extended to the mouth of Red River. It is now the standard type of willow mattress for use between Cairo and the mouth of Red River.

For a detailed description of the fascine mattress, the reader is referred to the pamphlet "Bank Protection, Mississippi River". This mattress consists essentially of a series of cylindrical bundles of willows (or "fascines") bound tightly side by side into a continuous bank covering. The name "fascine mattress" is no doubt derived from the similarity of these cylindrical bundles to the brush "fascines" at one time widely used in the art of military fortification. In revetment construction, the fascines are continuous throughout the entire width of the mattress (i.e., dimension athwart the current). In the Daniels Point work, the fascines were about 12 inches in diameter and were from 50 to 100 feet long. Each fascine was tightly compressed and was wired together every 3 feet throughout its length. Fascines were bound into the mattress by heavy longitudinal cables and sewing wires extending throughout the length of the mattress (i.e., parallel to the current).
The present mattress is of somewhat heavier construction. The fascines are 16 inches in diameter and from 250 to 300 feet long, depending upon the bank and the depth of the river. The length of the mattress usually varies from about 1,000 feet to about 1,200 feet, but may be longer or shorter depending upon the exigencies of the work. Occasionally, under favorable conditions, mattresses as long as 3,000 feet have been constructed and sunk. The original practice of binding the fascines together before placing them in the mattress has been discontinued. At present they are compressed into bundles by the same operation which weaves them into the mattress.

As has been previously stated, the fascine mattress must be built over the bank which it is designed to protect. The plant required for construction consists of fascine barges; mattress or sinking barges; a crane barge (with small locomotive crane); stone barges; willow or brush barges; and the necessary towboats. The first operation is the construction of the "header". This is a large fascine from $2\frac{1}{2}$ to 3 feet in diameter made up of hardwood or large willow poles. To the header are attached the longitudinal cables which bind the mattress together. The header is held in position by "lowering" lines secured to the mattress barges which are lashed end to end and moored across the current (see Plate LI). If necessary, a mooring barge is anchored upstream of the mattress barges. The header is frequently constructed while the mattress barges are moored along the bank. When it is completed, the barges are swung out into position across the current. The header being in place, the fascine barges are brought into position below it and parallel to the mattress barges. These fascine barges carry ways spaced 8 feet apart and sloping upstream to the water surface. At the top of each way is a Y-shaped yoke which receives the willows prior to their placement in the mattress. The yokes are so designed that they may be tripped simultaneously thus allowing the willows to slide into place on the ways. Behind the line of yokes is a working platform running the length of the fascine barges. Alongside the fascine barges and downstream of them is moored the crane barge. Below the crane barge are moored the willow barges which carry the supply of willows. The locomotive crane begins operations by placing a supply of willows on the working platforms of the fascine barges. These are then placed in the yokes by laborers known as "willow chasers", and are inspected to insure that there is a sufficient supply, uniformly distributed. The yokes are then tripped and the willows slide down the ways into position. They are then bound into the mattress in the following manner:

Bottom cables from $\frac{5}{8}$ to $\frac{3}{8}$ inches in diameter, and spaced 8 feet apart, extend from the header throughout the length of the mattress. The fascine ways (on the fascine barges) are spaced so that they feed directly on to these cables. Each successive fascine is bound to the bottom by $\frac{3}{4}$-inch sewing strands. Each sewing stand is attached to the header and is continuous throughout the mattress. The sewing strand is carried completely around each fascine, including the bottom cable in the loop thus formed. The sewing cable is clipped to the bottom cable at 25-foot intervals. The sewing strands are tightened by hand-operated sewing ratchets which are placed at the foot of each fascine way. The fascine is compressed and sewed into the mattress by a single opera-
Constructing Fascine Mattress at Sunflower Landing
47 miles below Helena, Ark., December, 1927
THE FASCINE MATTRESS
Memphis Front—General view of mattress
October 2, 1930.
THE FASCINE MATTRESS
Sinking operation, Sunflower Landing, 47 miles below
Helena, Ark., December, 1927
tion. As successive fascines are made, the mattress is extended up the fascine ways. The ways are cleared by slacking off the fascine barges which slide out from under the mat, thus launching it.

About ten fascines downstream from the header, a heavy fascine is sometimes placed in the mattress. This second header is almost as heavy as the first header. It serves as an anchor for the double straps by which the sinking cables are attached to the mattress and thus relieves the first header of part of the strain placed upon it during construction. The last fascine in the mattress, known as the tail, is also oversize and serves as an anchor for the longitudinal cables.

As the fabrication of the mattress progresses, longitudinal surface cables are laid over it for additional strength and a grillage of poles is built upon it to hold the ballast stone. Anchor cables are also run from the shore side of the mattress to deadmen in the bank. The spacing between these shore cables varies from 75 to 200 feet depending upon circumstances. When the tail fascine has been placed, the mattress is complete and ready for sinking. The fascine barges with attendant plant are then removed.

The fascine mattress is sunk in a manner very similar to the sinking operation for the framed mattress. The whole mattress is first evenly ballasted to the point of sinking by stone placed from wheelbarrows. Frequently ballasting is begun before the mattress is completed but no ballasting should be done more than four days before sinking. The floating mattress quickly becomes waterlogged and is sometimes heavily loaded with sediment. Under such conditions, a heavily ballasted mattress is likely to sink prematurely. When the entire mattress has been ballasted, additional stone is placed at the head of the mat and the lowering lines slacked off until the head of the mattress is submerged to a depth of 10 feet. Stone barges are then placed over the head of the mattress and more ballast is thrown on. The lowering lines are again slacked off and the head of the mattress is allowed to sink slowly. Meanwhile the stone barges move slowly downstream over the mattress discharging ballast as they go. By the time they reach the foot of the mattress the entire structure is sunk. Additional ballast is then spread on evenly by dumping from barges.

The fascine mattress is difficult to sink in a deep swift current, especially where there is much drift. Fascine mattress plant must frequently stay in place for as much as fifteen days. The accumulation of drift during this period may place a heavy strain upon the floating plant and may even necessitate the premature sinking of uncompleted revetment. Except when such conditions are encountered, however, the fascine mattress is constructed with comparative ease.

The restrictions now placed upon laying willow mattress above the water line have been briefly described earlier in this chapter under the heading "The Framed Mattress".

THE MONOLITHIC CONCRETE SLAB REVETMENT.

The first attempt to substitute concrete for willow as revetment material was experimentally undertaken in the Memphis Engineer District in 1914. The object of the Memphis experiments was the de-
velopment of a subaqueous concrete mat which could be cast as a monolith and then sunk in place. The slab adopted for experiment was 4 inches thick and was reinforced with wire mesh and %/inch cables. It was believed that such a slab, when placed on the river bottom, would crack sufficiently to conform to the river bottom and would yet be held together by its reinforcement so as to form a mosaic-like pavement on the river bed. The first slabs constructed gave some promise of effective results and experimental plant was built for quantity production. This plant consisted essentially of a casting barge and a sinking barge. The casting barge was provided with sloping ways upon which the slab was cast. The sinking barge was equipped with a series of Pratt trusses for handling the completed mat. Sinking was accomplished by moving the sinking barge alongside the casting barge so that the trusses spanned the latter. Each truss carried a set of lifting cables equipped with special releasing hooks which engaged in loops embedded in the concrete mat. When the hooks were so engaged, the casting barge was slid out sidewise from under the mat, the outer ends of the Pratt trusses being supported by rollers running on tracks laid across the casting barge. When the casting barge was completely withdrawn, the slab was suspended between two barges. It was then lowered away and sunk in place. The largest slabs sunk by this method were 50 feet wide by 125 feet long. These monolithic mats proved ineffective. They could not be accurately placed in position and afforded no real protection to the river bank. The plant was later modified to permit the use of precast slabs, 240 feet long, 12 feet wide, and 3/4 inches thick, and heavily reinforced both transversely and longitudinally with \%2-inch by 2-inch steel bars and with wire mesh. These long, comparatively narrow slabs were joined together before sinking. The general scheme of placement of this modified mat was very similar to that of the larger slabs previously tried. This modified slab was used at Cache River, Illinois, in 1922-1923, and again at Avenue, Arkansas (about 40 miles below Helena) in 1923-1924. Because of the difficulty in placement and its great cost, the use of this slab mattress was almost immediately abandoned.

The monolithic slab, and its modified form, never passed beyond the experimental stage. They proved utterly impracticable and were abandoned. This revetment is, however, noteworthy in that it constituted the first attempt to use concrete for revetment on the lower river. Notwithstanding its defects, the concept of the monolithic slab was based upon a sound appreciation of the underlying principles which govern the construction of revetment. The monolithic slab was developed in an effort to secure a bank covering flexible enough to conform intimately to the irregularities of the bank but yet strong and impermeable enough to protect the bank from the attack of the current.

ARTICULATED CONCRETE MAT REVETMENT.

The articulated concrete mat revetment originated with experiments made in the Vicksburg Engineer District in 1915. Until 1928 the use of this mat was limited to the Vicksburg District, but in that year it was introduced into the Second New Orleans District where cer-
ARTICULATED CONCRETE MAT
Casting Plant at Nine Mile Point (9 miles above New Orleans)
June 5, 1931
tain minor modifications in design were made. There were thus developed two types known respectively as the Vicksburg and New Orleans mat. In 1931 the articulated mat was introduced into the Memphis Engineer District where it was extensively used. In that year, after considerable study, an improved design was developed which has superseded the Vicksburg and New Orleans types and is now considered standard.

The present articulated concrete mat is illustrated on Plate LII. It is composed of units each 25 feet long and 4 feet wide (including space allowance for connections with adjoining units). Each unit is made up of twenty reinforced concrete blocks each 3 feet, 10½ inches long, 14 inches wide, and 3 inches thick, spaced 1-inch apart on heavy reinforcing fabric. It will be noted from inspection of Plate LII that this reinforcement consists of three longitudinal wires which are continuous throughout the length of the unit and which are looped at each end for fastening to corresponding longitudinal wires in the abutting ends of adjoining units. The transverse reinforcement is provided by rectangular brackets, the ends of which extend beyond the concrete blocks to permit joining with brackets in adjacent units in the mat. Blocks are recessed as shown on Plate LII to permit end and side fastenings with adjacent units to be made by clips. It will be noted that cables are run between adjacent mat units and that the side fastenings not only pass around the ends of the reinforcing brackets but also around the cables. These cables are known as launching cables and extend throughout the width of the mat (i.e., direction athwart the current). These cables are used in the sinking operation. Their use will be explained later in this chapter.

The mat units are constructed at a floating casting plant, the main unit of which is a concrete mixing barge equipped with a bucket mounted on a conveyor long enough to extend across the entire width of the barges on which the mat units were cast. The mixing barge is accompanied by the necessary sand, gravel, and cement storage barges. Sand and gravel are obtained by floating plant from river bars. Quick setting cement is used. The units are cast in forms slotted to hold the reinforcing mesh in place while the slabs are being poured.

In operation, eight to sixteen casting barges are moored end to end and the forms are laid across their decks with narrow aisles between forms. The mixing plant moves along the line of barges, pouring concrete as it progresses. After the concrete has acquired an initial set, the forms are loosened and as soon thereafter as safe they are lifted off the green concrete slabs. Kraft paper, 50 inches wide, is then laid over each mat unit and a form reset on the Kraft paper in readiness for the second pouring. The second slab can be poured in about 6 hours in summer and in from 12 to 24 hours in winter. Successive pourings result in the gradual even loading of each barge with stacks of green mat units. As barges are loaded, they are cut out of line and moored at a convenient point until the slabs are properly seasoned and ready for sinking. In pouring slabs, barge loadings are staggered to prevent all barges in the line from being completely loaded simultaneously. The work is planned for continuous uniform operation of both casting and sinking plant.
The sinking operation is illustrated on Plate LII. The principal unit of the sinking plant is the so-called "sinking" barge itself. This barge is equipped with a superdeck which slopes toward one gunwale and terminates in a smooth curve over that gunwale to the water line. Across this superdeck are double trains of rollers, each train being so spaced as to support a mat unit when laid upon it with the length of the unit across the barge. The mat units are placed upon these rollers joined together by clips as described above and then launched in the manner described below. Winches and friction pulleys for the operation of the launching cables are provided on the main deck below the upper edge of the superdeck. The sinking barge is of sufficient size to permit the assembly of the mat in sections 25 feet long (dimension normal to the bank line) and from 100 to 140 feet wide (dimension parallel to the bank line). It is provided with small cranes to permit the transfer of mat units from casting barges to the sinking barge itself. When handled by these cranes, the mat units are supported by frames to prevent sagging or buckling. The sinking barge is also provided with retractable cantilever arms or "fingers" to permit the mat to be laid at the water line in water too shallow to float the barge itself.

The operation of sinking is progressive from the bank line to the outer edge of the revetment. The sinking barge is placed downstream of a line of "string-out" barges as shown on Plate LIII and a section of mat is assembled upon it. Anchorage to the upper bank is provided by non-corrosive anchor wires which are attached to the looped ends of the longitudinal reinforcing wires which protrude from the edge of the mat. The three anchor wires leading from each mat unit are seized together to form a single cable at a convenient distance from the mat. These cables are anchored to deadmen buried in the top of the bank. Anchor cables are ultimately covered over by the upper bank paving.

When the first section of mat is assembled, it is run out on the retractable "fingers" and launched with its shore edge above the water line on the bank. If this is impracticable, the shore edge of the mat is placed as near as possible to the water line and the gap between it and the upper bank pavement is later covered by a small "connecting" mat placed by a derrick boat. In launching the first section of mat, care is taken that, after launching, its outer edge still remains on the sinking barge to permit the second section of mat to be clipped to it. The superdeck being clear of the first section of mat, the second section is assembled upon it and is launched. Successive sections are then launched as the sinking barge is moved progressively out toward the outer edge of the area to be covered by revetment. The retractable "fingers" are used only in shallow water. They are not needed in water deep enough to float the sinking barge. As each section of mat is launched, the launching cables are paid out only far enough to permit the next section to clear the superdeck sufficiently to permit the assembly of the next section. The loops in the ends of the longitudinal reinforcing wires of each unit are clipped to the corresponding loops in the abutting unit in the preceding section, thus making the mat strip continuous throughout. The function of the launching cables is now apparent. They are provided to furnish the mat sufficient strength for the launching operation and to control the movement of the mat from the superdeck of the sinking barge until it comes to rest on the river
ARTICULATED CONCRETE MAT, SINKING BARGE
Giles Bend (above Natchez, Mississippi)
August 28, 1930
ARTICULATED CONCRETE MAT
Deer Park Bend (24 miles below Natchez)
September 25, 1931
bed. It will be noted that these launching cables are not intended to strengthen the mat when in place. Their ultimate breakage through erosion or corrosion does not, therefore weaken the mat. The reinforcing fabric is clipped together to form a continuous net throughout each strip of revetment. The fabric and the clips are now fabricated from metal highly resistant to corrosion.

The articulated concrete revetment is thus laid in continuous strips from 100 to 140 feet wide extending from the bank line to the outer edge of the revetment. When such a strip has been laid, the sinking plant is moved upstream and a second strip is laid with a 5-foot downstream overlap over the first strip.

CONCRETE SLAB REVETMENT.

In 1924 the development of a lapped slab concrete revetment was undertaken in the Memphis Engineer District. By 1927 this development had reached the point of standardization and quantity production. The first experimental revetment of this type was laid in 1926 at Cow Island (21 miles below Memphis) and at Knowlton (59 miles below Helena). By 1927 the sinking plant had been developed and constructed. In the working season of that year, 4,300 feet of slab revetment was laid in the "Hickman to Huffman" Bend (about 130 miles below Cairo), and 1,400 feet at the Bend of Island No. 8 (43 miles below Cairo). Since 1927 lapped slab mat has been extensively used in the Memphis District. During the past working season (1931) a change was introduced in the method of laying the mat. Instead of lapping, the slabs were laid butt to butt, thus effecting a very considerable increase in the effective area of each slab. The butt slab is now standard for this class of concrete revetment.

The slab mat is illustrated on Plate LIV. It will be noted that the changes introduced in design by the introduction of the butt slab method of placement were not great. The plant developed for the construction and placement of the lapped slab will also serve for the butt slab. The butt slab is 9 feet, 11 inches long, 6 feet wide, and 3 inches thick. It is bevelled at the corners to permit the necessary fastenings to be made to launching cables and is provided with two handling loops. The slab is heavily reinforced. The assembly loops and handling loops which are exposed to corrosion and erosion are fabricated of non-corrosive metal.

The method of casting the slab is essentially the same as that employed for the articulated concrete mat and will not be described here.

The sinking barge is illustrated on Plate LV. It is a steel hull from about 120 feet to about 260 feet long completely decked and equipped with a vertical framed scaffold along one gunwale from which slabs are suspended and sunk. Inboard from this scaffold are lines of drums and winches for the manipulation of the cables necessary for the sinking operation. Small locomotive cranes are provided to handle the slabs.

The operation of sinking the mat is illustrated on Plate LVI. A line of mooring or "string-out" barges is placed across the current as shown on the plate and the sinking barge is moored below them and
CONCRETE SLAB MAT
(Lapped Type)

Slabs laid in dry by derrick boat for connection between subaqueous mattress and upper bank paving.
New Madrid, September 4, 1930
broadside to the bank. In practice, two or three sinking barges are usually moored end to end below the mooring barges. The width of the mat strip laid at one operation may be as great as about 500 feet (parallel to the bank line). The slab mat is laid progressively from the outer edge of the mat strip to the bank line. In this it differs from the articulated mat which is laid from the bank line outward.

The first operation of sinking consists in the placement of a line of heavy concrete header blocks or sills along the outer edge of the proposed revetment. To these blocks are attached the guide cables which run in pairs from these blocks over sheaves on the top member of the sinking barge scaffolding and thence to the cable drums on the barge deck. These pairs of guide cables are spaced to permit the assembly of the mat (see Plate LIV). The header blocks are sunk in much the same way as is described below for the slabs themselves.

The header blocks being in place, slab sinking now begins. The crane lifts each slab by the two handling loops and swings it into position so that it hangs vertically against the vertical scaffold with its longer sides horizontal. The slab is now clipped at its four corner loops to guide cables and a sinking cable is attached to its handling loops (see Plate LV). This sinking cable is operated by a winch and lowers away the slab to the river bottom. The slab is guided into place by the guide cables to which it is attached. The sinking cable couplings are designed so that they disengage when the slab is in place. The slabs are thus sunk butt to butt in successive rows. As the work progresses the sinking barges are moved toward the bank line. Where the water is too shallow for the sinking barge to operate, slabs are placed by a derrick boat. It is thus apparent that the slab mat is laid in wide strips extending from the outer edge of the revetment into the shore line. Adjacent strips overlap about 5 feet.

The slab mat is designed to be held in place by the weight of the individual slabs themselves.

**UPPER BANK PROTECTION.**

On the Lower Mississippi River, upper bank protection is an essential part of revetment and must be provided as soon as the subaqueous mats have been sunk. Otherwise the strong attack of the current during subsequent high waters will cause caving above the mat which will soon extend behind it and flank it out. An instance of revetment failure due to lack of upper bank protection occurred at Hopefield Bend (at Memphis) in 1883. The end of that working season found about 4,250 feet of bank in this bend protected by low water but bare above. This condition was not due to design but was the result of failure of the upper bank paving parties to keep up with the mattress sinking operations. The situation was not considered critical at the time since it was believed that caving above the mattress would not be extensive enough during one high water to endanger the work. It was planned to place the upper bank protection during the next low-water season. A single high water, however, totally undermined the mat and caved the bank behind it for distances as great in some places as 500 feet.
CONCRETE SLAB MAT
(Lapped Type)

Casting Slabs
New Madrid Bend
August 28, 1930

Slabs laid in dry by derrick boat for connection between subaqueous mattress and upper bank paving.
New Madrid, September 4, 1930
UPPER BANK GRADING BY HYDRAULIC METHOD
Hopefield Point opposite Memphis, 1910
The first step in the preparation of the upper bank for protection is bank clearing. This includes the removal of trees, logs, stumps, and brush both from the bank itself and from a zone on top of the bank which is normally 25 feet wide but may in special cases be as much as 100 feet wide. Clearing should be also extended as far as possible below the water surface. Submerged snags are removed by derrick boats or are blown out with explosives. Snag boats are sometimes employed. The hidden snag is a great danger to the subaqueous mattress. A mattress sunk over such a snag is subjected to heavy strains which ultimately cause its rupture over an extensive area. Perhaps, however, the most effective treatment of snags is the prevention of their occurrence rather than their removal. The snags found along a caving bank are not normally those which drift down with the current from above. They are usually formed by standing timber which falls into the river as the bank caves. It is probable that the removal of timber on caving banks a season or two before active revetting operations are undertaken, will materially decrease the number of snags encountered when the revetment is actually placed. Recent studies support this theory and indicate the advisability of a progressive revetment policy which is planned far enough ahead to permit this early timber removal.

The routine by which bank grading precedes the sinking of the subaqueous mattress is of recent development. In fact, the 1931 working season was the first one in which standing instructions rigidly required this sequence. Before that year, many construction outfits habitually sank the mattress before grading the upper bank. This procedure resulted from the relative speeds of the two operations. Until recently improved, bank grading was a much slower operation than mattress sinking. Thus, if the grading were done first, it either delayed the sinking operation or else preceded it so far that much graded bank caved badly before it could be protected by a mattress. The development of more rapid grading methods has relieved this situation, however, and has made the present routine possible.

Present practice prescribes within narrow limits the stage range within which subaqueous revetment can be sunk. Whenever the river rises to a stage exceeding 15 feet above mean low water, all subaqueous grading and mattress operations cease, not to be resumed until the river has again fallen to this stage. Concrete mats laid between this stage and the 6-foot stage (above mean low water) are not considered as subaqueous revetment but as upper bank paving. This fixes the upper limit of concrete subaqueous revetment at 6 feet above mean low water. No willow mattress can be laid above a 2-foot stage above mean low water. Bank grading must be extended to 5 feet below the mean low-water surface. Under extreme conditions (at a stage 15 feet above mean low water), it is necessary to grade the bank to a maximum distance of 20 feet below the water surface. The wisdom of this requirement is apparent. A caving bank is usually bluff or nearly so, and therefore affords a poor support for a mattress. In fact, mattresses placed on ungraded banks have been found, at subsequent low waters, to be vertical or nearly so at their bank ends. Such a mat is difficult to anchor to the bank. Moreover, the joint between the mat and the upper bank paving constitutes a dangerous point of weakness.
A reasonable amount of subaqueous grading makes for economic and permanent construction, and increases materially the effectiveness of the revetment. During the early revetment work under the Mississippi River Commission, bank grading was to a slope of 1 on from $2\frac{1}{2}$ to 3. Later the slope was changed to 1 on 3 to 4. Present specifications call for a grade of 1 on 3.

Bank grading was first done by hand or by small equipment such as slip scrapers. During the fall of 1879, however, the Engineer Department undertook bank grading by hydraulic methods on the Missouri River at Omaha, Nebraska. A description of these early operations is found in the Annual Report of the Chief of Engineers for 1880. The success of the Missouri River operations resulted in the introduction of hydraulic methods on the Lower Mississippi. The Annual Report of the Mississippi River Commission for 1882 contains a description of hydraulic bank grading operations at Delta Point (on the west bank of the river opposite Vicksburg). Further and more detailed descriptions of hydraulic methods are found in the pamphlet "Bank Protection, Mississippi River" and in Occasional Papers No. 41, Engineer School, U. S. Army.

The hydraulic grading plant normally consisted of two centrifugal turbine driven pumps mounted on a barge each of which was connected by pipe and hose lines to two nozzles having orifices of from $\frac{3}{4}$-inch to $1\frac{1}{4}$ inches diameter, depending upon the size of the stream required. Usually one line equipped with a small nozzle was used to undercut the bank while the other line with a large nozzle was used to sluice away the undercut material. The nozzles were mounted on swivels and directed by long handles. In operation, the barge was moored alongside the bank to be graded and the lines run ashore into position for cutting the upper bank away. An initial guide cut was made to grade in the bank by mechanical means. The hydraulic jets were set up in the cut and directed against its downstream face. As the face was under cut and sluiced away, the jets were moved progressively downstream. The cut was kept at a slight angle to the normal (to the shore line) so that the flow of drain water would be along the base of the face being cut and would sluice away material as it caved. The slope was normally dressed by hand or mechanical means as it was extremely difficult to avoid gullies and pockets during hydraulic grading. Stumps, roots, and buried logs also frequently necessitated removal by hand.

Hydraulic grading was cheap and easy where the bank was comparatively easy to cut and was free from logs and roots. Mechanical and hand dressing added considerably to the cost, however. Hydraulic methods possessed one inherent disadvantage, the sluiced material generally formed a broad flat bench extending out from the bank just under the water surface and terminating in a steep slope. This bench greatly increased mattress sinking difficulties. In fact, willow and articulated concrete mattresses sometimes broke and tore at the angle of this slope and the concrete slab mat piled up at the foot of the slope and did not completely cover it. Hydraulic grading has been superseded by mechanical grading methods.

Mechanical bank grading was first experimentally done in 1928 in the Vicksburg Engineer District. At present (December 31, 1931) the mechanical method has so demonstrated its superiority over other
MECHANICAL BANK GRADING OPERATION
Grading being done by dragline mounted on barge.
New Madrid Bend, September 26, 1930
methods that it is now standard for the lower river. Several types of grading plant have been developed largely by adaption of levee building machines. At present four draglines mounted on barges are in operation. These draglines are equipped with booms from 125 to 165 feet long and with from 4 to 8-yard buckets. Two of these machines were experimentally equipped with so-called "pull-hoe" attachments for the final grading operation. These "pull-hoes" were steel scraper blades which were operated by adjustable dipper sticks set in cradles which moved along the dragline booms. The "pull-hoe" was not successful since the positive operation of the scraper caused excessive jar and strain on the whole machine when a snag was encountered. The plant which at present gives the greatest promise of successful operation is a slack line bank grader now in use in the Memphis Engineer District. This consists of a "head" barge equipped with a vertical A frame and a mobile "tail" tower of the caterpillar type. In operation the head barge is moored some distance out from the bank and the tail tower operates just back of the top of the proposed grade. A bucket operates on a slack line between the barge and the tail tower. This bucket makes its cut from the top of the bank toward the head barge. At the end of the cut the bucket "dumps" into the river. This unit is particularly well adapted to subaqueous work.

As compared with hydraulic methods, mechanical bank grading methods possess one advantage too great to be overlooked. The hydraulic method merely cut the upper bank to an approximate grade. It rounded off projecting points but did not remove them. A mechanical grader, on the other hand, not only cuts the bank to grade but can, when necessary, cut back projecting points thus producing a uniform horizontal bank curvature at the water line. This is of great advantage not only as to ease and economy of mattress sinking but also as to the permanency and effectiveness of the completed revetment.

The mechanical graders are about twice as fast as hydraulic methods in terms of linear feet of bank graded. In terms of yardage moved, however, the ratio is 3 or 4 to 1 in favor of the mechanical plant. The final step in revetment construction is the pavement of the upper bank. In constructing this pavement, it is a cardinal principal that bank drainage must be facilitated so far as is practicable. This applies not only to surface drainage but also to subsurface seepage. The interruption of surface drainage causes scour holes at the upper edge of the pavement and ultimately undermines large portions of pavement. Seepage through the banks is likely to undermine the pavement at any point. Seepage can, however, usually be corrected with comparative ease since it normally results from standing water in depressions behind the bank such as old bayous, lakes, or levee borrow pits. The drainage of these depressions usually removes the trouble.

For a discussion of early bank protection methods, the reader is referred to the pamphlet "Bank Protection, Mississippi River". The earliest forms of upper bank protection consisted of mats of willow brush anchored by stakes driven into the bank and ballasted with stone. Usually these mats were made up of two layers of brush, the lower one perpendicular, and the upper one parallel to the bank line. It was at first thought that these mats would silt up with a protective coating of al-
luvial soil but this did not prove to be the case. The willows rotted rapidly and were entirely unsuitable for bank paving. By 1891 riprap had superseded willow brush as a means of bank protection. At that time approved practice called for a 10-inch layer of riprap laid on a 4-inch foundation of quarry spalls and extending from the low-water line to the height at which vegetation would flourish. Above the riprap a growth of willows or Bermuda grass was encouraged.

Attempts were also made to use brick for bank pavement. Experiments to this end were initiated as early as 1883. In 1894 experiments were conducted with baked clay blocks made on the site by the construction forces. In 1899 and later, a few practical tests were made with brick. Brick is relatively costly, however, and it has never been adopted as a standard bank pavement. In 1894 the first experiments were made to determine the utility of concrete for bank paving. These first tests, which were made on low grade concrete blocks, proved unsuccessful. In 1899 and 1900, a total of 230,500 square feet of upper bank was paved with different types of pavement in order to determine their comparative values. Concrete blocks, bricks, and monolithic concrete pavement were placed. Of these, the monolithic concrete pavement only appeared to give promise of successful use.

In 1909 experiments on a small scale were made with concrete monolithic pavement reinforced with wire mesh. Between 1910 and 1914 bank pavements made up of alternate stretches of riprap and concrete were placed at several points to test the relative merits of the two types of pavement. In 1915 the Mississippi River Commission adopted monolithic concrete pavement for normal use on the lower river. Experience has shown that the wire mesh reinforcement was unnecessary and its use has been abandoned. The present monolithic concrete pavement is 4 inches thick, laid on a carefully prepared 1 on 3 slope. At the upper edge of the pavement, an inverted wedge-shaped curb is cast integrally with the slab. This curb penetrates to a depth of 3 feet and serves as a cut-off against undermining the pavement by surface drainage. The upper edge of the pavement is of course the main point of attack by drainage. When serious undermining is threatened, additional protection is frequently afforded by placing a narrow strip of riprap immediately above the concrete. Expansion joints 4 inches wide are spaced 200 feet apart in the pavement. The bank beneath each joint is protected by a strip of concrete 2 feet wide cast far enough ahead of the main slab to take an initial set before the slab is poured. This strip is covered by a layer of tar paper before the slabs are poured upon it. Weep holes in the monolithic pavement are provided at intervals of 6 feet in each direction. The anchor cables extending from the subaqueous revetment to deadmen in the bank are embedded in the concrete slab when the latter is poured. Care is taken to provide a complete and impervious junction between the upper bank paving and the subaqueous mat. In cases where the upper edge of the subaqueous mat does not completely connect with the bank pavement, small connecting mats are laid which overlap the subaqueous mat. The junction between the concrete upper bank paving and the concrete subaqueous mat is illustrated on Plate LII. Present specifications require that all willow mattresses above mean low water be covered either with articulated concrete mat or with a layer of riprap.
CONCRETE UPPER BANK PAVING
Bank paving plant in operation, Avenue, Ark.
(38 miles below Helena), October, 1929
When riprap is used, it must be 20 inches thick on the willow mattress and must extend up the bank from the upper edge of the mattress for a distance of 6 feet. Throughout this zone the thickness of the riprap layer decreases until it is but 10 inches thick at its upper edge. Above this zone of riprap, the upper bank paving extends to the top of the bank.

Normally, the concrete pavement is placed from concrete mixing barges equipped with booms which reach to the top of the graded slope. Along these booms, bottom dump buckets operate thus allowing for positive placing of the concrete. Two such plants in the Memphis District are each capable of paving 500 squares per 20-hour day. A tower mixing plant has been used for bank paving but has proved unsatisfactory. A mix, wet enough to chute, is too fluid to hold to a uniform thickness on a slope of 1 on 3.

Present specifications for riprap upper bank pavement call for a regular pavement 10 inches thick laid by hand. The belt of riprap covering the upper edge of a willow mattress has been described above. The junction between a concrete mat and riprap paving is somewhat similar. The riprap must overlap articulated concrete mat by three blocks. On slab mat the overlap may be much shorter. In each case, the riprap is 20 inches thick at the upper edge of the mat. Specifications prohibit the placement of riprap pavement under water when the stage is more than 6 feet above mean low water.

Normally, the bank paving extends to the top of the bank, but observation of current action may sometimes permit economies to be introduced. For example, the point of maximum current attack is normally at the head of a caving bend during low water. As the stage rises, however, the point of maximum attack usually moves downstream so that at bankfull stages the greatest bank caving will be observed at the foot of the bend while the head of the bend may be free from erosion. Thus it is not always necessary to carry the bank pavement to the top of the bank at the upper end of the bend. Such an economy should not, however, be practised until actual observation and inspection at the site of the work indicates that it is safe.

Concrete cast in situ withstands the action of drift better than riprap or concrete blocks. Little, if any, concrete monolithic pavement on the Lower Mississippi has failed because of inferior mix. Monolithic concrete is the cheapest and most effective bank paving so far produced.

In special cases articulated concrete mat and concrete slabs are used for upper bank pavement. These methods are resorted to when the river stage does not permit the construction of monolithic pavement. They are also used for connecting mats when it is impossible to bring the monolithic pavement down to the upper edge of the subaqueous mat. Slabs and articulated mat are, however, inferior to monolithic concrete as upper bank pavement and should be used only when the construction of the monolithic slab is impracticable.

THE PROTECTION OF NECKS WITHIN DEEP BENDS.

Works designed to protect areas behind the river bank from overbank flow can not properly be classified as "bank protection" works as the term is defined in this chapter. A work planned as an indirect aid
to true bank protection is, however, of interest here. The dikes constructed to prevent erosion across the necks of deep river bends are therefore described in this chapter. As has been stated in Chapter II, the high-flood stages which now prevail as a result of the development of the levee system often cause dangerous erosion across these necks. This erosion frequently takes the form of deep "blue holes" whose upstream lips cave progressively from year to year. If no protection work is done, these blue holes ultimately break through the bank on the upstream side of the neck and a cut-off occurs. In the past, it has been customary to protect such a neck where necessary by an earthen dike extending longitudinally down the neck from the levee line near its base. These dikes were built with 8-foot crowns and with slopes of 1 on 3. They were built to grades above maximum high water and served to force the flood flow to follow the bends instead of cutting across them. The outer ends of these dikes were usually heavily revetted to protect them against the heavy scour. Structures of this type were built at the following sites:

Cowpen Neck (immediately above Natchez)—a dike 3 miles long was built in 1895.

Leland Neck (immediately above Greenville)—a dike over 1-mile long was built in 1903 and was subsequently extended until, in 1929, it was 2½ miles long.

Slough Landing Neck (61 miles below Cairo)—a dike built in two sections having an aggregate length of 6½ miles constructed in 1915. (Also known as Watson Point dike, see Plate LXIII.

Ashbrook Neck (head of the Greenville Bends)—a dike 5 miles long was built between 1915 and 1918.

These impermeable earthen dikes merely move the point of current attack away from the necks further out toward the points of the bends. Beyond their ends, deep blue holes frequently develop which sometimes cause the loss of portions of the dikes themselves. The dikes must frequently be extended until they reach nearly to the extreme ends of the necks.

The Leland Neck dike is of especial interest because of its importance and the difficulty of its maintenance. In 1882 this neck was 5,500 feet wide at its narrowest part but by 1903 this width had been reduced to 3,500 feet. Flood flow in that year caused deep scour across the neck near the controlling levee line at its base. Immediately after the flood, an earthen dike 6,250 feet long was built out from the controlling levee line along the neck. The high waters of 1904 to 1907 caused scour along the upper side of this dike and at its end. After the flood of 1907 it was extended an additional 3,000 feet. The 1922 flood again caused deep scour around and beyond the end of the dike. In 1926 the riprap protection at the end of the dike was renewed and a small brush and stone sill was built to prevent the extension of the blue hole at its end towards the upper bank of the bend. The flood of 1927 dangerously enlarged this blue hole, however, and the dike was thereafter extended an additional 4,000 feet making its total length about 13,250
LELAND NECK
Permeable dike built in 1930 to prevent overbank scour across the neck.
feet. The scour caused by the 1927 flood amounted to approximately one-half million cubic yards and the flood of 1929 threatened to cut off the neck. A total of about two and one-half million cubic yards was scoured out by this last flood, leaving a blue hole at the end of the dike 80 feet deep; 2,600 feet long; and 600 feet wide. The hole had an arm 300 feet wide which extended to within about 500 feet from the upstream side of the neck. Three hundred and thirty feet of the dike were destroyed during the flood of 1929 and an additional 1,200 feet was badly wave washed. It was evident that another high water would complete the cut-off unless effective steps were taken to prevent it. In 1929-30 a permeable dike was built. This permeable dike leaves the old earthen dike at a point 8,750 feet from the controlling levee and extends along the bank on the upper side of the neck for a distance of 5,000 feet. For 4,600 feet of its length, the top of the dike is 20 feet above the ground surface. At its outer end, however, the grade of the top is gradually decreased over a distance of 400 feet to a minimum of 10 feet above the ground surface. The right of way on which the structure is built is 80 feet wide over the greater part of its length.

The dike is a permeable pile bent structure designed to concentrate between its upstream and downstream faces fully one-half of the head across the neck. It is expected that this structure will be comparatively free from violent scour at its outer end. The dike itself consists of a line of pile bents, 20 feet wide and spaced 10 feet apart. Each bent consists of three piles driven 10 feet on centers, in line normal to the axis of the dike. The piles are driven to a penetration of 20 feet and the bents are substantially cross braced. The upstream face of the dike consists of horizontal 4 by 8-inch walings spaced 8 inches apart. The downstream face consists of vertical 4 by 4-inch screen poles spaced 8 inches on centers and bolted to horizontal stringers. The right of way is heavily paved with a reinforced concrete mat to prevent scour through the structure. To check any flow of water along the dike, five short wing dikes extend from its downstream face across the right of way and well into the woods below. These wings are constructed of timber cribs filled with concrete blocks. The upper lip of the 1929 blue hole has been heavily riprapped to prevent its extension upstream into the right of way of the dike.

COMPARISON OF VARIOUS TYPES OF BANK PROTECTION.

A comparison of various methods of bank protection may be restricted to consideration of the different types of revetment. Non-continuous protection works have in general proved to be failures on the Lower Mississippi River. The present standard type of lower river bank protection is a continuous revetment extending from the top of the bank to a line somewhat beyond the thalweg. The standard types of mattress in present use are the framed mattress, the fascine mattress, the articulated concrete mat, and the slab mat. Other mattress types described in this chapter have all proved unsatisfactory and have been abandoned.
Of the two willow types, the framed mattress is cheaper and less permeable; it has a longer life; and is the easier to construct and sink. It possesses an additional advantage over the fascine mattress in that it can be built in a convenient location and then towed into position and sunk. The fascine mattress must, on the other hand, be built on the site where it is to be sunk. Thus, the framed mattress is particularly adapted to revetment construction in locations such as harbors where river traffic is heavy. Willow has in the past been considered as being relatively permanent under water but recent investigation indicates that this is not so. Where protected by a covering of silt and free from direct current attack, willow mattresses last for long periods under water. Under the direct attack of the current, however, they disintegrate relatively rapidly. Under such conditions the fascine mattress usually fails before the framed mattress. The individual willows in the fascines are not so compactly bound together as in the framed mattress and are therefore subject to greater vibration and chafing. They wear rapidly and rot. All willow mattresses are particularly vulnerable against attack at two points, the water surface and the toe of the revetment. At the water surface, rapid rotting of the willows necessitates expensive maintenance while at the toe (or outer edge) of the revetment, undermining is very likely to occur as a result of the river's normal tendency to deepen its channel in a revetted bend. The stiff willow mattress does not conform to this undermining but extends out over the caving bank without touching it. Ultimately this unsupported belt of mattress fails. The only measure which can be successfully applied to prevent or retard such failure is the extension of the mattress to a line well beyond the thalweg. In general the life of a willow mattress subjected to strong current attack is not more than about ten years.

Concrete revetment possesses certain definite advantages over willow. Concrete mat is cheaper than willow both in first cost and in maintenance. The effective weight of a concrete mat (in place) is about 18 pounds per square foot as compared with about 5 pounds per square foot for willow mattress. This makes the concrete mat much the more stable of the two. The deficiency in the effective weight of the willow mattress can be overcome by greatly increasing the amount of rock ballast, but this in turn entails a proportionate increase in cost of construction. The flexibility of concrete mat is its second great advantage over willow. This permits a concrete mat to conform intimately to the irregularities of the river bank and bottom and also insures against undercutting at the toe of the revetment. A properly designed concrete mat may also be expected to have a longer effective life than a willow mattress, since the concrete does not decay and since the metal ties between units are comparatively well protected. Present specifications require these metal ties to be made of non-corrosive metal.

The concrete slab mat has in the past been more expensive than the articulated concrete type. Recent changes both in production and in method of sinking are reducing the cost of this mat to a figure comparable with that of the articulated concrete mat. The slab mat is, however, less adaptable than the articulated mat to changes in bank conformation. It is difficult to sink and is likely to pile up on the river bottom rather than form a continuous covering. Recent inspections by diving have indicated that this tendency to pile up is greater.
than had been previously assumed. The present practice of laying the slabs butt to butt (instead of overlapping as had been the previous practice) may overcome to some extent this tendency to pile up.

The manifest advantages of concrete over willow together with the increasing scarcity of willow growth have resulted in a gradual decrease in willow construction and a corresponding increase in the amount of concrete revetment placed. While the willow mattress is still considered a standard type, concrete revetment has been used almost exclusively during the past two years. It is probable that improvements in the concrete mattress will ultimately lead to the complete exclusion of all other types of subaqueous revetment on this river.

REVETMENT FAILURES.

The usual causes for revetment failure have been discussed earlier in this chapter. For willow mattress, the principal cause of failure is rotting. This is most active at the water surface when the revetment is subjected to alternate wetting and drying. Undermining also frequently occurs at the toe of a willow mattress. The articulated concrete mat depends for its continuity upon the metal connections between blocks and units. When these fail, the mat disintegrates. The life of the articulated mat is therefore limited to the life of these fastenings. As has been stated, present specifications require the use of non-corrosive metal for this purpose.

The above does not, however, include all causes of failure. The revetment may be flanked out at either its upstream or its downstream end, or the upper edge of the bank paving may be attacked. This attack on the upper bank paving may be the result of improper drainage, rain wash, or overbank scour. The cures for improper drainage and rain wash are obvious. There is no need to discuss them here. The overbank scour which attacks revetment is usually the result of flood flow across the necks of deep bends. Methods of controlling this scour have been discussed earlier in this chapter.

Attack at the upstream or downstream end of a revetment is usually the result of miscalculation of proper revetment location and length. The revetment must extend over the whole zone of active river attack. Otherwise caving sets up either above or below it and a large part of the revetment is flanked out. The caving of the bank behind it causes one end of the revetment to project into the current. This sets up eddy action which increases the rate of caving. The extension of the revetment to cover these eddy pockets is both difficult and expensive. It is better to avoid the necessity for such repair by adequate construction in the first instance.

HISTORICAL OUTLINE OF THE DEVELOPMENT OF BANK PROTECTION ON THE LOWER RIVER.

The individual types of bank protection have been briefly discussed elsewhere in this chapter. It is, however, of interest to outline the growth of bank protection operations in general. As compared with levee construction, bank protection operations are of relatively recent
development. In fact, active bank protection construction was not undertaken until 1878 although the need for it was recognized at much earlier dates. In his report of 1851 (see Committee Document No. 5, 70th Congress, 1st Session), Charles Ellet strongly recommended the protection of the river banks either by "sheathing" them to prevent caving, or by deflecting the direction of the current so it would cease to wear the bank away at points of danger. He regarded bank protection as necessary principally for the prevention of cut-offs. The report of the Delta Survey (Humphreys and Abbot) is silent on the subject of bank protection. The Suter report of 1875 (see Annual Report of the Chief of Engineers for that year) advocated bank protection as a part of the improvement of the river for navigation. Suter recommended the use of bank protection works along banks whose recession would be injurious to the channel. He proposed to afford this protection either by a succession of stone dikes or by a continuous stone revetment. The report of the Board of Engineers of 1879 (see Annual Report of the Chief of Engineers for that year) agreed with Suter in recommending the use of bank protection where bank recession was prejudicial to navigation. This board proposed the use of mattresses for this bank protection with the proviso, however, that lighter and less costly structures designed to induce silting would be used wherever possible.

Coincidentally with the studies of the Board of Engineers of 1879, the first bank protection works on the Lower Mississippi were actually begun under the direction of the Engineer Department. Upon the creation of the Mississippi River Commission in 1879, navigation improvement works passed under the jurisdiction of that body, and with them passed the experimental bank protection operations then under way. These early operations were undertaken for the improvement of the harbors at Memphis, Vicksburg, Natchez, and New Orleans. In New Orleans Harbor, cane mattresses were sunk in 1878 in an attempt to protect the city wharves. As has been shown elsewhere in this chapter, this work was entirely unsuccessful and the use of cane mattresses was not again attempted. In 1878 bank protection was constructed at Delta Point opposite Vicksburg. After the Centennial Lake Cut-off of 1876, the attack of the current caused this point to recede and a movement of the river channel away from Vicksburg Harbor resulted. The protection works at Delta Point were planned to prevent further channel movement. Framed mats were used here and proved fairly successful. Floating screen dikes were also employed but they proved entirely inadequate in the deep water and strong currents encountered at this locality. Screen dikes were installed in Memphis Harbor in 1881 and in Giles Bend immediately above Natchez during the same year. These dikes proved entirely unsuccessful and their use on the river was soon abandoned. These early protection works were generally of little or no value. Already experience was demonstrating the need for heavier and more effective construction. The Mississippi River Commission found itself confronted with a bank protection problem which was beyond the experience of engineers up to that time. The extent of the work and the severity of the current attack necessitated operations on a scale probably greater than had ever before been attempted. The Commission at first concentrated its efforts on certain reaches where the need for protection was immediate, and
where it was hoped that experience would lead to the development of inexpensive yet adequate forms of bank protection. Perhaps the most notable experimental work conducted by the Commission during this period was that at Plum Point reach (168 miles below Cairo) beginning in 1881. In that year mesh screens, hanging vertically and supported by piles, were constructed in the reach. These screens failed utterly and no attempt was thereafter made to use them. Woven willow mattresses of differing degrees of thickness and of differing degrees of compactness, elasticity, and roughness of surface were then installed at Plum Point reach and elsewhere beginning in 1882. The process followed consisted first in the placement of the subaqueous revetment; second, in the upper bank grading; and third, in the placement of the upper bank protection. At first this upper bank protection consisted of a mattress of willow brush ballasted and anchored to the bank. As has been shown elsewhere in this chapter, this method of upper bank protection proved utterly unsatisfactory and was soon superseded by riprap paving. The woven willow mats also proved a failure. A heavier subaqueous mattress, more impervious to the current, was needed.

The period 1878 to 1882, however, witnessed the first development of the framed willow mattress which was first placed in Memphis Harbor between the years 1878 and 1882. The value of this mattress was immediately apparent. Its use was soon adopted on the lower reaches of the river below Vicksburg where, with improvements dictated by experience, it continued to be the standard mattress type below the mouth of Old River. Its supremacy has, however, been successfully challenged since 1928 by the articulated concrete mat.

The year 1884 marks the introduction of the submerged spur dike; first used in New Orleans Harbor during that year. The submerged spur failed, however, to meet expectations and its use after 1907 was discontinued.

The river and harbor act of August 5, 1886 contained a provision which prohibited the construction of protection works until it should be found that the desired bank stability could not be secured by contraction works. This limitation was based upon the theory that a river when once regulated will cease to scour its banks. Experience soon showed that the unprotected banks opposite contraction works receded rapidly and revetment was again admitted to be a necessary adjunct to channel stabilization in the interest of navigation.

In 1888 the Commission’s policy was extended to include protection of levees by the revetment of caving banks. The first revetment built for this purpose was at Bolivar, Mississippi (417 miles below Cairo) in that year.

The fascine mattress now made its appearance upon the river. In 1893 the first mattresses of this type were placed at Daniels Point (154 miles below Cairo). The fascine mattress was almost immediately adopted as standard for the river between Cairo and Vicksburg and in 1925 its use was extended to the mouth of Red River. The fascine mat, like the framed mattress, is now being superseded by concrete revetment types.

In 1896 the Mississippi River Commission adopted a policy of dredging for channel improvement. The policy of channel improve-
ment by means of contraction works was thereupon abandoned and was not readopted until the passage of the present flood control act (act of May 15, 1928). Although contraction works had been abandoned, the necessity for bank protection still remained. Indeed, the Mississippi River Commission in 1911 adopted a general policy of bank protection to protect exposed levee lines from caving. Between that year and 1927 a considerable amount of revetment was constructed for this purpose. As has been stated in Chapter VI, however, this policy is generally economically unsound. Present policy contemplates the construction of set-backs as against the use of bank protection.

In the year 1898 abattis were used experimentally near Point Pleasant, Mo. (81 miles below Cairo). The use of the abattis was intermittently continued until 1907. Since that year, however, none has been built on the lower river.

Upper bank protection also claimed the attention of the Commission. Since 1891 riprap pavement had been considered the standard type of upper bank protection. From time to time substitutes for stone had been experimentally tried. Brick, baked clay blocks, and concrete blocks had been experimentally used for upper bank pavement and for mattress ballast but were not adopted. The use of monolithic concrete upper bank protection was first adopted in 1900 in the Second New Orleans District. Further experience in that district and in the other districts resulted in the adoption by the Commission in 1915 of this type of bank paving as standard on the lower river.

In 1900 an unsuccessful attempt was made to use board mattresses at Bondurant Chute (about 50 miles above Natchez). This has been described earlier in this chapter. Since 1900 the use of board mattress on the lower river has been confined mainly to foot mattress for permeable pile contraction dikes. Two sections of board mattress were constructed in Plum Point reach in 1931. This mattress has not yet been in place long enough to permit an estimate of its value to be made.

The growing scarcity of the supply of willow for revetment and the defects inherent in willow revetment construction directed the attention of the Commission toward the development of more durable revetment types. In 1907 the District Engineers were directed to make studies to this end. In the following year (1908), an attempt was made in the Memphis District to substitute rough sawed lumber for willows in fascine mattress construction. The experiment was a failure. The mat was so heavy that it broke the launching ways of the mattress barge. Moreover, it had insufficient buoyancy, became water-logged, and sank before any considerable length of mattress could be built.

The efforts to develop a better type of revetment continued, however, and resulted in the experimental development of the monolithic concrete mat in the Memphis District in 1914. This was followed in 1915 by the development of the articulated concrete mat in the Vicksburg Engineer District. The monolithic slab proved unsatisfactory and was soon abandoned. The articulated slab, however, proved successful and soon became widely used in the Vicksburg District. In 1928 this mat was introduced into the Second New Orleans District, and in 1931 into the Memphis District. Improvements in design have
resulted in the development of the present standard articulated concrete mat which has been described earlier in this chapter. In 1924 the Memphis Engineer District developed the concrete slab mat. This mat is now a standard revetment type. It has been described earlier in this chapter. Within the last few years, concrete mat types have tended to supersede the willow mattress.

The development of bank protection works on the Lower Mississippi thus far may be summarized as having demonstrated the superiority of the continuous form of bank protection (revetment). Early efforts to provide protection by the use of non-continuous structures were all failures. Experience has indicated that revetment must be heavy, substantial, and impermeable to be successful on the lower river.

A recent development of great importance is the under water revetment survey which was undertaken in 1931 and has only recently been concluded. The report of this survey has not yet (December 31, 1931) been submitted. It may, however, be stated that the survey has shown that the attack of the current upon subaqueous mattresses is much more severe than had been assumed in the past. The report of this survey will doubtless point the way to material improvements in revetment design.

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 which are no longer effective due to channel changes. In addition, about 19,000 linear feet 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. Present status of revetment on the lower river is shown on Plate LVII.

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 linear feet per river mile. Between the mouth of the Arkansas and the mouth of the Yazoo River (at Vicksburg, Miss.) there are about 1,224 linear feet of effective bank protection per river mile. Between Vicksburg and the mouth of Red River, the average is 732 linear feet of effective bank protection per mile. Between the Red River and New Orleans, the average is only 109 linear feet of effective bank protection per mile, excluding 68,000 feet of bank protection in New Orleans Harbor proper. No bank protection has been placed by the Mississippi River Commission below New Orleans.

COSTS.

No purpose is served in discussing costs of bank protection methods which experience has shown to be unsuitable for Lower Mississippi River use. The present discussion is therefore limited to those types of revetment and bank paving which have proved successful in actual use.

Table XXXIX shows the total expenditures for bank protection for the period 1881 to 1931, inclusive (fiscal years). For the period 1881-1917, the aggregate expenditure for the entire period is shown. Yearly expenditures are, however, given for subsequent years.
In studying this table, one is struck by the wide variation in the cost of bank protection per running foot of bank and the high cost of maintenance and repair. The costs of the period 1881-1917 are not to be compared with later figures. In the early years, much work of an experimental nature was done which proved to be totally inadequate for its purpose. The maintenance of such works was usually very small since the works themselves were soon destroyed or abandoned. Moreover, the insufficiency of funds in the early years prohibited the prosecution of much maintenance and repair works even though urgently needed. Thus the relatively small size of repair and maintenance expenditures as compared with expenditures for new work during the period 1881-1917 is not a real index of the efficiency of these early works.

The cost per linear foot of bank protection is not a true basis of comparison between jobs. The height of bank, the depth, the amount of clearing and grading necessary, and other important factors all vary with the location of the individual job. It is worthy of note, however, that since the adoption of the present flood control act (1928 to 1931, inclusive) the cost per linear foot of new bank protection has exceeded the average cost for the period 1918-1931 in only one fiscal year (1930). The average cost of new work for the period 1928-1931 inclusive, is $61.33 per running foot as compared with a cost of $64.86 for the period 1918-1931. This indicates that despite improvement in bank protection design since 1928, the first cost of bank protection is decreasing. The ratio of cost of new work to that of maintenance and repair for the...
period 1928-1931, inclusive, is 1.62 to 1. The same ratio for the entire period 1918 to 1931, inclusive, is 1.42 to 1. This rise in ratio does not, however, indicate that the revetment now being constructed has a shorter life than former work. The maintenance during this period was done largely on bank protection placed before 1928. These comparisons therefore indicate better work; pursued on more comprehensive lines than formerly.

Table XL affords a comparison in the costs of various types of revetment. It will be noted that this comparison is based on a unit area (the square—100 square feet) rather than on unit length of bank protected. As has been stated above, the unit length of protected bank is not a true basis for comparison.

**TABLE XL**

Average Costs, Bank Protection Work—Working Seasons 1929-30 and 1930-31

<table>
<thead>
<tr>
<th>Type of Revetment</th>
<th>Cost per Square (100 Sq. Ft.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Slab Mat</td>
<td>$22.38</td>
</tr>
<tr>
<td>Articulated Concrete Mat</td>
<td>13.36</td>
</tr>
<tr>
<td>Willow Fascine Mattress</td>
<td>18.47</td>
</tr>
<tr>
<td>Framed Willow Mattress</td>
<td>15.42</td>
</tr>
<tr>
<td>Upper Bank Paving</td>
<td>11.75</td>
</tr>
<tr>
<td>Monolithic Concrete</td>
<td>11.75</td>
</tr>
<tr>
<td>Riprap</td>
<td>16.18</td>
</tr>
</tbody>
</table>

The cost of the slab revetment as shown in the above table is higher than any other type of revetment in present day use. Experience has pointed the way to improvements in slab construction which point to a reduction in its cost to a figure more directly comparable with other types. This table indicates that articulated concrete mat is probably the cheapest form of bank protection.
CHAPTER IX.

CONTRACTION WORKS.

INTRODUCTION.

The improvement of a river for navigation by means of structures designed to decrease the low-water channel width, and increase the low-water channel depth is known as "regularization" or "regulation". Two classes of works are used in river regulation; bank protection and contraction works. The term "bank protection" includes all works designed to prevent the widening of the channel through bank scour. Bank protection has been described in Chapter VIII. Contraction works include all structures designed to restrict the low-water flow of the river to a channel narrower than its natural one. Any sound plan for river regulation must call for a navigation channel not only of the requisite width, but also free from excessive curvature and adapted to the navigation methods which will be used on the river when regulation has been fully effected.

The object sought by river regulation is in general opposed to that of flood control by the levee method. The primary object of levees is the creation of a flood channel capable of passing the flood flow of the river at reasonable flood stages. Regulation works, on the other hand, are designed to constrict the low-water flow. They normally effect this purpose at the expense of some sacrifice of the between-banks channel cross section, with a corresponding possible reduction in the flood capacity of the channel. In the execution of a river regulation plan on the river subject to dangerous floods, it is necessary to exercise care to effect a reasonable compromise between the divergent needs of navigation and flood control.

The construction of contraction works in any given river reach interferes with the free flow of water at low stages through that reach. It therefore follows that the immediate effect of the works is to increase stages above the contracted reach. No change occurs, however, in the channel below the zone of contraction. As a result, slope and velocity of flow through the zone of contraction are increased. When the bed of the river is composed of readily erodible materials (as is the case with the Lower Mississippi), scour of the bottom results. This erosion will extend throughout the zone of contraction, thus producing a tendency for the depression of the surface profile of the water at low stages throughout and above the treated reach. Thus, while the immediate effect of contraction works is to raise the low-water surface above the reach, the ultimate effect may be the depression of the low-water surface through and above the zone of contraction. The extent of this depression depends upon the degree of contraction. Over-contraction causes a lowering of low-water stages which may have serious effects elsewhere in the channel, while insufficient contraction fails to produce satisfactory navigation conditions in the reach under improvement.

This tendency for contraction works to lower stages is better understood if the relation of contraction works to pools and crossings in the river channel is considered. As has been stated previously in this volume, crossing reaches are characterized by bar formations and diffi-
cult navigation conditions. Pool reaches, on the other hand, normally possess ample depths for navigation at all stages. Contraction of the crossing reaches is therefore normal procedure. The bars in these crossing reaches act as submerged dams which maintain the water levels in the pools above. When, as a result of contraction works, these bars are subjected to considerable scour, the water levels in the pools above are appreciably lowered. Over-contraction may therefore result in lowering the bars without increasing the depths of water over them. In extreme cases, navigation is often harmed rather than benefited.

In any general plan for river regulation, two primary objectives are sought, viz., the development of adequate navigation channel dimensions and correct channel alignment. While the first objective is of course the more important, experience has shown that no plan is entirely satisfactory which does not fulfill the second objective also. Navigation demands a channel of smooth, easy sinuosities, free from sharp or objectionable bends. Moreover, in a channel of smooth, uniform curvature, bank caving is reduced to a minimum and the growth of objectionable bars is lessened. While abrupt changes in the direction of flow are to be avoided, it does not follow, however, that long, straight reaches are desirable. In fact, such reaches should be avoided since they are usually subject to deterioration.

Where the channel of the river is divided by an island, deficient depths usually prevail. For such cases, contraction works generally offer a simple and economical remedy, the typical procedure being to construct dikes in such a manner as to concentrate the flow in one channel. In making the choice as to which channel shall be left open, several things must be considered. The main consideration is the channel alignment above and below the divided channel. In deciding which channel will be the better for navigation; the greater slope and velocity of the shorter one must be weighed against the longer distance and flatter slope of the longer one, and the decision must be made largely on a basis of experience. If the deficiency in depth in the longer channel is slight, and particularly if this channel is already carrying a major part of the flow, there will be less disturbance to the regimen of the river if the shorter channel is closed. On the other hand, if the entire flow is diverted through a shorter channel, the acceleration of its velocity is likely to be followed by bank caving which may necessitate extensive bank protection.

The distinction between river regulation and channel shortening for flood control must be thoroughly understood. Channel shortening is discussed in Chapters II and XI. The purpose of regulation is the development of an adequate navigation channel which can be economically maintained. Any channel shortening which may result from regulation is merely incidental to this purpose. Proposals for channel shortening, on the other hand, take the form of radical channel realignment (usually by means of wholesale cut-offs) and material increases in slopes.

Regulating works may be classed under two general headings; transverse works and longitudinal works. The transverse work is built at a decided angle to the direction of the current (usually pointed slightly downstream). Transverse works are frequently known as spur dikes. The longitudinal work approaches parallelism to the direction of flow. Longitudinal works are frequently known as training works.
Their purpose is either to prevent undue disbursement of the current, or to contract it gently and gradually. In the former case they are approximately parallel to the desired direction of current; while in the latter case they are normally constructed to cause a gradual but progressive constriction of the channel. Regulating works may be either permeable or impermeable, depending upon their purpose and the materials from which they are constructed.

The theory of contraction works is not new. In fact, the use of such structures has been known to man from times of remote antiquity. Perhaps the earliest instance of such use is found on the Nile River where a system of stone spur and longitudinal dikes was built by the Pharaohs. Regulating works have been constructed on many important European rivers. Among the important rivers in the United States which have been improved by regulation are the Upper Mississippi, the Tennessee, and the Sacramento. An extensive project for the improvement of the Missouri by this method is now in progress. Contraction works were first employed on the Lower Mississippi as early as 1882. The present project dates from 1928, however.

Any proposed system of regulating works must have a definite objective. So far as navigation is concerned, on the Lower Mississippi the objective of the present navigation project is a channel 9 feet deep and 300 feet wide, between Cairo, Ill., and Baton Rouge, La. This includes a stretch of river some 842 miles in length. For the preparation of a comprehensive plan for regulating this extensive reach, considerable data are required. Accurate hydrographic and hydrometric surveys covering the entire reach are essential. This information exists in the records of the Mississippi River Commission which are complete and continuous from 1879 to the present.

The effect of the completed Lower Mississippi River regulation project upon flood heights is an interesting subject for study. The permeable pile spur dikes used for contraction works on the lower river encourage the deposition of silt. These dikes have been used on the Mississippi River between St. Louis and Cape Girardeau; and dikes of a very similar type have been built on the Lower Missouri River. On that river they have been constructed to about the height of the bank and have caused the bank line to build out almost to their ends. This has of course caused a decrease in the total channel cross section between banks which has been only partially offset by scour on the river bottom. On the Lower Mississippi, the tops of the dikes are approximately 17 feet above mean low water, while the banks average from 30 to 35 feet above that datum. It is expected that deposit will occur between these dikes (as has occurred on the Middle Mississippi and Lower Missouri), and that this deposit will ultimately reach an elevation where the growth of vegetation will take place during the low-water season. This will induce further deposit and the fill should eventually reach the elevation of the natural bank. This loss in channel cross section can be compensated for only by deepening the channel, since channel widening will be prevented by bank protection. As has been shown above, however, scour on the river bottom is very dangerous and can not be allowed to occur to an extent sufficient to compensate fully for the loss in channel width. It is to be anticipated, therefore, that flood stages will be increased where any considerable stretch of the river is
affected by contraction works. The amount of this increase may, however, be somewhat less than might normally be expected. The present regulation plan is designed to produce a channel of uniform, easy curvature and even slopes whose relatively great hydraulic efficiency may be effective in lessening the amount of this increase in flood stages. Studies have been initiated which should settle this question definitely thus permitting a determination of the increase (if any) in levee grades necessary to compensate for the loss in channel capacity due to the regulation plan.

This introductory discussion is preliminary to the description of works on the Lower Mississippi which is set forth on the following pages. No attempt is made here to prepare a complete treatise on river regulation. The statements made and the conclusions reached should be considered solely in their application to the Lower Mississippi River.

CONTRACTION WORKS AT THE MOUTHS OF THE MISSISSIPPI.

This volume relates primarily to the improvement of the Lower Mississippi for flood control and river navigation. Since navigation improvements at the mouths of the river relate to seagoing navigation, no attempt will be made to describe them in detail here. Discussion in this chapter will be confined to a brief summary of the jetty work undertaken in the Passes. Early attempts to improve the Passes by agitation, scraping, and dredging are outlined in Chapter VII. European engineers had utilized jetties for the improvement of the mouths of alluvial rivers for many years before any serious attempt to build jetties in the Mississippi Passes. While these European works had not been universally successful, the main principles underlying the design of such works were understood as early as 1840.

Humphreys and Abbot, in the report of the Delta Survey (1861) considered the possibility of jetty construction at the Passes. They recommended against jetties, however, since they believed that bars would form at the seaward ends of the jettied channels at rates which would necessitate the extension seaward of the jetties at the rate of about 700 feet a year. A more economic method would be, they believed, systematic agitation and scraping.

The first attempt to construct jetties at the Passes was that of Craig and Rightor. In 1856 these men were awarded a contract for opening ship channels, 300 feet wide and 20 feet deep through Southwest Pass and Pass a l’Outre. The contract was later amended to require a channel depth of only 18 feet. Plans for the work included the closure of all mouths except the two under improvement. Craig and Rightor actually constructed two lines of jetties. One was 3,000 feet in length and was situated on the west side of Southwest Pass. The other was located on the north side of Pass a l’Outre and extended diagonally across the current for a distance of 550 feet. These jetties were constructed by driving piles at intervals of 5 feet and connecting them by a framework of heavy timbers and planking. These works were unsuccessful. A portion of the line in Southwest Pass was destroyed by storms. Eventually the contractors abandoned the jetties as being impracticable.
In the year 1874 Mr. James B. Eads, a civil engineer, proposed to open navigation channels at the mouths of the river by means of jetties. The act of March 3, 1875 authorized him to carry out his project in South Pass. As first approved, the project called for a navigation channel, 350 feet wide with a depth of 30 feet. Subsequent legislation, however, reduced the channel width to 200 feet and the depth to 26 feet, provided that a depth of 30 feet be maintained along the center line of the channel. Eads was given wide latitude in the character and location of his improvement works, the main limitation upon his operations being that the jetties be not less than 700 feet apart. The Eads jetties were patterned after those at the mouth of the River Maas in Holland. They consisted of successive layers of willow mattress sunk in place and heavily loaded with stone. These mattresses were about 2 feet thick, 35 to 50 feet wide, and 75 to 100 feet long. In construction, piles were driven along the lines of the proposed jetties and the mattresses were moored to these piles and sunk. Two lines of jetties were constructed prolonging the east and west bank of the Pass respectively. These lines were laid in curves so as to make the outer end of the channel normal to the direction of the littoral currents of the Gulf. The actual construction of these jetties required four years. By the summer of 1879, project dimensions had been attained in the channel. Since the construction of the Eads jetties, some dredging has been required in the channel from time to time. The amount of this dredging can not be considered excessive, however, nor has it been necessary to prolong the jetty lines as Humphreys and Abbot feared would be the case.

In 1902 the improvement of Southwest Pass by jetties was authorized by the act of June 13 of that year. By 1923 this improvement was finally completed. The width between the jetties is about 1,750 feet. These works have been successful in maintaining a channel somewhat less than 1,000 feet wide and about 35 feet deep without resort to dredging except during flood stages. For detailed discussions of the jetties in the Passes, the reader is referred to "The Mississippi Jetties" by E. L. Corthell and to the Annual Reports of the Chief of Engineers for 1905, 1906, 1907, 1908, 1909, 1914, 1919, 1922, and 1923.

CONTRACTION WORKS ON THE LOWER MISSISSIPPI PRIOR TO 1928.

The construction of contraction works on the Lower Mississippi River may be said to have begun with the organization of the Mississippi River Commission in 1879. The need for contraction as a means of channel improvement was, however, recognized before that year.

Maj. C. R. Suter, Corps of Engineers, covered the subject of navigation improvement on the Lower Mississippi in his reports of 1875 and 1878. These reports are published in the Annual Reports of the Chief of Engineers for those years. Suter stated that reaches of excessive channel width were characterized by poor navigation channels. He advocated the contraction of the low-water channel width throughout such reaches by means of spur and longitudinal dikes and sills. He recommended that these dikes be built to an elevation of about 10 feet above extreme low water. He regarded it as feasible to obtain a 10-foot low-water channel by contractions. Suter recommended that the
low-water channel as fixed by contraction works should not coincide with the high-water channel. This recommendation was in line with his theory that the heaviest detritus would be found in the path of high-water flow.

The Board of Engineers of 1879 discussed low-water navigation in their report (see Annual Report of Chief of Engineers for that year). This board concluded that difficult low-water navigation conditions characterized crossing reaches whenever the low-water channel widths of these reaches exceeded about 3,500 feet. The contraction of the low-water channel width throughout these reaches was recommended. Plum Point Reach (145 miles to 183 miles below Cairo) was at that time characterized by very difficult low-water navigation conditions. The board considered that this reach would offer a severe test to any experimental contraction works which might be constructed and accordingly recommended that the improvement of this reach be undertaken. The board further recommended that the first experimental contraction works be of light timber and brush construction. It believed that inexpensive structures of this nature would suffice. The possibility was, however, recognized that heavy, more costly works might be required.

Actual construction of contraction works on the Upper Mississippi antedated the organization of the Mississippi River Commission. In 1878 the improvement of the reach from the Des Moines Rapids to the mouth of the Illinois was undertaken by the Engineer Department. In 1881 the improvement of the river between the mouths of the Illinois and the Ohio Rivers was similarly undertaken. Improvements in these reaches were placed under the jurisdiction of the Mississippi River Commission by the act of August 2, 1882 and remained under that Commission's control until 1886. Since they are outside the scope of this volume, however, they are not described here. Detailed descriptions of these works are found in the Annual Reports of the Chief of Engineers for the years 1879 to 1886, inclusive.

The first Commission report (February 17, 1880) contains an outline of the existing Commission policy regarding navigation improvement on the lower river. The Commission stated in this report that, in certain reaches of the river, bank erosion produced excessive channel widths which in turn caused excessive shoaling to the great detriment of navigation. The more nearly the high-water channel approached uniform width, the more uniform the depth and velocity of the current would become. Examination of the river below Cairo had shown that wherever the low-water channel width was much greater than 3,000 feet, navigation difficulties were experienced. When the channel width did not exceed 3,000 feet, good navigation conditions existed. The Commission therefore recommended the contraction of the low-water channel to that critical width. The Commission believed that this degree of contraction could be obtained by means of light, inexpensive works of poles, brush, and similar material. These contraction works were to be permeable. The report approved and adopted the plan of the Board of 1879 for the improvement of Plum Point Reach (145 to 183 miles below Cairo).

The Commission report of March 25, 1881 amplified the policy set forth above. In addition to Plum Point Reach, the Lake Providence Reach (extending about 25 miles upstream from Lake Providence, La.)
was selected for improvement. The Commission believed that the complete improvement of these two reaches would be of greater permanent value than the partial improvement of the six reaches in which the need for improvement was recognized. Plum Point Reach contained seven shoal areas. Its minimum low-water depth was about 4\(\frac{1}{2}\) feet. Its width was excessive; in some places approaching 10,000 feet. Lake Providence Reach contained six shoal crossings, had a minimum low-water depth of 4\(\frac{1}{2}\) feet, and varied in width from 5,000 to 10,000 feet. It was proposed to contract the low-water channel width to 3,000 feet throughout both of these reaches by permeable spur dikes and training works. As has been seen in Chapter VIII, this program also called for bank protection works where necessary. It was believed that the deposition of sediment by the decelerated current behind these contraction works would build up new river banks which could be covered by bank protection works where necessary, and would serve effectually to stabilize the river channel. Navigation depths of 10 feet were anticipated as a result of these proposed improvements. As to types and methods of construction, the fact was recognized that the were still in the experimental stage. Experience would, however, the Commission believed, point the way to sound practice.

These proposals received the approval of Congress and the necessary funds to undertake them were appropriated. The Plum Point Reach project is illustrated on Plate LVIII. The dikes built here may be divided into four groups namely, the Osceola Bar dikes, the Plum Point dikes, the Keyes Point (or Gold Dust) dikes, and the Elmot Bar-Island No. 30 dikes. The Osceola Bar dikes were constructed to close the channel behind Osceola Bar and Bullerton Towhead. Work was begun in 1881 and completed in 1884. These works consisted of both longitudinal and spur dikes. Due to poor location and inadequate design, the longitudinal dikes soon failed but the spur dikes were successful in closing the channel behind the bars. The Plum Point dikes (below Plum Point) were begun in 1883 and completed in 1884. These dikes were designed to contract the channel opposite and below Bullerton Towhead by causing a bar to form below Plum Point. In this, the dikes were successful but they also served to deflect the current against Craighead Point, three miles downstream, and increased the rate of bank caving at that point. The Keyes Point dikes were built in 1883-4. They were located above the head of Elmot Bar and were designed to close the channel between that bar and the left (east) bank of the river. These dikes were seriously damaged by floods occurring during their construction. The repair of this damage increased the amount of construction work by about 50 per cent. For a short time after their completion, the Keyes Point dikes gave some evidence of being successful. They caused heavy sedimentation in the channel below, and materially reduced the discharge through it. They did not, however, completely close off the channel. The dikes deteriorated rapidly and were subsequently abandoned when it was decided to close the channel by the Elmot Bar-Island No. 30 dikes. These dikes were built in 1889-90. They were two in number. One extended from near the foot of Elmot Bar to the head of Island No. 30. The other extended from Island No. 30 to the main river bank. These dikes were never more than partially successful. Breaks occurred in them almost immediately after their
construction thereby necessitating expensive repairs. Although the structures were higher and stronger than those previously built, they were inadequate and required heavy maintenance. No provision was made for the disposal of drift which accumulated along the dikes during flood and placed a heavy strain upon them. By 1893 these dikes were abandoned and in that year construction was begun on a solid brush and stone dam across the channel (see Plate LVIII). The dam was completed in 1894. The details of its construction are given later in this chapter. After its completion, the dam settled considerably and, during the high water of 1894, it broke in two places. During the following winter these breaks were repaired and the structure was somewhat strengthened. Subsequent breaks occurred, however, and the structure was again extensively repaired in 1899. In 1900, however, a 600-foot length of the dam failed entirely, probably as a result of undermining in the foundation. The widths of the breaks in the dam rapidly increased and by 1912 the channel behind Elmot Bar and Island No. 30 became the main river channel.

The types of dikes constructed in Plum Point Reach between 1881 and 1884 are illustrated on Plate LIX (a) and (b). The first design used (1881) consisted merely of a wire mesh screen hung vertically along a row of piles (see Fig. A, Plate LIX). It was hoped that this screen would collect drift to such an extent that it would become almost impermeable. This screen proved an utter failure, and its use was abandoned almost immediately. It was followed by the design shown by Fig. B, Plate LIX. This dike was essentially a screen made of wattle, supported by vertically driven piles (braced by inclined piles) and protected against undermining by a mattress sunk at the foot of the structure. This design was of some effect in inducing deposits below but suffered severely in high water. It lacked sufficient strength for effective use in the strong currents of the Lower Mississippi. Figures C, D, and E, Plate LIX show progressive development of this screen dike design which took place before 1884. These types all proved inadequate and the idea of a screen dike was abandoned and a permeable pile dike design was adopted (see Fig. F, Plate LIX). This dike consisted of rows of piles or pile clusters spaced \( \frac{7}{2} \) feet apart. The number of rows varied from three in shallow water to five in the deepest water. The upstream row of piles consisted of single piles or clusters of two or three piles, depending upon depth. The second row was composed of single piles or clusters of two piles. Rows downstream of the second row all consisted of single piles. Rows were spaced 20 feet apart and were well braced by longitudinal and transverse stringers. Additional bracing was provided by wire cable tension members. The structure was protected from undermining by a ballasted woven mattress which extended longitudinally throughout the length of the dike and covered the river bed and transversely from 10 feet upstream of the dike to 20 feet below it. This mattress was sunk first and the piles were driven through it. The banks at the shore end of the dikes were well revetted to prevent flanking by the current.

The solid brush and stone dam built at Elmot Bar in 1893-94 was about 3,000 feet long. It varied in height from about 3 feet to about 13.3 feet. This minimum height occurred where the dam crossed a bar which was considerably above low water. The elevation of the
dike was fixed at 16 feet above low water. From this we may deduce that the entire channel behind Elmot Bar was above low water at the time. The dam rested on a mattress foundation 100 feet wide and extending entirely across the channel. It was built of successive layers of brush mattress ballasted heavily with stone. Each mattress was 50 feet wide and was placed 8 feet farther upstream than the one below it. Foot mats were sunk in position where dangerous scour was likely to occur in the channel downstream of the dam.

For more detailed descriptions of the construction of the Plum Point Reach contraction works, the reader is referred to the Annual Reports of the Chief of Engineers for 1882, 1883, 1884, 1889, and 1894.

Discussions of the results of the Plum Point Reach contraction works are contained in Occasional Papers No. 41, U. S. Engineer School, by Maj. E. Eveleth Winslow, Corps of Engineers.

While the Plum Point Reach contraction works were in a measure successful for the time being, it should be noted that the only permanent improvement has been the closure of the channel behind Osceola Bar and Bullerton Towhead. Attempts to close the channel between Keyes Point and Elmot Bar failed completely and by 1912 the river had adopted it as the main channel. It is interesting to compare this work with the present projected improvement of this reach. These improvements are illustrated on Plate LXIV and are described later in this chapter.

The Lake Providence Reach regulating works (built in the period 1881-84) are illustrated on Plate LX. These works may be divided into four groups as follows: The Ducansby dikes; the Mayersville and Cottonwood dikes; the Baleshed dikes; and the Elton dikes. The Ducansby dikes were begun in 1882 and completed the following year. These dikes aggregated a length of about 20,000 feet. They were designed to improve navigation by closing off the river channel behind Ducansby Towhead. About 800 feet of dike were destroyed by the river during construction and in 1884 about 4,000 feet were destroyed. The destroyed dike was replaced but bank caving in the channel above permitted the river to outflank the entire Ducansby system. No further maintenance was attempted and the dikes were gradually destroyed. In 1912 the main river channel occupied the channel which the Ducansby dikes had been constructed to close.

The Mayersville and Cottonwood dikes were begun in 1882 and completed the following year. Their primary purpose was the closure of Mayersville chute and the closure of an incipient channel which was opening on the west bank of the river. By 1889 these dikes had been utterly destroyed. Mayersville chute eventually closed of its own accord. It is not apparent that the dikes had any effect on this closure.

The Baleshed dikes were built in 1882-84. They were designed to shut off the channel along the east bank of the river behind Baleshed Bar and Stack Island. For a few years the Baleshed dikes were effective as a channel improvement but they were gradually destroyed. In 1913 the main river channel passed over the site of these old works.

The Elton dikes were built in 1883 at the upper end of Elton Bend to close a small chute. The Elton dikes and the Stack Island dikes (at the lower end of the Baleshed dike system) concentrated the current
in the middle of the river between Elton Bar and Stack Island. Like the other dikes in the Lake Providence Reach, the Elton dikes soon deteriorated and ceased to have any effect on the river channel.

The types of dikes built at Lake Providence differed somewhat from those built in Plum Point Reach. Descriptions of these dikes can be found in the Annual Reports of the Chief of Engineers for the years 1882, 1883, and 1884. It is unnecessary to describe the Lake Providence Reach dikes in detail here. Suffice it to say that, like the Plum Point Reach dikes, they were not strong enough to be of permanent value.

The effects of this regulation in the Lake Providence Reach are discussed in the Annual Reports of the Chief of Engineers for the years 1883, 1884, and 1889.

The direct benefits of this regulation were temporary. It may, however, be noted that since the construction of these regulating works, the navigation conditions in this reach have been permanently improved. Whether this permanent improvement is an indirect result of this regulation can not be stated. The direct results of these works have disappeared long since.

Following the completion of the regulation programs in Plum Point and Lake Providence Reaches, the Commission confined its regulation program to the maintenance of existing works. The ultimate destruction of these works marked the virtual abandonment by the Mississippi River Commission of river regulation as a means of improvement. The adoption of a policy of navigation channel maintenance by means of hydraulic dredging by the Commission has been outlined in Chapter VII. Between 1890 and 1928 the construction of regulating works was confined to attempts to close secondary river channels behind islands, towheads, and bars. These attempts were in general unsuccessful. The first structure of this sort was the brush and stone dam constructed in 1893-94 across the Elmot Bar channel in Plum Point Reach. This dam has been described above. The only other works built prior to 1928 were the abattis dikes (developed between 1898 and 1907), and the sand dams built at Island No. 35 (about 195 miles below Cairo) in 1925.

The abattis dike (Plate XLVIII) was developed in an attempt to evolve a light structure capable of closing off chute and secondary channels to low-water flow. A condition of the development was that the dike should be so constructed that it would not form an obstruction in case subsequent channel changes placed it in the path of navigation. The abattis consisted of a series of timber frames bolted together by walings and stringers to form longitudinal sections from about 60 to about 250 feet in length. The abattis framework had two faces, a horizontal face and an inclined one. On the horizontal face a mat or grillage of poles was constructed to hold the ballast by which the structure was sunk. On the inclined face of the framework a screen of poles and brush was built. The abattis sections were built on barges, ballasted, anchored to deadmen or piles, launched, and sunk in place. When sunk in place, the sloping face of the abattis was inclined downstream and presented an obstruction to the current. Successive sections were placed with an overlap of
about 5 feet, each section being slightly downstream of the one preceding it. The top of the abattis dike was fixed roughly at the elevation of low water.

Between 1898 and 1927 about 26,500 feet of abattis dike were placed at seven localities in the main river channel. Two of these locations were in Plum Point Reach (Elmot Bar and Ashport Bar). The other locations were Point Pleasant (80 miles below Cairo); Cherokee Crossing (90 miles below Cairo); Island No. 14 (105 miles below Cairo); Walnut Bend (278 miles below Cairo); and Old Town Bend (323 miles below Cairo). At Cherokee Crossing only can the abattis be said to have been successful. At other locations, they either failed or were only partially successful. Experience indicated that the abattis was unsuitable for use in a stream of the size of the Lower Mississippi. The failures were generally due to underscour or flanking. For a detailed description of the abattis, the reader is referred to the Annual Reports of the Chief of Engineers for 1889 and 1901.

In 1925 two so-called sand dams were built at Island No. 35 (about 195 miles below Cairo). At this point (see Plate LXV) great difficulty has been experienced in maintaining a navigation channel. In 1925 the main river channel followed the west bank west of the island. The channel on the east side of Island No. 35 was a chute channel. The first and larger one of the two sand dams was designed to close this chute thus concentrating the entire low-water discharge in the main river channel. The second sand dam was designed to close a small secondary channel near the foot of the island. These sand dams were constructed by hydraulic methods and were covered with brush mattresses ballasted with stone. The first dam could not be built entirely across the chute channel due to the high current velocities through it after the partial completion of the work. This dam failed utterly. In 1928 the chute became the main river channel and at present little of the dam remains. The second dam was completed and is reported to be in a fair state of repair at present. Channel changes have, however, relieved it from attack.

For a discussion of these sand dams, the reader is referred to the Annual Report of the Chief of Engineers for 1926.

The development of contraction works on the Lower Mississippi before 1928 may be summarized as follows: Before the organization of the Mississippi River Commission in 1879, the relation between excessive widths and poor navigation conditions was appreciated. The Commission's first navigation improvement policy was based on contractions in reaches of excessive width. The contraction projects at Plum Point Reach and Lake Providence Reach were, however, failures and in 1896 the Commission turned to dredging as a means of navigation channel maintenance. Attempts were, however, made to develop a type of dike capable of closing chutes and secondary channels to low-water flow. Brush and stone dams, abattis, and sand dams were tried for this purpose but all proved to be failures. In 1928 channel dredging was the standard method of lower river navigation improvement.
With the adoption of the present flood control project (act of May 15, 1928), a new policy of navigation improvement came into being. Hydraulic dredging ceased to be considered the main means of channel maintenance on the Lower Mississippi. The present project calls for river regulation by means of contraction works and bank protection. It is not expected, however, that this program will do away with all need for channel dredging. Provision has been made for a limited amount of dredging to supplement the effect of the regulating works. Dredging will be needed especially during years of extreme low water and in localities where the need for improvement is intermittent and does not justify the expense of permanent works. Dredging of course remains the principal means of channel maintenance during the period of construction, but as the regulation program advances, the need for dredging is expected to diminish progressively. The present project calls for a navigation channel, 9 feet deep and 300 feet wide.

The present status of the regulation program is indicated on Plate LXI. It will be noted that the improvement is confined to that reach of the river within the Memphis Engineer District. For many years the reach most difficult for navigation has been that between Hickman, Ky., and Memphis, Tenn. The present project contemplates the complete regulation of this reach. Below Memphis the need for complete regulation is less pressing. The construction of contraction works below that point will be confined largely to the closure of objectionable chutes and secondary channels. For the time being at least, no attempt is being made to modify the curvature of the river bends below Memphis. The extension downstream of an elaborate regulation program will depend on the success of the program in the river above Memphis and the development of an economic need for such extension. Bank protection is of course an essential part of the regulation program, but as has been seen in Chapter VIII, bank protection is needed, and has been constructed at numerous locations throughout the lower river channel. The construction of bank protection is not confined to the limits of the regulation project.

The regulation program between Hickman and Memphis does not call for drastic or radical changes in the river channel. The river is being trained in a course of smooth, easy curves so designed that the displacement of the river channel is reduced to a minimum. Uniformity of channel width is of course essential. A regulated low-water channel width of 3,300 feet was determined upon after study. The average low-water channel width throughout the reach was about 3,200 feet before improvement. Wherever the channel width materially exceeded 3,300 feet, bad navigation conditions were likely to be encountered. It would seem, therefore, that the width selected for the improved channel will be effective in improving the channel without unduly constricting the river.

In designing a system of contraction works, it is of course necessary to study carefully the low-water profile throughout the reach under improvement. A study of profiles of the Hickman-Memphis Reach indicated that troublesome shoals exist during low water in reaches hav-
PERMEABLE DIKE CONSTRUCTION
New Madrid Bend, August 10, 1931
PERMEABLE PILE DIKE CONSTRUCTION
View showing mattress placed on sloping sand bank during low water. Dike is unfinished, shore end has not yet been constructed. New Madrid Bend. August 10, 1931.
ing steep slopes. Most of the channel dredging has in the past been done in those crossing reaches, which were characterized by steep slopes. As an example of this fact, the following instance may be cited: Between Hickman and Memphis the average low-water slope is approximately 0.39-foot per mile. In the stretch in the vicinity of Island No. 35 the slopes are approximately 0.63-foot per mile, or about 1.6 times the average slope. In addition, the river channel is very wide at this point. During the decade 1920-1930 about 23.4 per cent of all the channel dredging below Cairo was done in the vicinity of Island No. 35. The improvement of this stretch is discussed later in this chapter. It is not to be expected that a general uniformity of slopes between Hickman and Memphis will result from the construction of regulating works. A general modification of slope is neither necessary nor practicable at this time. Local modifications are, however, feasible and are contemplated by the project.

The program of regulation in the Hickman-Memphis Reach does not call for radical modification of channel lengths. Table XLI shows that the present project does not contemplate a material change in the low-water thalweg length through this reach.

**TABLE XLI**

Comparison of Low-water Thalweg Lengths, Hickman to Memphis

<table>
<thead>
<tr>
<th>Surveys of 1879-80</th>
<th>Surveys of 1912-13</th>
<th>Surveys of 1931</th>
<th>Proposed by Present Project</th>
</tr>
</thead>
<tbody>
<tr>
<td>216 miles</td>
<td>218 miles</td>
<td>212 miles</td>
<td>217 miles</td>
</tr>
</tbody>
</table>

In the plans for channel realignment the radii of the curves gradually decrease from the crossing toward the bight of the bend. It has been observed that, in the Hickman-Memphis Reach, serious caving occurs in bends having minimum radii of from 6,000 to 10,000 feet. In bends of minimum radii of from 10,000 to 15,000 feet, caving is moderate and navigation conditions are good. In curves of minimum radii greater than 15,000 feet, however, the thalweg has a tendency to wander in the channel with consequent channel deterioration. The radii of the adjusted channel curves have therefore been fixed at from 10,000 to 12,000 feet wherever practicable.

A number of factors enter into the determination of the proper elevation to which contraction works should be built. The regulation program depends upon the fixation of the bank lines in smooth curves. This fixation is attained, on the Lower Mississippi River, not only by protection of exposed bank lines but also by the building up of new bank lines along the front of and below the contraction works. The accretions which compose these new bank lines appear to build up best during stages between mid-bank and bank full. The dikes should therefore be high enough to be effective at these stages. On the other hand, the cost of their construction increases greatly with their height. Moreover, very high dikes are more liable to destruction from various causes than are lower structures. It is of course desirable that the
contraction works begin to become effective on a falling river at a stage at, or a little above, the stage when dredging would normally become necessary. After study, the elevations of the dike tops were fixed at the local equivalents of about a 20-foot stage on the Memphis gage. The dikes are usually located normal to the current or are inclined slightly downstream toward the channel. They are spaced from 500 to 5,000 feet apart depending upon the length of the dike, the current direction and velocity, the depth, and the location of the structure with respect to possible attacks. In a series of dikes each one should theoretically receive some part of the current attack. The interval between dikes should be sufficiently close to prevent attack upon the bank line by eddies created by the dikes themselves.

The life of a permeable pile dike, of the type used on the Lower Mississippi, is of course relatively short. This does not necessarily mean that maintenance costs will be unreasonably high. These dikes are designed to encourage accretions to the bank line. The operation of such a structure is therefore most successful when it is rapidly buried in the accretions induced by it. Where a dike is wholly successful in its operation therefore, maintenance costs will normally be confined to maintenance at its exposed end.

Two types of contraction work have been used in the present regulation program. They are the permeable pile dike and the permeable crib dike or retard. The retard was experimentally introduced on the Lower Mississippi in 1928 but proved unsuitable for the deep channel and high current velocities on the lower river. The retard consists of successive rows of whole trees laid with their butts upstream and fastened together by stringers; the whole forming a crib-like structure which was placed athwart the stream. The retard is anchored to deadmen or anchor piles and is normally sunk by the force of the current against the face of the structure. Ballasting the retard to make it sink is ordinarily unnecessary. The use of the retard has been abandoned on the lower river.

The permeable pile dike is illustrated on Plate LXII. This type of construction was not developed on the lower river but was adapted from the type of dike which had been successfully used in the reach between the mouths of the Missouri and Ohio Rivers.

The dike itself consists of parallel rows of piles driven in clusters of three. The number of rows varies from two to six, according to the depth of the water, the exposure to current attack, and the length of the dike. A line of heavy, round timber stringers is laid between rows at an elevation close to the top of the dike. The shore end of the structure is protected against flanking by the current by a strip of revetment 100 feet wide (normal to the bank line) and extending for a distance of from 130 to 300 feet along the bank line. Beyond the outer edge of this mattress, the dike is protected against scour by another mattress 80 to 100 feet in width which terminates in an "ell" pointing downstream at the channel end of the dike. The normal construction procedure is as follows: The bank at the shore end of the proposed dike is graded from its top down to the water line on a slope of from 1 on 2½ to 1 on 3. This graded bank is then covered as far down as the water line with a stone pavement from 130 to 300 feet in length (parallel
PERMEABLE PILE DIKE
Shore end of line of Pile Clusters.
Vicinity of Caruthersville, Missouri, December 21, 1930
to the bank line). At the lower edge of the paving, a woven mattress of lumber or brush is placed and ballasted with stone. The main line mattress is then woven, proceeding continuously from the outer edge of the shore mattress to the end of the dike. This mattress is cribbed along its upper and lower edges. The "ell" mattress is from 100 to 125 feet in width and from 150 to 200 feet in length. It is placed with approximately one-fifth of its length above the dike and one-fourth of its width inside the outer edge of the dike. After placement of the mattress, the driving of the piles begins. It is not necessary for the entire mattress to be in place before driving is commenced. In actual construction, piles are frequently driven at the shore end while mattress weaving is still in progress at the outer end of the proposed dike. At the shore end, a single line of piles is driven beginning with the tops of the piles at an elevation several feet below the top of the bank and sloping rapidly down to about the local equivalent of 20 feet on the Memphis gage (or as near that elevation as existing river stages permit). At the lower end of this line of single piles and extending outward to the outer end of the dike, the first line of pile clusters is driven. The alignment of this work is such that the line of stringers between the first and second rows of pile clusters may be extended up the bank along the line of single piles. The spacing between pile clusters in the line varies from 10 to 20 feet, according to the number of rows in the dike. The center line of this row of pile clusters is located about one-third of the width of the mattress below its upstream edge. Piles are driven to a penetration of from 20 to 30 feet and the tops of the clusters are fixed at an elevation corresponding to the local equivalent of approximately 20 feet on the Memphis gage. At the channel end of the dike, the grade of the line of pile clusters is depressed until it finally terminates in a cluster, the elevation of whose tops corresponds to the local equivalent of from 3 to 12 feet on the Memphis gage. The outer end of the dike is specially strengthened by a buttress of piles to resist the scour which is always severe at this end of the dike. Successive rows of pile clusters are then driven to the rear of the first row. Each succeeding row is driven about 5 feet from the rear of, and 2 feet lower in elevation than the next line upstream except that the last row is always driven as low as the stage of the water will permit. The pile clusters are drawn together and fastened at the tops with heavy strand wire. Where trail dikes are employed, they are similar in construction to the main line dike.

On Plate LXIII there is presented a map showing the method of improving a typical reach of the Lower Mississippi River. This map covers a channel section of 36.5 miles in length extending from just above Donaldson Point (53 miles below Cairo) to below Tiptonville, Tenn. (89.5 miles below Cairo). Works installed on January 1, 1932 are indicated by the heavy solid lines; proposed works are indicated by broken lines. The navigability of the channel through this section has been very much impaired by reason of excessive widths, and the diversion of part of the discharge to the rear of the several bars and islands. It is interesting to note the close relationship that exists between bank protection works and contraction works. The construction of these bank protection works should proceed simultaneously with dike construction if permanent improvement is expected. Examining this project in de-
tail, it will be noted that the river enters this reach with a fairly straight section of channel, about 7 or 8 miles in length; makes a sharp bend around the end of Donaldson Point; reverses its curvature and passes into a long swing around Watson Point into New Madrid Bend; makes reverses at Toney's Towhead and at Point Pleasant; and then enters a straight reach for several miles until, approaching Tiptonville, it again enters a long, gently curving bend. In the straight section of the river opposite and above Donaldson Point the channel will be reduced to a uniform width by placing three dikes on the left bank and five on the right bank. The concave bank of the channel around Donaldson Point is to be held by bank protection works, a part of which is already in place. Five dikes will be placed in the pocket on the right bank below Donaldson Point to produce a uniform shore line through that stretch. About 25,000 feet of bank protection will be placed on the right bank, beginning just below these dikes and extending to about the lower end of Morrison Towhead. It will be noted that several thousand feet of this bank protection is proposed along a line that extends through the lower center of the Morrison Towhead. It is expected that the shore line of the lower end of this towhead will recede by reason of caving banks, and the bank protection will be placed when the bank approaches the proposed project line. The chute behind Morrison Towhead will be closed by a dike extending from the mainland to the head of the towhead. The excessive width of the channel opposite Morrison Towhead will be reduced by placing three dikes on the left bank. An extensive bank protection program is proposed for the bend immediately above and below New Madrid. A large part of this is already in place. Four dikes are located on the left bank in the bend just above Toney's Towhead, and the shore line immediately below will be protected by bank protection. The right bank along the Point Pleasant front will be similarly protected and the channel width reduced by placing four dikes on the opposite convex bank. The comparatively straight section immediately below will be stabilized by placing four dikes on the left bank in front of Darnell's Towhead and bank protection along the opposite shore. As the channel enters the bend opposite Tiptonville, it will be held to a smooth course by bank protection beginning at the upper end of Bixby Towhead and extending intermittently along the left bank around the entire bend. Four dikes will be placed in the pocket in the bank opposite Bixby Towhead to produce a uniform channel width. In examining the above project, the reader will note that in several instances the dikes do not tie into the high-water bank line. It is not always necessary that they should do so. However, where they do not, three conditions must be satisfied, viz., the bank must be higher than the top elevation of the dike; it must be stable; and there must be no high-water chute or channel between the dikes and the high-water bank line which would subject the dike to danger of flanking.

The present channel improvement planned for Plum Point Reach forms an interesting contrast with the early contraction works built in that reach which have been described above. The present project is illustrated on Plate LXIV. Comparison of this plate with Plate LIX plainly shows the changes which have taken place in Plum Point Reach since the first improvement works were undertaken. The pres-
PERMEABLE PILE DIKE
Vicinity of Caruthersville, Missouri.
September 22, 1931

Detail view showing Piles driven through Board Mattress.

Detail view showing outer end of Dike.
PERMEABLE PILE DIKE
Vicinity of Caruthersville, Missouri.
September 22, 1931
ent project calls for the contraction of the excessive river width through this reach by means of six permeable dikes on the right bank below Forked Deer Island and eight on the left bank below Ashport. In addition, four dikes are located in the bend opposite Island No. 30 and five dikes are proposed on Driver Bar at the lower end of the reach.

We have seen in Chapter VII that the reach of Island No. 35 has in the past been characterized by serious navigation difficulties. Efforts to improve the navigation channel by sand dams have been discussed earlier in this chapter. The present plan for the improvement of this reach is illustrated on Plate LXV. This plan consists mainly in the closure of the channel west of Island No. 35 and the contraction of the excessive width of the main river channel. It will be noted construction has already (December 31, 1931) been partially completed.

**Costs.**

Discussion of cost is limited to the permeable pile dike since this is the only contraction work in present day use on the lower river.

The cost of permeable pile dikes varies widely according to the depth of water in which the dike is constructed. This variation is due not only to the relative difficulty of deep water operations but also to the fact that the number of rows in the dike varies directly with the depth. The unit cost per running foot of dike varies from about $14.00 to $53.00. During the fiscal years 1929, 1930, and 1931, the average unit cost approximated $32.00 per running foot of dike. The unit costs for the fiscal year 1931 were markedly lower than they had been for previous years. This was due to the period of depression through which the nation was passing; and also to the fact that the dikes built that year were built on locations favorable for low construction costs.
CHAPTER X.

NAVIGATION IMPROVEMENTS.

INTRODUCTION.

This chapter traces the progress of river navigation on the Lower Mississippi River and describes the improvements made for the benefit of navigation. Channel dredging, bank protection, and regulating works have been described in Chapters VII, VIII, and IX respectively and are therefore not discussed in detail here. Snagging, which is not covered elsewhere, is briefly discussed in this chapter. Since this chapter is confined to river navigation, navigation by seagoing ships on the lower river reaches will be mentioned briefly only. The term "snagging" used in this chapter applies to operations which have for their object the removal of "snags" from the river channel. A snag is a tree or branch fixed in the bed of the river and rising nearly, or quite to the surface of the water. "Snagging" does not consist in the complete removal of all snags from the river bed. It contemplates the removal of only such snags as are, or may become, a menace to river navigation.

The extent of the Mississippi navigation system is illustrated on Plate LXVI. It will be noted that this system is composed of three main trunk lines; the Mississippi proper, the Ohio, and the Missouri. These trunks are fed by numerous navigable tributaries. The Mississippi trunk line extends from the head of navigation at Minneapolis, Minn., to the Gulf of Mexico. The Ohio trunk extends from Pittsburgh, Pa., to the mouth of the river at Cairo. The Alleghany and Monongahela are navigable above Pittsburgh but must be considered as "feeders" to the Ohio. The present comprehensive project for the improvement of the Missouri for navigation extends to Sioux City, Iowa. Above that point navigation improvement is restricted to a limited amount of snagging and bank protection.

This chapter is concerned primarily with that stretch of the river between Cairo and Baton Rouge. Cairo is the natural junction point between upper and lower river navigation operations. The jurisdiction of the Mississippi River Commission over navigation does not extend above that point. Baton Rouge is the head of deep water navigation. Below that point a 35-foot channel is maintained by the Gulf of Mexico Division of the Engineer Department. The maintenance of the river navigation channel is therefore necessary only above Baton Rouge.

THE GROWTH OF NAVIGATION.

History does not record with accuracy the first entry by white men into the Mississippi River. We can not therefore establish the identity of the first white mariner to navigate the river. The existence of the Mississippi seems, however, to have been known or suspected by the Spaniards at a very early date. The so-called "Admiral's Map", said to have been engraved in 1507, shows the mouths of the Mississippi River beyond reasonable doubt. Many historians nevertheless agree
that the Spaniards had no real knowledge of the Mississippi River before 1519. There is some evidence that Pineda visited the mouths of the river in that year. More than 150 years were to elapse after De Soto’s death before true river navigation by white men began. La Salle’s expedition (1682) was made by Indian canoes and, while he traveled the greater length of the river, he can scarcely be credited with the inauguration of river navigation. The entry of Iberville into the river from the Gulf in March, 1699, may perhaps be properly assumed to mark the beginning of river navigation. Iberville was accompanied by his brother, Bienville, who spent the summer of 1699 exploring the delta of the Mississippi. About the middle of September of that year Bienville encountered an English vessel of 12 or 15 guns commanded by one Captain Barr. A claim to possession by England of the Mississippi River was subsequently made on the basis that Barr was the first mariner to navigate a ship on the waters of that river. This claim was not pressed, however, and nothing came of it.

As was stated in Chapter I, Iberville’s exploration definitely decided the location of the mouth of the Mississippi River. By 1699 therefore the existence of the great Mississippi navigation trunk line was known to the French. The importance of the Ohio River as a transportation route was also generally appreciated at an early date; and the Upper Mississippi, the Missouri, and the lower river tributaries were visited by Frenchmen.

For early river transportation, the French trappers and traders depended largely on Indian canoes. Dugout canoes sometimes attained lengths as great as 30 feet. The so-called "bull boats" were also used for early river travel. These were made by stretching skins over wooden frames. They were from 20 to 30 feet long and had modeled bows. With the English settlement of the Ohio River Valley, however, real river commerce began. Craft capable of carrying heavy cargoes began to replace the canoes and bull boats which were inadequate for the needs of a growing commerce. The Ohio River pioneers at first used rafts. Settlers seeking new homes made rafts of logs and floated down the Ohio to new country. Upon arriving at their destinations these rafts were broken up and the logs were used for building dwellings. Rafts were also used on the Mississippi River but were soon superseded by flat boats. The flat boat was constructed for downstream navigation only. It could not be propelled upstream. Upon arrival at its destination, its cargo was unloaded, the boat was broken up, and its timbers were sold. The Ohio and Kentucky traders floated down the river as far as New Orleans; sold their produce and flat boats; and then returned to their homes overland. There was, however, a constantly increasing need for craft capable of upstream navigation. This need was met by the evolution of the keel boat. Keel boats were shallow draft craft about 50 feet long; from 10 to 15 feet wide; and having modeled bows and sterns. They were fitted with walking planks along their gunwales and were propelled by poling in shallow water. When practicable, sails were used.

Trade between the Illinois and Ohio River country and the Lower Mississippi soon reached important proportions. The small, unwieldy keel boat soon proved inadequate for this growing trade, and the invention of the steamboat pointed the way to new methods of river
navigation. The first steamboat to navigate the Mississippi River system was built in 1811 on the Ohio River by Robert Fulton. Fulton was associated with Mr. Robert R. Livingston (who, as Minister to France, had negotiated with the French Government for the Louisiana Purchase) and with Nicholas Roosevelt. These men attempted to secure a monopoly on the navigation of the Mississippi River system by steamboats. The venture was not successful. Livingston did succeed in obtaining from Louisiana a monopoly on steamboat navigation of the river within its boundaries but this monopoly does not seem to have been observed by later steamboat builders. Fulton built two steamboats which reached New Orleans, but which were unable to go upstream against the river current. Between 1813 and 1815 two more steamboats were built by a man named French. The second of these boats (the Enterprize) was the first steamboat to ascend the river. The first successful steamboat, and the forerunner of the later packet boats, was built by Henry N. Shreve. His boat, the Washington, constructed in 1817, was successfully used in general freight and passenger business between New Orleans and Louisville. Shreve's steamboats were widely copied and, by 1834, there were about 230 packets in Mississippi River service. By 1849 there were about 1,000 steamboats in active service, approximating 250,000 tons. The packet boat era reached its culmination in the decade prior to the Civil War. During that war, however, many of the packets were sunk, burned, or were placed in military service. River trade was effectively broken up. After 1865 there was a brief revival of activity for the packet lines but the growth of the railroads diverted more and more business from the river traffic. The days of the steamboat and commercial river navigation seemed to be over.

The packet boat was dead, but, the decrease of commercial river navigation was only apparent. It is difficult to fix a date for the beginning of towboat navigation. The transition from packet to towboat was so gradual that for many years it was not apparent. Even in the heyday of the packet boat, towboats were used to some extent. Packets frequently towed fuel scows for the time required to take fuel aboard. The practice of transporting freight by tow began at an early date, and with the decline of the packets, the towboat came into its own. The towboat with its tow is well adapted for the cheap transportation of heavy, bulky commodities. With the beginning of the present century there began a gradual but progressive increase in the tonnage transported by this method. At present, the amount of freight moved annually on the Mississippi River is more than double the amount moved during any year in the 19th Century.

From the exigencies of the World War arose a powerful impetus in the revival of Mississippi River transportation. Created originally as a war measure to relieve transportation congestion, the Federal Barge Line was, in 1924, reorganized as the Inland Waterways Corporation, a Government-owned corporation which is at present (December 31, 1931) in operation on the Mississippi River. Privately owned commercial lines are also in successful operation. Among these lines may be mentioned the Mississippi Valley Barge Line, a recently organized concern which is engaged in lower river traffic.
The types of modern towboats are illustrated by the following examples: The *St. Louis* (of the Inland Waterways Corporation) is 200 feet long; 40 feet wide; has a draft of from 6 to 6 1/2 feet; and a gross tonnage of 757 tons. It is a tunnel boat, that is, propelled by twin screws set in tunnels in the hull. It is powered by 1,800 I. H. P. engines. The self-propelled barge *Birmingham* (of the Inland Waterways Corporation) is used for fast river freight. The *Birmingham's* gross tonnage is 1,587 tons. It is 280.1 feet long; has a 49.1-foot beam; and its draft loaded is about 8 feet. It is propelled by 800 I. H. P. engines which drive twin screws operating in tunnels in its hull. The towboat *Herbert Hoover* (also of the Inland Waterways Corporation) is the most powerful shallow water towboat in the world to be operated by Diesel engines. The *Hoover* is 226.4 feet long over all. Its two Diesel motors develop a total of 2,932 I. H. P. The towboat *Sprague* (owned by the Standard Oil Company of Louisiana) is reputed to be the largest stern-wheel paddle boat in the world. The *Sprague* is 276 feet long and has a 61-foot beam. Its gross tonnage is 1,479 tons and its propelling engines total 1,600 I. H. P.

The term “tow” as here used must not be misunderstood. On the Mississippi River, steamboats do not actually “tow” barges as the term is understood in deep sea navigation. In reality, the towboat pushes its tow. The number of barges making up a tow varies of course with the size of the barge and of the towboat. With the increase of towboat power and the increasing ease of river navigation, there has been a corresponding increase in size of tows.

The Mississippi Valley Barge Line has developed a type of barge with wedge shaped bows and sterns. These barges are smaller than the ones in ordinary use on the river but make up in convenient tows as the bows and sterns interlock. They are designed for ease and speed in towing, and their small size permits unit shipments.

The extreme size reached by downstream tows may be illustrated from the following examples: In 1904 the *Sprague* passed Memphis with a tow about 880 feet long and 312 feet wide. This tow consisted of 56 coal barges and 2 fuel barges. The tonnage was 53,200 tons. The *St. Louis*, of the Inland Waterways Corporation, arrived at New Orleans in December, 1931 with a shipment of cotton believed by many to be the greatest ever made on the Mississippi River. Twenty-five thousand bales were placed aboard five barges in Memphis, three barges carrying thirty-two hundred bales were added at Vicksburg, bringing the total cotton shipment up to twenty-eight thousand and two hundred bales. In addition, the *St. Louis* carried three other barges containing grain and merchandise. Upstream tows are of course smaller than downstream tows.

**CHANNEL IMPROVEMENT BEFORE THE ORGANIZATION OF THE MISSISSIPPI RIVER COMMISSION.**

In an unimproved alluvial river (such as the Lower Mississippi once was) the low-water depths over the bars and crossings limit the draft of the vessels which can navigate the river during low water. During the early days of Mississippi River steamboating, the packets
were built to navigate an unimproved river. With the growth of steam-
boating, however, there was an increasing demand for aids to naviga-
tion. The first evidence of Federal interest in the Mississippi River
was the passage of a law in 1820 appropriating the sum of $5,000 for a
survey of the Ohio and Mississippi Rivers with a view to navigation
improvement. As has been stated before, this survey was made in
1821 by Captain H. Young assisted by Captain W. T. Poussin and
Lieut. S. Tuttle, all of the Engineer Corps of the Army. In 1821 Brig.
Gen. Simon Bernard and Major Jos. G. Totten (both Engineer Officers)
detailed to make a thorough investigation of the Mississippi and
Ohio Rivers with a view towards navigation improvement. Their study
of the Mississippi River extended from St. Louis to the Passes. Their
report was submitted in 1822 and was published in House Document
Number 35, 17th Congress, 2nd Session. As has been stated in Chap-
ter I, Bernard and Totten devoted much attention to the removal of
snags. Their report was followed by the act of Congress of May 24, 1824
which appropriated $75,000 for snagging in the Mississippi River (below
the mouth of the Missouri River) and in the Ohio River. This marked
the beginning of snagging operations on the lower river. Snagging
operations were begun on the Red River in 1828 and on the Arkansas
in 1832. The work was carried on under the Engineer Department
and, once begun, has been continued until the present. Between 1824
and 1879 a total of approximately $3,093,000 was expended for snag-
ging operations in the Mississippi, Ohio, Missouri, and Arkansas Rivers
under various appropriations calling for the joint improvement of the
Mississippi and one or more of the tributaries above mentioned.

Special snag boats were constructed by the Engineer Department
to carry on snagging operations. These boats are equipped with heavy
steam operated tackle for raising snags and have the necessary facil-
ities for cutting them up. Massive roots which can not be cut up
are dropped in deep holes or outside the channel. Hydraulic jets
and dynamite are occasionally used to dispose of very large unwieldy
snags. Snag boats have proved useful for miscellaneous work other
than snagging. They have been employed for the removal of wrecks,
old docks, old revetment, etc. In the early days of lower river improve-
ment large snag boats were built. Such was the snag boat Macomb
constructed in 1874. This boat is described in the Annual Report of
the Chief of Engineers for 1894. Very large snag boats have, however,
not proved necessary on the lower river. The great bulk of snagging
operations now being carried on below Cairo is done with the standard
snag boats of the Engineer Department.

The improvement of the mouths of the river for seagoing naviga-
tion was undertaken at an early date. Subsequent to the founding of
New Orleans several water routes to the city were used. One approach,
much used in early times, lay through Mississippi Sound, the Rigolets,
and Lake Pontchartrain. Another lay through Lake Borgne and the
Lake Borgne Canal. Other approaches were to be found west of the
Mississippi. With the improvement of the Passes, however, these routes
fell into disuse.

As has been stated elsewhere, dredging in Southwest Pass was un-
dertaken by Captain Talcott in 1839. In 1852 a study was made by a
Board of Engineer Officers to determine the best methods of establish-
ing a navigable channel through the Passes. The report of this board was discussed in Chapter VII. As a result of this report, a certain amount of dredging by the New Orleans Towboat Association was undertaken in 1853 in Southwest Pass. In the year 1856 Craig and Rightor undertook the improvement of Southwest Pass and Pass a l'Outre by means of jetties, and later by dredging. Craig and Rightor's operations were unsuccessful. They are described in Chapters VII and IX. In 1867 the dredges Essayons and McAlester were employed in the Passes (see Chapter VII). In 1875 Mr. James B. Eads and his associates undertook the construction of jetties in the South Pass. As is well known, the Eads jetties were successful, and with certain modifications which experience has necessitated, they have effectually solved the problem of channel maintenance at the Passes. The Southwest Pass was later improved by jetties, the work being done by the Engineer Department. These works suffice, with moderate assistance by dredging, to keep the port of New Orleans open to seagoing navigation. These jetties are discussed in Chapter IX.

The improvement of river harbors was not undertaken until a fairly late date. Work by the Engineer Department was begun in New Orleans Harbor in 1878. In the bend below the foot of Canal Street the piling supporting the wharves retarded the current and caused heavy deposits of sediment during high stages. During succeeding low stages these high water deposits frequently slid toward the middle of the channel causing damage to the wharves. Attempts were made to prevent this sliding by mattresses designed to prevent erosion. These mattresses were light and poorly designed. Their operation was unsuccessful and the plan was ultimately abandoned. Between 1878 and 1882 work was undertaken for the improvement of Memphis, Vicksburg, Natchez, and Vidalia Harbors (Vidalia is on the west bank of the river directly opposite Natchez). The work at Memphis consisted of the placement of bank protection along the harbor front above and below the mouth of Wolf River. Some experimental work was also done with screen dikes to divert the current. About 3,900 linear feet of effective bank protection was placed between 1878 and 1882. The screen dikes were not successful. Bank protection works similar to those at Memphis were undertaken in 1878 to stop the caving in the upper side of Delta Point (opposite Vicksburg). This caving was causing a recession of the channel away from the harbor entrance at Vicksburg. By 1882 about 4,800 linear feet of bank had been protected. The results were considered satisfactory. Improvement of the Natchez and Vidalia Harbors was begun in 1880 and consisted of bank protection in Giles and Marengo Bends a few miles above Natchez. The work at Giles Bend was done to prevent a possible cut-off. Willow mattress construction similar to that at Vicksburg and screen dikes were used. This work was not successful. The bank protection which had been placed was destroyed by continued caving. Work was discontinued in 1882 not to be resumed until 1892.

Attempts to improve the channel itself were made at a comparatively early date. The Shreves Cut-off (see Plate IX) and the Racourci Cut-off (see Plate XVIII) were artificial cut-offs made in 1831 and 1848 respectively in an attempt to improve navigation in the
main river at the mouth of Red River and in the lower reaches of Red River itself. These cut-offs are described in Chapter II. As navigation improvements, they must be classed as failures. These cut-offs demonstrate the fallacy of the theory that navigation is always improved by channel shortening.

As has been stated before, the Civil War was a heavy blow to Lower Mississippi River navigation. Despite this fact, however, navigation soon again engaged national attention. The act of June 23, 1874 appropriated funds for the investigation of certain transportation routes from the interior to the seaboard of the United States. The selection of the routes to be investigated rested with a Senatorial Committee on "Transportation Routes to the Seaboard". The Mississippi River was naturally one of the routes chosen for investigation. To Major C. R. Suter, Corps of Engineers, was intrusted the duty of making the report on the lower river. Suter made a reconnaissance and a map of the Lower Mississippi. His map is described in Chapter V. Suter reported that he considered a navigable depth of 10 feet at all stages feasible of attainment. He, however, recommended that improvements be made gradually and that no attempt be made violently to disturb the regimen of the river. Otherwise he considered that the cost of the project would be prohibitory. As a first step, Suter recommended an increase in the low-water navigation from 4½ feet to 6 feet. The reader is referred to the Annual Reports of the Chief of Engineers for the years 1875 and 1878 for Major Suter's original report and for a supplemental one made by him.

Interest in navigation persisted and in 1878 a Board of Engineer Officers was appointed to report on a plan for the improvement of low-water navigation on the Mississippi. Later instructions were also given the board to consider the effects of a permanent system of levees below the mouth of the Ohio, not only upon low-water navigation but also upon high-water navigation. This board was composed of the following engineer officers: Col. J. G. Barnard, Col. Z. B. Tower, Lt. Col. H. G. Wright, Maj. C. B. Comstock, and Maj. C. R. Suter. The report of this board is found in the Annual Report of the Chief of Engineers for 1879. The board recommended the contraction of the river channel to a width of about 3,500 feet by means of light, permeable works. It believed that this contraction would be sufficient to produce adequate navigation depths in general. In case the bed of the river proved to be too hard in some localities to be scoured away by this means, the board considered that dredges would probably be needed. It was not thought proper, however, to embark upon an ambitious program of navigation improvement without sufficient data or experience. It was recommended, therefore, that Plum Point Reach be selected as a site for experiment. This reach was one of the worst on the lower river and would, the board felt, be a severe test of the improvement methods proposed. As to levees, the board concluded that the levee lines were an aid to navigation and commerce during high water, but that they had no influence upon low-water navigation. To summarize; this board considered the bank caving the greatest obstacle to navigation improvement, it recommended the improvement of Plum Point Reach by experimental works designed to reduce the low-water channel width to 3,500 feet, and by bank protection.
NAVIGATION IMPROVEMENTS SINCE THE ORGANIZATION OF THE MISSISSIPPI RIVER COMMISSION.

The organization of the Mississippi River Commission in 1879 marks the opening of a new era in navigation improvement. The act creating this Commission directed the preparation of a plan to correct, permanently locate, and deepen the river channel; to protect the river banks; to improve navigation; to prevent destructive floods; and to facilitate commerce and the postal service. On February 17, 1880 the Commission submitted its first report. In studying this report, one is struck by the similarity between it and that of the Board of Engineers of 1878. The Commission stated its belief that the levee system tended to deepen and enlarge the river during floods, and that its repair and maintenance would hasten channel improvement. The Commission did not, however, consider the levee system a necessary adjunct to a navigation improvement project. Like the Board of 1878, the Commission recommended a program of regularization and bank protection for the navigation improvement of the lower river. This program was to be supplemented by dredging where necessary, but reliance was placed primarily in light contraction works which were to limit the low-water channel to 3,000 feet (instead of 3,500 feet as recommended by the Board of 1878). It was hoped that these measures would provide a channel depth of 10 feet. The Commission further recommended the locations at which initial improvement works were to be undertaken, namely, New Madrid Bend, Plum Point Reach, Memphis, Helena, Choctaw Bend, and Lake Providence. The appropriation of funds for these works placed the stamp of Congressional approval on these plans and work was begun.

The history of these early contraction and revetment works is contained in Chapters VIII and IX. Suffice it to say here that they were too light and proved inadequate for their purpose. In 1891 the Commission undertook the study of dredges for channel improvement. A program of channel improvement by dredging gradually developed (see Chapter VII). Experience has shown, however, that a program based solely on dredging is too expensive for practical use. The present improvement project calls for improvement by regulating works supplemented when necessary by dredging (see Chapter IX). The regulating works now under construction are permeable but are of much stronger construction than those originally proposed by the Commission.

Two snag boats stationed at St. Louis made periodic trips for many years to snag the main river below St. Louis. Snagging on the tributaries of the lower river was done by small snag boats constructed for the purpose. The reorganization of the Engineer Department in 1929 was followed by the removal of the headquarters of the Mississippi River Commission from St. Louis to Vicksburg and the separation of the Mississippi River Engineer Operations into two distinct divisions (the upper and lower river divisions, respectively). This was naturally followed by a separation of the snagging operations of the two divisions.

This localization of snagging operations has been beneficial on the lower river. At present nine snag boats are available for snagging the lower river and tributaries. Small snag boats appear to be more eco-
nomical than the larger craft and are adequate for use on the main stream. Each lower river district has sufficient snag boats both for the tributaries and the main stream within district limits. The following table lists the snagging operations between 1868 and 1930.

**TABLE XLII**

Snagging Operations Between March 28, 1868 and June 30, 1930

Note: This table includes work done between the mouth of the Missouri River and New Orleans, La., and including Atchafalaya and Ohio Rivers

<table>
<thead>
<tr>
<th>Fiscal Year</th>
<th>Snags Destroyed</th>
<th>Trees Cut</th>
<th>Drift Heaps Destroyed</th>
<th>Wrecks Destroyed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mar. 28,1868 to June 30, 1871</td>
<td>3,271</td>
<td>37,438</td>
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<tr>
<td>1872</td>
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<td>1,365</td>
<td>1,713</td>
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<td>1874</td>
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<td>515</td>
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<td>1876</td>
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<td>1,589</td>
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<td>1880</td>
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<td>660</td>
<td>19</td>
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<td>1881</td>
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<td>6</td>
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<td>2,861</td>
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<td>3,389</td>
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<tr>
<td>1899</td>
<td>3,300</td>
<td>30,695</td>
<td>34</td>
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</table>
**TABLE XLII—Continued.**

Snagging Operations Between March 28, 1868 and June 30, 1930

Note: This table includes work done between the mouth of the Missouri River and New Orleans, La., and including Atchafalaya and Ohio Rivers

<table>
<thead>
<tr>
<th>Fiscal Year</th>
<th>Snags Destroyed</th>
<th>Trees Cut</th>
<th>Drift Heaps Destroyed</th>
<th>Wrecks Destroyed</th>
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</thead>
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As to levee construction under the Mississippi River Commission, the reader is referred to Chapter VI. Suffice it to say here that Federal expenditures for levee construction were at first limited to that construction which could be justified as an aid to navigation. This policy continued until the passage of the first flood control act on March 1, 1917.

The reader will recall that the Mississippi River Commission at first considered a 10-foot channel depth as feasible. The difficulty of this task soon became apparent. With the adoption of dredging as the main method of channel improvement, a definite directive became necessary. This was provided by the act of June 3, 1896 which called
for a channel below Cairo, 250 feet wide by 9 feet deep. The act of January 21, 1927 amended this project by providing for a 35-foot channel for seagoing vessels below Baton Rouge. The maintenance of this deep channel was placed under the Gulf of Mexico Engineer Division which was already charged with navigation maintenance below New Orleans. This restriction of Commission activities was apparent rather than real since the channel below Baton Rouge had required little active improvement work for navigation by river craft.

The construction of a deep water navigation channel from the Great Lakes to the Gulf is a proposition which has always appealed to the popular mind. In the early part of the present century this interest culminated in the passage by Congress of the acts of June 13, 1902 and March 2, 1907. These laws directed studies and reports on the practicability of the construction of 14-foot waterways between Chicago and St. Louis and between St. Louis and New Orleans respectively. The reports made pursuant to these acts were published in House Document No. 263, 59th Congress, 1st Session, and House Document No. 50, 61st Congress, 1st Session. The board on the river below St. Louis was composed of the following members: Col. W. H. Bixby, Corps of Engineers, President of the Mississippi River Commission; Messrs. H. B. Richardson and H. P. Ritter, Members of the Commission; Lieut. Col. C. McD. Townsend; and Lieut. Col. J. G. Warren. The board report which was rendered March 20, 1909, stated that the minimum width of a 14-foot channel should be 500 feet. The most practical means of obtaining this depth was believed to be a combination of dredging and contraction works which it was estimated would cost $128,600,000 for construction and $6,500,000 per year for maintenance (for river below St. Louis only). The board stated that 14 feet was considered deeper than was needed for economic river transportation and less than was necessary for economic lake or ocean going ships. The navigation project was not recommended by the Engineer Department, nor was it adopted by Congress.

The existing project was, however, changed by the present flood control act (May 15, 1928) which adopted the flood control plan set forth in House Document No. 90, 70th Congress, 1st Session. This plan called for a channel with a width of 300 feet and a depth of 9 feet. Such is the present navigation project. The additional channel width called for by this plan is needed for the wide tows now plying the river.

The maintenance of channel works such as navigation lights and buoys is a function of the Lighthouse Bureau which now maintains a system of beacons on the lower river. However, pending the completion of the lower river navigation project, channel buoys are placed by the local engineer districts which also provide most of the buoys.

Although the navigation of the Upper Mississippi is outside the confines of this chapter, it is of interest to mention the fact that a system of headwater reservoirs has been constructed on the upper river as a navigation improvement. As originally authorized by the act of June 14, 1880, the project called for the construction of forty-one reservoirs in Wisconsin and Minnesota. Only six reservoirs were actually constructed, however. These headwater reservoirs have of course no appreciable effect upon the lower river.
The progress to the present (December 31, 1931) of navigation improvement on the lower Mississippi may be summarized thus: In its first report, the Mississippi River Commission recommended a plan for improvement by regulation and bank protection. This recommendation was basically correct. The first improvement works were, however, too lightly constructed for their purpose and funds were insufficient for more elaborate structures. Dredging therefore became the adopted method of channel improvement. Dredging was expensive, however, and proved to be an uncertain method of channel maintenance on the lower river. The present project therefore calls for river regulation by training and contraction works and by bank protection. These works are to be supplemented by dredging when necessary.
CHAPTER XI.
FLOODS AND FLOOD CONTROL IN THE LOWER MISSISSIPPI VALLEY.

INTRODUCTION.

This chapter discusses flood control primarily in its application to the Lower Mississippi River Valley. Different methods of flood control and the theories underlying them are described only insofar as is necessary to make the text clear.

In the preparation of a comprehensive plan for the control of floods on any river, the following principles are so self-evident as to be axiomatic:

a. Soundness.—The plan must be based on sound principles of river engineering. The engineers who prepare a flood control plan must have not only a broad general knowledge of river hydraulics but also actual experience in the construction of flood control works.

b. Economy.—The plan must be economically sound; in other words, its construction and maintenance costs must be reasonable as compared with the benefits derived from it.

c. Maximum benefits.—Consistent with the principles of soundness and economy, the plan must be devised to secure maximum benefits to the protected areas.

d. Simplicity.—The plan should be simple and, as near as possible, automatic in its operation.

The principles of soundness and economy are vitally important. Plans which do not meet their requirements are unsuited for use and must be discarded. The principles of maximum benefits and simplicity are of less importance. They are valuable principally in determining the comparative merits of two or more acceptable plans for the solution of any given problem.

The preparation of a flood control project involves the following steps:

a. Study of the river system and collection of detailed data, especially that relating to the maximum floods of record.

b. Selection of method or combination of methods.

c. Formulation of plan of procedure.

Step "a" includes not only a study of the river system itself but also a thorough determination of the economic considerations involved. Step "b" involves a broad knowledge of and experience in flood engineering. The methods chosen must be those best calculated to meet the conditions developed in step "a". Step "c" involves a determination of the sequence by which the plan is put into effect. Other things being equal, the most vital protection must be provided first.
The principal methods usually advocated for the control of floods on the Lower Mississippi are briefly described below. No attempt has been made to describe all methods. For discussions of various flood control measures, the reader is referred to the following texts:

River and Harbor Construction—by Col. C. McD. Townsend.

Broadly speaking, flood control methods fall into two groups, flood prevention and flood protection. Flood prevention measures are designed to prevent flood stages in the watercourses traversing the protected area. Flood protection measures permit the passage of flood stages through the protected areas but prevent damage from them or minimize their effects. There is no sharp line of demarcation between flood prevention and flood protection; nor is a comprehensive plan necessarily limited to the employment of one of these classes alone. The plan is usually a combination either of prevention or protection methods, and sometimes of both.

Reforestation has in the past been frequently advanced as a natural means of Mississippi River flood control. The proponents of this method maintained that deforestation in the Mississippi watershed destroyed the natural carpet of humus which formerly covered the forested areas, thereby causing increased erosion; reducing the absorptive power of the ground; and increasing the amount and rapidity of surface run-off. They also claimed that the volume of spring freshets was accentuated by the increased rate of melting of snow and ice in deforested areas. They stated that deforestation seriously reduced the amount of water returned from the earth to the atmosphere by plant transpiration. Whatever the merit of these theories may be, reforestation offends the basic principle of "economy" and can not therefore be accepted as a basic method of flood control for the Lower Mississippi. This is apparent when one considers the cost of reforestation necessary to return the watershed to its primitive condition. The partial reforestation necessary to reproduce conditions even as late as 1848 would require an expenditure out of all proportion to the value of the areas protected. However, the return of the Mississippi River watershed by reforestation to a condition similar to that of 1848 would not eliminate severe floods. It has been shown in Chapter III that the lower river had experienced severe floods before that time. In fact, the passage of the swamp acts is evidence of the need for flood control at the time.

The term "channel relocation" may be employed to cover a group of methods sometimes advocated for Mississippi River improvement. These methods may be divided into two groups; those which divert the flow of tributary streams to the sea through artificial or partially artificial channels, and those designed to carry the flow of the Lower Mississippi itself to the sea in an artificial channel. The first group seeks to reduce the flood discharge of the main stream while the second group aims to accommodate that discharge in an improved artificial channel.
"Channel relocation" schemes fail to fulfill the first principle governing flood control plans. They are technically unsound. The advocates of tributary diversions fail to realize that the capacities of alluvial stream channels are determined by the volume of their "within bank" flows. The diversion of tributaries would not only cause channel deterioration on the main river channel but would also seriously disturb the channels into which they were diverted. It would doubtless be necessary to provide flood protection on these partially artificial streams. Although modified, the flood menace would still exist. The advocates of enormous artificial lower river channels neglect to appreciate the enormous capacity of the river for erosion and silting. The task of maintaining an artificial channel would be enormously difficult if not impossible. Neglecting, however, all question of technical soundness, the cost of "channel relocation" renders it utterly uneconomic for the control of the Lower Mississippi.

Channel improvement must be clearly differentiated from those channel relocation schemes which involve the partial or complete abandonment of the natural channel of the river and the diversion of its flow into artificial channels. Channel improvement does not involve the abandonment of the natural channel of the river, but seeks merely to improve the natural channel. Channel improvement includes three distinct methods; enlargement of the discharge capacity of the channel between banks, channel shortening, and clearing the channel. The enlargement of the channel discharge capacity between banks would involve a very extensive dredging program. An alluvial river excavates a channel suited to its "normal" discharges. Any enlargement beyond that point is not maintained by the river and channel deterioration follows. Such a method of flood control would therefore require constant maintenance at a great cost.

Proposals for improvement by channel shortening usually take the form of plans for wholesale artificial cut-offs within extended reaches of the river channel. Cut-offs have been discussed at length in Chapter II. The effects of wholesale cut-off programs are covered there.

Channel clearing is solely an adjunct to other flood control measures. Clearing is done on the battures of the flood channel and on river islands. Such operations increase the velocity of flow over the cleared areas from about two to three times. Clearing is, however, a constant and expensive maintenance task. Its expense renders its general use impracticable.

Reservoirs provide a method of flood control which has received wide and serious consideration. From a strictly engineering standpoint, the reservoir is perhaps the soundest method for flood control. Reservoirs may be constructed and maintained with greater certainty and safety than any other flood work. The location of reservoir storage is, however, governed by the two following principles which are not always appreciated:

a. Reservoir storage becomes progressively less valuable as its distance from the protected area is increased.

b. Reservoir storage at any point is valuable up to the amount of the discharge provided for at that point by the flood control plan. Storage above that discharge has a very rapidly decreasing value.
The truth of principle "a" is apparent. A reservoir controls the discharge of the entire drainage system above it. As the reservoir site is moved upstream, the area controlled by it decreases. Tributaries are progressively uncovered and separate reservoirs must be built to control them. Thus a given amount of protection can usually be afforded more economically by "downstream" reservoirs than by storage higher up. Aside from reasons of economy, however, the reservoir should be located as closely as possible to the protected area. The shorter the time of stage transmission from the reservoir site to the protected area, the more valuable the reservoir storage is. The inaccuracies of long range stage prediction have been discussed in Chapter III. The farther the reservoir is from the protected area, the more difficult it is to utilize its storage to best advantage.

Principle "b" is also self-evident. A properly conceived flood control plan is based upon assumed maximum discharges for the various streams of the system under improvement. Reservoir storage at any given point is valuable only to the point where it can contain the assumed maximum discharge past the reservoir site. Additional storage beyond this point may at times be useful but this use is only occasional and its value decreases rapidly.

There is a general belief among laymen that flood control reservoirs are also valuable for power and irrigation. This is, however, a fallacy in the general case. Power and irrigation reservoirs are of greatest value when full, while flood reservoirs are of greatest value when empty. These conflicting characteristics usually prevent the economic use of the same reservoir for both purposes. The use of reservoirs for flood control is also likely to conflict with their use for navigation. Since flood reservoirs are of greatest value when empty, sound practice demands that they be kept empty as long as possible in order that their entire capacity may be available when needed. On the other hand, a reservoir designed to control low-water navigation depths in the channel below is most valuable when full. Thus, such a reservoir should be filled as soon as possible during periods of high water in order that its complete storage capacity may be available for the low-water season which follows. The effective use of one reservoir for both purposes is therefore likely to be impracticable.

Flood reservoirs may be either storage reservoirs or detention reservoirs. In the storage reservoirs, all water up to the limit of the reservoir capacity can be held or released as desired through controlled outlets. Storage reservoirs constructed for flood control differ from those intended for general water supply mainly in that the flood reservoirs are provided with facilities for rapid emptying. Detention reservoirs have outlets which are permanently open and are of sufficient capacity for ordinary discharges. The reservoir becomes operative only when the inflow exceeds the capacity of the outlet. Reservoir storage then follows. The size of the reservoir outlet is of course limited by the discharge which can be safely carried in the channel below. Natural bodies of water possess to some extent the properties of reservoirs. Lakes act as reservoirs to regulate the flow of streams flowing from them. The regulating effect of the Great Lakes on the discharge of the St. Lawrence River is well known. There now exist on the Lower Mississippi River a series of reservoirs partly natural and partly the result of levee con-
struction. These are the backwater areas at the mouths of the St. Francis, the White and Arkansas, the Yazoo, and the Red Rivers. They have been discussed in Chapters II and III. Reservoirs have been frequently advocated for the control of the lower river. This will be discussed later in this chapter.

The terms "outlet" and "floodway" are easily confused. An outlet is a channel by which water is diverted from the main river channel and delivered to the sea usually by a shorter route. A floodway, on the other hand, is a channel which operates during flood stages to relieve the main river channel of part, or all, of its flood flow (i.e., flow in excess of the bank full capacity of the stream). The floodway either empties into the sea or returns its flow into the main river channel at some point below. From the above it is apparent that the two terms really overlap in their meanings. To clarify the use of terms here, however, the term outlet as used in this chapter will refer in general to an all-stage, natural outlet.

Natural outlets are frequently found on the lower reaches of alluvial streams. The recorded history of the Lower Mississippi mentions four such outlets; the Atchafalaya River, Bayou La Fourche, Bayou Plaquemine, and Bayou Manchac. These outlets have been discussed in Chapter II. An outlet diverts part of the flood flow from the main river channel and thereby decreases flood stages in the main river below the head of the outlet. Outlets have been condemned on the grounds that they cause deterioration in the main river channel below which may ultimately result in the abandonment by the river of its channel in favor of a route to the sea through an outlet. These fears have been unnecessary so far as the Lower Mississippi has been concerned. The lower river outlets have all tended to close up and are no longer operative except the Atchafalaya River. The enlargement of this outlet is the result of the works of man. The deterioration in the main river channel below the head of the Atchafalaya River has been neither marked nor dangerous.

Floodways are not, in themselves, a method of complete flood control. They are normally constructed as an adjunct to a levee system. Floodways may be either controlled or uncontrolled. In the latter type they become operative as soon as the main stream passes above its banks, and remain operative until flood stages have passed. In the former type, the head of the floodway is closed by a structure which prevents operation until the safe capacity of the main river flood channel has been exceeded. When this stage is reached, the floodway takes off the excess flood waters. The structure at the floodway head may be a fixed or moveable weir. In both types, flow down the floodway ceases as soon as the main river stages fall below weir crest elevation. The closing structure may, on the other hand, be an earthen dike or levee designed to crevasse when dangerous flood stages are reached. Such a dike is known as a fuse plug. After rupture of a fuse plug, flow down the floodway continues until flood stages cease on the main river. Objection is advanced to the fuse plug on the ground that a seasoned levee can not always be relied upon to crevasse to such an extent and with sufficient rapidity to accommodate the entire excess flood discharge. This criticism springs from an incomplete concept of proper fuse plug
design. The elevation of the fuse plug crest must be such that it will be overtopped before the safe capacity of the river flood channel is exceeded. Overtopping is always followed by a crevasse which will lower main river stages at and above the fuse plug. Should this crevasse be insufficient to accommodate the entire excess flow, rising main river stages will again overtop the fuse plug and the process will be repeated. The topography characteristic of an alluvial valley readily lends itself to floodways. An alluvial valley is usually made up of a series of side basins whose bottom lands are lower than the elevation of the main river banks and whose drainage generally parallels the main river. These side basins are in fact natural floodways. Artificial floodways are habitually confined within training works (usually levees) to prevent excessive inundation of the side basins in which they lie. Floodways may be either cleared or uncleared. The increased hydraulic efficiency of the latter type permits them to be much narrower than the former type. The heavy maintenance expense of cleared floodways is, however, an obstacle to their general use.

Levees are the most ancient method of flood control. As has been stated in Chapter VI, their use dates back to remote antiquity. The levee is the most widely used present day flood control expedient. It is perhaps the only method which can alone completely control an alluvial river. Other works are frequently used for partial control. Ordinarily, these works supplement a levee system. The principal objection to a levee system lies in the increased flood stages which it causes. It is, however, claimed by many competent engineers that these high flood stages increase the channel cross section and consequently increase channel discharge capacity. This theory is probably sound, but its operation is so gradual as to be almost imperceptible.

THE PREPARATION OF A FLOOD CONTROL PLAN.

The basic principles governing the preparation of a flood control plan, and the processes involved in its preparation, have been outlined in the introduction to this chapter. For convenience, they are repeated here.

Principles of flood control:—

Soundness.
Economy.
Maximum benefits.
Simplicity.

Steps in the preparation of a flood control plan:—

Study and collection of data.
Selection of method or combination of methods.
Formulation of plan of procedure.

The first principle (soundness) calls for knowledge, experience, and information. Engineers charged with the preparation of a plan must possess a broad general knowledge of river hydraulics. They must understand the theories underlying the various flood control methods and in particular they must be familiar with the histories of the major flood control projects attempted by man. They should also possess practical experience in the construction of flood works. If
possible, this experience should be obtained on the particular river for which the flood control plans are designed. This last statement would be absurd if an elaborate flood control system were planned and executed at a single stroke. Flood control systems are usually slow growths, however, which progress with the economic development of the protected areas. As compared with Old World rivers, the progress of flood control on the Lower Mississippi has been relatively rapid. The development of flood works on this river has all taken place since the settlement of New Orleans; a short period as compared with that in which the development of the Po River project took place. Since the beginning of Federal operations on the Mississippi, the lower river has served as a laboratory for the development of river engineers especially qualified to cope with its individual problem. The recurrence in technical reports of such names as Humphreys, Abbot, Suter, Eads, Ockerson, Henry Flad, and Townsend demonstrates the successful exploitation of the opportunity thus afforded.

Accurate and detailed technical information is of course essential for the preparation of detailed plans. This information can be obtained only by observations extending back over periods of years. The most necessary data are of course those concerning the maximum known floods both in the main river and in its tributaries. Estimates of the size of the maximum probable flood against which protection is to be provided are based on the flood history of the river and its tributaries. It was fortunate that the economic exploitation of the Lower Mississippi River Valley has been gradual. Had a complete flood control system been necessary immediately after the settlement of the valley, the lack of technical river data would have seriously hampered its development. As a matter of fact, the lack of these data rendered inadequate the early plans for comprehensive flood control. These data include accurate surveys of the river channel and adjacent areas, surveys of outlets, natural side basin floodways, etc. It includes gage records both on the main river and its tributaries extending over a period of years; a record of major floods on the main river and its tributaries; and information covering both the areas naturally subject to overflow and those actually overflowed during floods of record. Information on the rates and amounts of bank caving and sedimentation is essential. Discharge observations at all stages and extending over a period of years are necessary. They must be sufficiently accurate and voluminous to permit the preparation of reasonably accurate discharge rating curves. Without these necessary data the engineer must depend upon the natural evidences of overflow such as high-water marks, etc., the unscientific statements of the local populace, and the histories of floods as preserved in local records. These data are obviously inaccurate and susceptible of misinterpretation.

The principle of economy requires that the construction and maintenance costs of a flood control system be reasonable as compared with the value of the benefits derived therefrom. These benefits may be divided into two classes, direct benefits and eventual benefits. Direct benefits are those which are immediately operative, while eventual benefits are those which result from the increased exploitation of the protected areas after the completion of the flood control system. With respect to agriculture, eventual benefits may be of doubtful value. The drainage and clearing of newly protected swamp lands is an expensive task. In
fact, the cost of such work is frequently as great as the value of the lands after improvement. This usually eliminates the probability of greatly enhanced agricultural values in the protected areas.

The proponents of untried and visionary schemes for Mississippi River flood control are prone to advance uncertain eventual benefits as a reason for the expenditure of enormous sums of public money. The expenditure of these sums on such visionary schemes would, in the general case, be folly. The builders of a large flood control system furnish the opportunity for increased exploitation of the resources of the protected area but they usually command neither the material means nor the power to make that exploitation a fact. A flood control system should be designed to meet present needs economically. It should also be sufficiently flexible to permit its modification or elaboration if, and when, needed to protect increased industrial or agricultural activities. Beyond this point it should not go.

It is therefore apparent that, in the general case, the economic set up of a flood control plan must rest entirely upon the direct benefits which it bestows upon the protected areas. Direct benefits are functions of the values of the land and physical installations within the protected areas, as compared with potential flood losses. The determination of these direct benefits rests upon an appraisal of existing values and an estimate of the potential flood losses. An accurate evaluation of land and improvement values can usually be obtained only by direct appraisal. Assessed valuations (for taxation) are frequently too inaccurate to be useful. Furthermore, the recorded valuations of public improvements (highways, water works, etc.) are sometimes widely different from the actual, fair values.

The estimate of potential flood losses must be based upon losses caused by previous floods of record. The accurate appraisal of such past losses is a difficult and intricate problem. Contemporary flood loss estimates are likely to be greatly exaggerated. This is natural since the inhabitants of the flooded areas are subject to excitement and despondency; and are therefore apt to write off as a total loss, damaged property which may later be salvaged for a very considerable proportion of its previous value. It may be noted here that, since the flood of 1927, a policy has been inaugurated looking to the prompt collection of flood loss data. After each flood, surveys are undertaken to determine the extent and character of the flood losses within the territorial limits of each of the three Federal Engineer Districts on the Lower Mississippi.

Flood losses may be either direct or indirect. Direct losses include the destruction of, or physical damage to, property. Indirect losses are those occasioned by the interruption of human activity in the flooded area and the decreased productivity of this area during the flood. The distinction between direct and indirect loss is best illustrated by example. Damages to buildings, the destruction of existing crops, livestock losses by drowning, and actual damages to highways are some examples of direct flood losses. Indirect flood losses on the other hand, are those resulting from decreased industrial or agricultural activity within the flooded area; losses due to the reduction in commercial activity during the flood and the period of recovery thereafter; losses due to the reduction in freight and passenger traffic, etc.
In the determination of the total flood losses resulting from a given flood, care must be taken to avoid duplication. Losses to individuals or industrial concerns, which are compensated for by gains to other individuals or concerns within the flooded area, do not represent net losses to the flooded area as a whole. This point is best illustrated by example. For instance, agricultural land rentals frequently appear as indirect flood losses. The rental of a farm is normally paid for from the proceeds of the sale of the agricultural products of that farm. If the indirect crop losses to that farm (through failure to cultivate it during the flood) are carried at their full value, it is improper to consider the rental value as a loss also.

In estimating indirect crop losses, it may be a mistake to compute them at commodity prices as they actually exist for the year following the flood. A flood which materially cuts down national production of any essential commodity is followed by a reduction in the existing reserves of that commodity and by a rise in its price. Thus the losses occasioned by a given flood may be partially offset by higher commodity prices for the next crop. To estimate crop losses at such increased prices is merely to exaggerate them.

One economic feature of flood control is not generally appreciated. Flood control may actually result in a damage to agriculture which may ultimately go far to offset the agricultural benefits derived from the occasional saving of crops from flood. The fertility of an alluvial valley results from periodic overflow in its state of nature. The successive deposition of layers of rich silt over the surface of the valley causes the enormous fertility characteristic of alluvial soils. The protection of an alluvial valley from flooding denies it this enrichment. A progressive decrease in fertility therefore inevitably results, which must ultimately be counteracted by expensive artificial fertilization. The valley of the Lower Nile is the classic example of this state of affairs. Care is taken, not to protect the Nile Valley from flood but, on the contrary, to secure its periodic inundation, otherwise the valley would lose its agricultural value.

The estimate of flood losses may therefore be summarized as follows: Direct flood losses should be included in the estimate at their full value after due allowance is made for salvage. Since these direct flood losses include only the physical injury or destruction of property, it is immaterial where they occur or what they are. They represent a loss in the total wealth of the flooded area. Indirect flood losses must, on the other hand, be carefully studied to eliminate duplication. From what has been said above, it is apparent that the net loss occasioned by a given flood is appreciably less than the aggregate of the individual losses sustained by all the inhabitants and industrial enterprises of the flooded areas.

The calculation of actual flood losses in the manner described above, furnishes an index of the amount of the potential flood losses. These potential flood losses are the losses which might reasonably be assumed to occur were flood protection not provided. They are a function of the frequency and extent of destructive floods as estimated from past records. The value of lands and installations within the protected area, and the potential flood damages to that area, are the factors upon which the determination of the economics of a proposed flood control plan must rest.
So far in this discussion we have not considered human life. It is not possible to justify an otherwise uneconomic flood control plan solely on the grounds that it furnishes protection to human life; for life alone can be protected without the construction of any flood works at all. Were this the sole purpose of flood control work on the Lower Mississippi, the plan adopted would have been materially different from the one now being carried out. The publication of flood warnings and the evacuation (by force if necessary) of endangered areas would be sufficient merely for the protection of life alone. As a matter of fact, the direct danger to life from Lower Mississippi River floods is greatly overrated in the popular imagination outside the lower river valley. In the popular mind, the failure of a lower river levee during flood is followed by the rapid inundation of vast areas; great loss of life; and the spread of ruin and desolation. This picture is overdrawn. No onrushing wall of water engulfs the helpless inhabitants. The progress of the flood waters over the land is slow. Time is normally available for the inhabitants to remove themselves, and their live stock, from the endangered area. The ruin and desolation resulting from a flood are real enough, but the flood can not rightfully be considered a serious menace to life. Fatalities directly due to floods are usually the result of recklessness or to the natural reluctance of the inhabitants to abandon their homes and possessions. The loss of human life in the flood of 1927 has been estimated by the American Red Cross as 246 souls. In that same year, the loss of life from automobile accidents in the State of Mississippi alone was 277 souls. In March, 1932 a tornado caused nearly 300 deaths in the State of Alabama. This tornado attracted little more than passing national interest. It is apparent, therefore, that the flood danger to life in the Lower Mississippi Valley is comparable with the ordinary hazards of modern day life.

The principle of "maximum benefits" is obvious in its application: Sound business demands that the benefits derived from expenditures for flood control should be as great as possible. This principle is usually served by the protection of the greatest area, but such is not always the case. For example, it would be a mistake to sacrifice a limited but valuable area in order to save a larger area of less aggregate value. The main object of the work must be kept clearly in mind. There is a tendency to confuse flood control with reclamation. The primary purpose of flood control is usually the protection of existing values. Reclamation contemplates the opening of undeveloped areas to exploitation. Flood control projects which are intended partially or solely for reclamation depend upon eventual rather than direct benefits for their economic justification. This has been discussed above. The reclamation aspect of a flood control problem is normally satisfied by the formulation of a plan sufficiently flexible to permit its extension to the reclaimable areas when economic pressure becomes great enough to justify it.

Like the principle of maximum benefits, the principle of simplicity is axiomatic in its application. The simple plan is usually the most efficient and the cheapest. Complicated plans should be critically studied and tested before adoption. They should be hesitatingly discarded if simpler plans can be devised which will provide equal protection at equal cost. Plans should so far as possible be automatic in their operation. Flood works operated by man are sus-
ceptible to misuse through error or intention. In time of flood, public excitement is high and the operation of flood works may be the cause of spirited controversy among local interests. For example, terrorized inhabitants of areas supposedly in danger might seek to compel the needless operation of flood works. On the other hand, local landowners whose property would be inundated by the operation of these flood works might seek to enjoin their operation. These disadvantages largely disappear when the operation of the flood works is made automatic.

The actual preparation of a flood control plan follows the steps outlined above. The first step (study and collection of data) culminates in an estimate of the maximum flood for which protection is planned. This maximum flood is the resultant of the maximum main river tributary floods so calculated as to produce maximum stages in the main river channel. When protection is planned for a river whose recorded history is short, this flood will probably be considerably greater than any one of record. For example, the present Lower Mississippi flood control plan is designed to afford protection against the maximum probable flood. This is greater by from 25 to 28 per cent than the flood of 1927, the greatest one of actual record. The determination of the mere size of this probable flood is, however, not sufficient. Some idea of its frequency is also necessary. Frequency estimates become more accurate with the passage of time. It may be stated that the maximum probable flood for the present Mississippi River flood control project has been estimated to occur with a frequency of about once in 200 years (see House Document 798, 71st Congress, 3rd Session). A flood of this magnitude has never yet occurred in the recorded history of the Mississippi River (over 200 years).

It is, of course, obvious that the collection of data should cover such features as natural side basin floodways, outlets, backwater area, etc. The economic set up must also be determined.

Having collected all requisite data, the engineer then proceeds to prepare a plan. It is seldom that one method of flood control will by itself suffice for the solution of his problem. He should proceed with an unprejudiced mind to select those methods which will best suit his needs and combine them into a harmonious whole which satisfies the four principles underlying flood control. In the preparation of this plan adequate provision must be made for maintenance as well as construction.

The flood control project having been developed, it then becomes necessary to formulate a plan of procedure. This plan of procedure should normally guarantee the most vital protection first. It should also be so fixed that the construction of no part of the work will hinder later work. Important navigation projects must not be interfered with or hindered. In the present flood control plan for the Lower Mississippi, this procedure was exemplified in the immediate construction of the Bonnet Carre' and New Madrid-Birds Point floodways. The former effectually protects the City of New Orleans while the latter relieves Cairo, Ill., from dangerous flood heights. Simultaneously with these works, the construction of the main river levees was initiated and is being vigorously pushed.
The early history of Lower Mississippi River flood control is the history of the lower river levee system. This has been outlined in Chapter VI. The early extensions of the levee system are illustrated on Plates XXXII to XXXV, inclusive. The interest of the Federal Government in the Mississippi River was at first confined to navigation. As outlined in Chapter X, navigation improvement by the United States was undertaken as early as 1820; while the swamp acts (1849-1850 et seq.) were the first evidence of Federal interest in the flood problem. Two early operations in the interests of navigation had some effect upon floods, however. These were Shreves Cut-off (1831) and Raccourci Cut-off (1848) which have been discussed in Chapter II.

The decade beginning with the year 1840 seems to have been characterized by considerable interest in the improvement of the river. At a convention held at Memphis in 1845, Mr. John C. Calhoun advanced the view that flood protection was a national problem as well as navigation improvement. This growing sentiment was increased by the floods of 1849 and 1850 which caused widespread damage. The passage of the swamp acts followed. Although these acts evidenced Federal interest in flood control on the lower river, they can not be considered as an assumption by the United States of the responsibility for flood protection. They were in reality an attempt by the National Congress to provide the several valley states with the means wherewith to protect themselves. Under their operation the following aggregate areas were turned over to the lower valley states. These acreages are based on the computations of the General Land Office (Department of the Interior) and are the latest available (June 30, 1931).

TABLE XLIII

Lands Conveyed to Lower Valley States By Operation of the Swamp Acts

<table>
<thead>
<tr>
<th>State</th>
<th>Acres</th>
</tr>
</thead>
<tbody>
<tr>
<td>Illinois</td>
<td>1,457,559</td>
</tr>
<tr>
<td>Missouri</td>
<td>3,346,936.01</td>
</tr>
<tr>
<td>Tennessee</td>
<td>None</td>
</tr>
<tr>
<td>Kentucky</td>
<td>None</td>
</tr>
<tr>
<td>Arkansas</td>
<td>7,686,455.57</td>
</tr>
<tr>
<td>Mississippi</td>
<td>3,290,285.79</td>
</tr>
<tr>
<td>Louisiana</td>
<td>9,433,348.79</td>
</tr>
</tbody>
</table>

The aggregate area of lands conveyed to the States of Louisiana, Mississippi, and Arkansas alone under these acts has been about 31,890 square miles. It is interesting to compare this figure with the areas overflowed by actual Mississippi floods. The total areas overflowed by the floods of 1882 and 1927 approximated 34,600 square miles and 23,000 square miles respectively. If we add to the lands conveyed to the three above mentioned states, those lands conveyed to the State of Missouri, we have a total approximate area of 37,120 square miles, an area greater than that overflowed by any flood of record. Illinois has been omitted from this calculation since the area of this state falling within the Alluvial Valley of the Mississippi is only
about 112 square miles. So far as this state is concerned, however, it is noteworthy that the area within the Alluvial Valley plus the area within the Illinois River Valley subject to overflow by Mississippi River backwater (and consequently within the scope of the present Mississippi River flood control plan) aggregates approximately 420 square miles. This may be compared with approximately 2,277 square miles conveyed to the state by the swamp acts. It is not to be assumed that all the lands discussed above are within the Alluvial Valley of the Mississippi. However, with the exception of relatively unimportant areas in Mississippi, Louisiana, and Illinois, these states all lie within the Mississippi River drainage area. It is fair to assume therefore that the area protected by the present Mississippi flood control plan is comparable with, and probably less than, the aggregate of the Mississippi Valley areas turned over to the interested states under the swamp acts. The swamp acts were a failure as a flood control measure, as has been stated in Chapter VI. Thus the Federal Government not only gave to the states the lands subject to Mississippi River floods but has, since the organization of the Mississippi River Commission, assisted in their protection. As will be seen later in this chapter, the present flood control act now places the financial burden of this protection principally on the United States.

In 1851 Mr. Charles Ellet rendered his report on the Mississippi and Ohio Rivers. (See Committee Document No. 17, Flood Control Committee, House of Representatives, 70th Congress, 1st Session.) This report was amplified, revised, and published in book form in 1853 under the title "The Mississippi and Ohio Rivers". The discussion below refers to Ellet's book rather than his official report. Discussion here will be limited to that part of Ellet's report which dealt with the Mississippi River. Ellet confined his discussions of Mississippi floods to those of normal occurrence in the lower valley. He believed that the magnitude of these floods had been limited by climatic conditions which prevented the synchronization in the main river channel of the maximum flood discharges of the tributaries. Abnormal meteorological conditions might, Ellet thought, occasionally permit such synchronization which would produce a flood of abnormally large proportions. He found evidence that such floods had occurred in past ages, and he thought that they might be expected to occur again. He considered the extension of the levee system as the primary cause of the disastrous floods to which the Alluvial Valley was subjected. He asserted, however, that contributory causes were the deforestation and cultivation of the Mississippi watershed which had, in his opinion, increased flood discharges and decreased low-water flow. Other causes were the prolongation of the delta into the Gulf and the effects of cut-offs. Levees raised stages by increasing the confinement of flood discharges. According to Ellet, the banks of the Mississippi were, in their natural state, high and were not deeply overflowed by floods. These banks had numerous low spots which served as natural overflow vents for the river and provided channels for the discharge of surplus flood waters into the swamps situated in the back areas. Many of these vents formed the heads of well-defined overflow channels or bayous. The first white settlers of the Alluvial Valley built their homes on these high banks which were conveniently situated and were least subject to overflow. The
gradual extension of the levee lines closed the natural overflow vents and raised flood stages to the point where successive increases in levee grades became necessary.

Ellet was concerned primarily in the protection of Louisiana from floods although he necessarily considered the entire Alluvial Valley. His proposals for flood protection included the prevention of cut-offs; the enlargement of natural river outlets; the creation of an artificial outlet; the construction of reservoirs; and the strengthening of the levee system.

Ellet regarded cut-offs as an unqualified evil on the Lower Mississippi River. In this he went farther than present day opinion. The value of the cut-off as a flood control measure is now believed to depend upon the individual case. (See Chapter II.)

Ellet's plan for the control of lower river floods is shown on Plate LXVII. His proposals for outlets included the enlargement of the two natural outlets (through the Atchafalaya River and Bayou Plaquemine); and the creation of an artificial outlet into Lake Borgne. He felt no apprehension that these outlets might cause deterioration in the main river channel. His observations had convinced him that natural outlets tended to deteriorate in spite of their relatively steep slopes away from the Mississippi River. Furthermore, he did not think that the flood flow in the main channel below the Atchafalaya and Bayou Plaquemine would be diminished by the improvement of these two outlets. The strengthening of the levee system with the consequent reduction of discharge losses through crevasses would, he thought, greatly increase the flood discharge in the lower river. The improved natural outlets would merely carry off part of this discharge increment thus allowing a sufficient flow down the main channel to maintain it.

The Lake Borgne outlet was intended primarily for the protection of New Orleans. At a point on the east bank of the river and about eleven miles below the city, Ellet proposed to cut a wide gap in the levee to allow the flood waters to escape into Lake Borgne through a deep, cleared floodway. A gap in the levees, 5,000 feet wide would, he believed, lower flood stages from 4 to 5 feet, an amount which he considered sufficient for the safety of New Orleans. He did not propose to limit the floodway to this width, however, but thought it should be made much wider if necessary. He did not fear that the Lake Borgne outlet would cause channel deterioration in the main river below. He considered that the bars at the mouths of the river were the result mainly of sea action rather than river discharge. In fact, the diversion of a considerable part of the Mississippi River discharge (with its load of sediment) into Lake Borgne might, he thought, even improve the navigability of the main river channel. He also believed that the Lake Borgne outlet might eventually become navigable.

Ellet considered Bayous La Fourche and Manchac as possible improved outlets but did not recommend them. He also regarded unfavorably a possible artificial outlet at Bonnet Carre'. His objection to the Bonnet Carre' site was based upon his belief that such an outlet would cause dangerous sedimentation in Lake Pontchartrain. He did, however, concede the possibility that the Bonnet Carre' outlet might ultimately be necessary.
Ellet's report contains a significant speculation as to the necessity for a floodway leaving the Mississippi near the mouth of the Arkansas and discharging through the Red and Atchafalaya Rivers to the Gulf. He made no study of such a floodway but recognized that it might ultimately become necessary.

Ellet's proposals for outlets are of great interest. They may be criticized on the ground that the proposed outlets were too small and were located too near the mouth of the river. Such a criticism is, however, not entirely just. His miscalculation of possible flood discharges was natural in the absence of recorded river data. He was moreover concerned primarily with the protection of Louisiana lands rather than the areas above. He nevertheless recognized that a floodway might ultimately be needed to relieve the main river below the confluence of the Arkansas. He may, however, be justly criticized for his selection of the Lake Borgne outlet site. His speculations as to the effect of the Lake Borgne outlet upon the mouths of the river were incorrect. His fear that a floodway at Bonnet Carre' would cause dangerous sedimentation in Lake Pontchartrain does not accord with later thought. Before the present Bonnet Carre' spillway was constructed as a part of the present flood control project, detailed sedimentation studies were conducted. These studies indicate that no dangerous shoaling will occur in Lake Pontchartrain as a result of the spillway operation. Ellet's concept of an artificial outlet at Lake Borgne was entirely different from the present Bonnet Carre' floodway. The Lake Borgne outlet was expected to be operative at all stages and ultimately to become one of the river mouths. The present Bonnet Carre' floodway is controlled by a fixed spillway and is operative during dangerous flood stages only. Ellet's objection to the Bonnet Carre' site was doubtless based largely upon his concept of an all-stage outlet.

As to levees, Ellet considered that the protection of Lower Louisiana against such floods as occurred at the time of his writing was a relatively simple matter. He thought, however, that increased cultivation would increase flood discharges while the extension of the levee system upstream would increase stages. It was necessary to provide adequately for the great floods of the future. For this additional flood protection, he proposed a line of setback guard levees below Red River. This levee line was to be at no place less than 6 feet above the highest known floods. The existing levees would, Ellet believed, protect the lands in front of the guard levees from minor floods.

The final step in Ellet's plan for flood protection on the Mississippi was the construction of reservoirs at the headwaters of the main river and of its tributaries. Levees had already deprived the river of a large portion of its natural flood reservoir storage. The further extension of the levee lines would deprive the river of more and more of this storage until little or none of it would ultimately remain. When this condition occurred, Ellet believed that any levee system which could be constructed would be inadequate for the protection of the valley. He proposed to substitute for this alluvial valley storage, reservoir storage in the upper reaches of the streams composing the Mississippi system. He advocated first the retention of the surplus waters in the lakes at the sources of the Mississippi and the Missouri; and also along the Red River. Meanwhile, reservoir sites should be located in the valleys
of the Alleghanies. Finally, sites should be sought in the Rocky Mountains. Specifically, he recommended that surveys for reservoir sites be made at the sources of the Monongahela, Alleghany, Kanawha, Cumberland, Tennessee, and other tributaries of the Ohio. In making this recommendation he had in mind not only flood control but also navigation improvement. He further recommended the extension of these surveys for the Red River and its tributaries for the same purposes. He anticipated that the lakes in the valley of the Red could be turned to good account for the prosecution of this plan and he believed that the valleys of its tributary streams afforded remarkable opportunities for the creation of artificial reservoirs. Ellet advocated the detention type of reservoir rather than the storage type.

Ellet's proposals for the use of reservoirs betray an incomplete knowledge of the proper functions of such structures. His book indicates that he advocated the location of flood reservoirs in the upper reaches of the streams. The possible exception to this statement is found in his recommendations concerning the valley of the Red River. The relative ineffectiveness of headwater reservoirs for flood control is well illustrated by the present headwater reservoirs of the Mississippi. The full stage reduction effect of these reservoirs during the 1927 flood amounted to only about one-fifth of an inch at Cairo. To be of real value, flood control reservoirs must be located relatively close to the areas for which they protect.

In general Ellet's studies are worthy of admiration. He not only prophesied the alarming increases in flood elevations which have since occurred, but his flood control plan is in many respects remarkably similar to the present adopted project. He advocated an artificial floodway or outlet for the protection of New Orleans. He called for enlargement of the Atchafalaya outlet, and he indicated the possible need for a floodway reaching from the mouth of the Arkansas to Red River. These measures are all included in the present plan. His concept of the Atchafalaya outlet differed, it is true, from the floodway now proposed. He advocated the enlargement of the Atchafalaya as an all-stage outlet. This was actually accomplished before the present flood control plan was adopted. In addition to this all-stage outlet, the present flood control plan includes a large floodway down the Atchafalaya Basin. This floodway is closed at its head by fuse plug levees. Ellet's greatest mistake was probably his advocacy of headwater reservoirs. His conclusions here were unsound.

While Ellet was engaged in his study of the river, field operations were undertaken by the Delta Survey in 1851. These field operations were discontinued later in that same year and were not resumed until 1857. The final report of the survey was submitted in 1861. During these years the attitude of the Federal Government toward flood control studies seems to have been somewhat apathetic. No steps were taken by the National Congress to effect the flood control measures advocated by Ellet; nor was there any apparent pressure exerted to push vigorously the operations of the Delta Survey. This lack of interest was natural. The recent passage of the swamp acts had for the time being placed upon the several states the burden of flood control. It remained to be seen how well or ill these states would use the benefits conferred by those acts. It was unfortunate that the report of the Delta Survey was made at a time when the nation was engaged in a Civil War. Dur-
ing this war, river improvement was at a standstill. After it, the National Congress could not be expected to listen sympathetically to proposals for the protection of territory, the greater part of which fell within the boundaries of conquered southern states. The report of the Delta Survey therefore was slow to receive the attention which it merited. This is indicated in the report of Gen. A. A. Humphreys made in 1866 on the condition of the Mississippi River levees (see Chapter VI). Humphreys stated in his 1866 report that but few copies of the Delta Survey report had been printed, none of which were available for distribution in the Alluvial Valley. He found there in 1866 the same ignorance and misinformation which he had encountered in 1851 and he recommended a wider distribution of the report. It, however, remained for later study to realize the true value of Humphreys' and Abbot's work.

The report of the Delta Survey is important largely for the hydraulic studies contained in it (see Chapter III). The flood control measures advocated by Humphreys and Abbot were recommended only after exhaustive consideration of all practicable methods. Nevertheless, informed present day opinion does not entirely agree with their proposals. Humphreys and Abbot used the flood of 1858 as a basis for their estimates of the flood protection needed. In this they were nearer right than Ellet who based his proposals largely upon the smaller flood of 1851. Humphreys and Abbot nevertheless fell considerably short of the flood discharge estimates indicated by present day information. This is illustrated by Table XLIV.

**TABLE XLIV**

**COMPARISON OF DISCHARGES (CONFINED) FLOODS OF 1858 AND 1927**

(Estimated maximum probable discharges contemplated by present flood control project also shown)

<table>
<thead>
<tr>
<th>Station</th>
<th>Flood of 1858 (as determined by Humphreys and Abbot)</th>
<th>Flood of 1927</th>
<th>Maximum Probable Flood (Present Flood Control Project—Act of May 15, 1928)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>c. f. s.</td>
<td>c. f. s.</td>
<td>c. f. s.</td>
</tr>
<tr>
<td>Cairo</td>
<td></td>
<td>1,800,000</td>
<td>2,250,000 to 2,400,000 (a)</td>
</tr>
<tr>
<td>Columbus</td>
<td>1,478,000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Helena</td>
<td>1,369,000</td>
<td>1,698,000</td>
<td></td>
</tr>
<tr>
<td>Napoleon</td>
<td>1,418,000</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Arkansas City</td>
<td></td>
<td>2,472,000</td>
<td>2,850,000 (b)</td>
</tr>
<tr>
<td>Vicksburg</td>
<td></td>
<td>2,278,000</td>
<td></td>
</tr>
<tr>
<td>Red River Ldg.</td>
<td>1,338,000(e)</td>
<td>1,779,000(e)</td>
<td>1,500,000 (c)</td>
</tr>
<tr>
<td>Carrollton</td>
<td>1,297,000</td>
<td>1,730,000</td>
<td>1,250,000(d)</td>
</tr>
</tbody>
</table>

**Notes:**

(a) Birds Point-New Madrid floodway will carry 450,000 c. f. s. of this discharge.
(b) Of this aggregate discharge, from 900,000 to 1,250,000 c. f. s. will be carried down the Bonnet Carre' floodway.
(c) In addition to this discharge down the main river channel, 1,500,000 c. f. s. will be carried down the Atchafalaya floodway.
(d) In addition to the discharge past Carrollton, the Bonnet Carre' floodway (above Carrollton) will carry 220,000 c. f. s.
(e) Does not include discharge down Atchafalaya River.
Humphreys and Abbot considered three general methods of flood control, namely, channel improvement; reduction of main river flood discharges; and confinement of flood discharges to the main river channel. Specifically, they discussed cut-offs, tributary diversions, reservoirs, outlets, and levees. They agreed with Ellet that the cut-off was a questionable method of river improvement. They considered impracticable any scheme to straighten by cut-offs the entire river channel, or any considerable portion thereof. They therefore confined their detailed studies to single cut-offs. They stated that a cut-off was always followed by an immediate lowering of the flood stage above with a corresponding stage elevation below. In the case of an unleveled stream, the reasons underlying these stage changes were easily understood. The lowering of upstream flood stages would actually increase channel discharge since it afforded room in the main river channel for flood waters which would otherwise have been discharged over the banks. This increase in channel discharge would naturally cause a stage increase below the cut-off. In the case of a completely leveed stream, however, no discharge increases could follow the creation of a cut-off. Nevertheless, Humphreys and Abbot believed that, on the Mississippi River, an increase in downstream stages would follow the cut-off, even in reaches fully protected by levees. To support this contention, they advanced the theory that after the cut-off, the maximum velocities in the discharge below the cut-off would not be found in the deepest part of the channel. In other words, they believed that the efficiency of the channel below the cut-off was decreased by changes in the direction of the discharge through it, and that a rise in stage below the cut-off would consequently follow even though the discharge remained constant. This rise they realized would continue only until the river could scour out a channel suited to its changed conditions of discharge. They believed, however, that the bed of the Mississippi River was not composed of alluvial soil but lay in an old geological formation of hard clay which eroded so slowly as to be practically permanent. Thus a Lower Mississippi River cut-off would always cause a rise in flood stages below. Subsequent investigation and experience has shown that these conclusions were erroneous. The bed of the Mississippi rests, in fact, in alluvial soils which are easily attacked by the current. As has been shown in Chapter II, present studies indicate that any stage rise which may occur below a cut-off is temporary and continues only until the river has scoured out a new channel suited to its needs. It must be admitted, however, that Humphreys' and Abbot's errors were probably due primarily to incomplete data. They apparently based their conclusions largely upon their observations at Shreves and Raccourci Cut-offs during the flood of 1851. It is probable that at this time considerable flood discharge losses normally occurred from the channel of the river within the zone of upstream influence of Shreves Cut-off since the west bank levee lines were probably inadequate and there were no continuous east bank levees below Vicksburg. In this reach the river follows the eastern escarpment of the Alluvial Valley. There is, nevertheless, a fairly considerable area east of the river and within the limit of upstream influence of the cut-off which was subject to overflow. There was thus available an overbank reservoir storage which might easily have temporarily increased channel discharge after the creation of the two cut-offs. It is thus very probable that Shreves and Raccourci Cut-offs actually in-
creased the channel discharge for the flood of 1851 at this point. While Humphreys' and Abbot's deductions as to cut-offs were erroneous, their conclusions as to their value for flood control are worthy of consideration. They considered cut-offs wholly impracticable for this purpose. As has been stated before, present day opinion dictates that no cut-off should be attempted until detailed and thorough studies indicate its advisability.

In considering tributary diversions, Humphreys and Abbot discussed four proposals relating to the Missouri, the Arkansas, and the Red Rivers. The first proposal contemplated the diversion of the Upper Missouri into the Red River of the North. Humphreys and Abbot considered this wholly impracticable from the standpoint of cost. Furthermore, they doubted its value as a flood control measure. The proposal concerning the Arkansas River contemplated the diversion of this stream either into Bayou Bartholomew or into Bayou Macon. They admitted that this diversion would have a beneficial effect on Mississippi River stages when the Arkansas River happened to be in flood. On the other hand, they stated that it would afford no real relief against Mississippi River floods which occurred when the Arkansas River was low. They also objected to the scheme on the grounds of injury to Arkansas River navigation. There were two diversion schemes for the Red River: The first called for its diversion into Bayou Teche while the second contemplated the closure of Old River. Humphreys and Abbot condemned the first proposal on the grounds of expense and lack of channel capacity in Bayou Teche. They considered that the closure of Old River would do serious harm. The Atchafalaya River was an outlet for Mississippi River floods which would be sacrificed by this closure. Furthermore, they estimated the maximum flood discharge of Red River at 225,000 cubic feet per second while the capacity of Atchafalaya River at bank full stage was only 130,000 cubic feet per second. If, therefore, Old River were closed, the Atchafalaya would not be capable of discharging the maximum Red River floods without serious overflows in the basins of the Red River, Bayou des Glaises, and the Atchafalaya River. On the whole, Humphreys and Abbot recommended against tributary diversions as a means of flood control on the Lower Mississippi River.

In their consideration of reservoirs, Humphreys and Abbot disagreed fundamentally with Mr. Ellet who, it will be recalled, advocated headwater reservoirs. Humphreys and Abbot held that the greater part of the water which composed the destructive floods in the lower river did not drain from remote mountain sides but originated in precipitation within or near the valley itself. Headwater reservoirs would therefore, in their opinion, be ineffective for flood control although they might be valuable for navigation improvement. To control lower river floods, the reservoirs should, they thought, be located near the mouths of the important tributaries which drained relatively large portions of the main river watershed. In their opinion suitable sites for the construction of such reservoirs were lacking. Humphreys and Abbot concluded, from their investigations, that flood protection by reservoirs was entirely impractical for the Lower Mississippi.

In considering outlets, Humphreys and Abbot disagreed radically with previous authorities who held that high-water outlets caused deterioration in the channel below. They found no evidence of such de-
terioration and reached the conclusion that, so far as the river itself was concerned, flood outlets were of great utility. They, however, considered that the necessary floodways (to connect these outlets with the Gulf of Mexico) would be costly and difficult to construct. As to specific outlets, they granted the value of one on the west bank of the river between the mouths of the Arkansas and the Yazoo. Such an outlet would discharge into Bayou Tensas. They stated, however, that while this outlet would be of great benefit above the mouth of the Yazoo, it would be of no service to the regions below Red River. It would necessarily discharge into Red River and the Atchafalaya River which was already a Mississippi River outlet. It could not, therefore, furnish complete relief. Nevertheless, they considered that an outlet with a capacity of 100,000 cubic feet per second might well be constructed in the vicinity of Lake Providence to discharge into Bayou Tensas. This outlet they thought would materially lower flood stages between the mouths of the Arkansas and the Yazoo, thereby necessitating less expensive levee construction than would otherwise be the case. They, however, urged caution in the construction of this outlet. Careful studies were, they thought, an essential preliminary. If these studies indicated the possibility of undue enlargement of this outlet, it should not be built.

Below Red River, Humphreys and Abbot recommended that no artificial outlets be constructed on the west bank of the river. The enlargement of the three existing natural outlets (Atchafalaya, Plaquemine, and La Fourche) they considered too expensive to be practicable. On the east bank they considered three possible outlet sites; Bayou Manchac, Bonnet Carre', and Lake Borgne. Bayou Manchac had at one time been a natural outlet discharging through the Amite River and Lake Maurepas into Lake Pontchartrain. The bayou had, however, long since been closed by a dike. In order to make Bayou Manchac of any value as an outlet, it would be necessary to reopen and enormously enlarge it. Its operation would moreover subject to inundation valuable lands then protected, and would therefore necessitate the construction of an expensive levee system along the bayou. Humphreys and Abbot therefore considered its use as an outlet impracticable.

Humphreys and Abbot gave serious consideration to an outlet at Bonnet Carre'. All that was needed for this floodway was the construction of two floodway guide levees and the destruction of the Mississippi River levee between them. This would create a high-water outlet which would reduce flood stages for many miles above and below. They believed, nevertheless, that the construction of this outlet would be followed by serious consequences. According to their calculations, the necessary capacity of the proposed Bonnet Carre' floodway was about 300,000 cubic feet per second. This discharge would seriously increase stages in Lake Pontchartrain and would necessitate the construction of levees for protection against inundation by the lake itself. They anticipated considerable scour in the floodway since the surface soils in this region were alluvial, and hard clays were not found for some distance below the surface. They considered that the outlet should be about 3,200 feet wide in order to be of the required capacity when it had scoured down to hard clay strata. They anticipated that eventually it would wear through these hard upper strata and would then greatly deepen its channel. Thus it would probably become a permanent low-water branch of the Mississippi. The consequent reduction in the main river
discharge would thereupon be followed by channel deterioration below Bonnet Carre'. It would then be necessary to close the outlet completely. Another serious result would, they believed, be the filling up of Lake Pontchartrain by sediment. Even were the bed of the outlet to be paved so as to prevent scour, Humphreys and Abbot calculated that its operation would fill Lake Pontchartrain by sedimentation in about 375 years. If the outlet scoured out its bed as anticipated, it would, they believed, fill Lake Pontchartrain by sedimentation in about 156 years. They therefore strongly recommended against the construction of the floodway.

The only other locality for an artificial floodway was the Lake Borgne site. The velocity of the current through an outlet here, in Humphreys' and Abbot's opinion, would be sufficient to scour out a channel down to the upper stratum of hard clay at whatever depth that might be found. In their opinion, the time required to fill Lake Borgne by sedimentation would be much less than for Lake Pontchartrain. Nevertheless, they considered the Lake Borgne site more practicable than that at Bonnet Carre'. They, however, recommended strongly against artificial outlets at either site.

Humphreys and Abbot considered levees to be the main expedient practicable for the control of Lower Mississippi River floods. The levee system as it existed in 1858 is illustrated on Plate XXXIII. Humphreys and Abbot found that great practical good had already resulted from this levee system although the levee lines were imperfect and had been crevassed during every previous flood of importance. They discussed three general factors which might affect a completed levee system. These were: prolongation of the delta into the Gulf; increased cultivation of the valley; and increased current velocities. While they believed that the growth of the delta would raise flood levels in the Alluvial Valley, the rate of this change was so slow that they neglected it. The effects of cultivation were, they believed, in a measure compensatory, although they thought that the aggregate effect would be an increase rather than a decrease in Mississippi River flood discharges. They, however, lacked the data upon which to base predictions of the rate of this increase. As to velocities, the effects of levees were compensatory. Levees increased flood stages but they diminished the duration of these flood stages. The increased flood velocities resulting from the development of the levee lines would, they thought, have but little effect in enlarging the river channel. They anticipated that whatever enlargement might result would take the form of widening rather than deepening the channel. They made no allowance for this possible increase in channel cross section. They disagreed wholly with the statements, attributed to M. de Prony and Cuvier, that levees caused the river bed to rise. These contentions Humphreys and Abbot wholly denied.

Humphreys' and Abbot's flood control proposals may be summarized as follows: No advantage could be derived either from reservoirs or from tributary diversions. Cut-offs and outlets were too costly and dangerous to be attempted. Levees were, on the contrary, relied upon for the protection of the alluvial lands below Cape Girardeau. The proposed levee grades were based upon the flood of 1858 and were considered as providing absolute security. (See Table XLV.) In building the levees, however, it might in certain cases be more economical to in-
certain risks of inundation rather than to expend excessive amounts for the construction of levees to provide absolute security. Below the upper limit of influence of the Arkansas and White Rivers, however, it would be unsafe to make any material reduction in the heights of levees computed with reference to restraining the flood of 1858. As has been stated above, the possibility of an artificial floodway on the west bank near Lake Providence emptying into Bayou Tensas was considered.

The Delta Survey flood control plan is illustrated on Plate LXVIII. Humphreys and Abbot plainly advocated the "levees only" method of flood control. Like Ellet they apparently failed to appreciate the possibility of outlets operative at flood stages only. They were also similar to Ellet in their underestimate of the size of the maximum probable flood. As to levees, however, Humphreys and Abbot radically disagreed with Ellet. The latter apparently regarded levees as a dangerous expedient to be used only when no other method of flood control was practicable. The levees advocated by him were merely auxiliaries to his proposed reservoirs and outlets. Humphreys and Abbot, on the other hand, advocated the complete protection of the Alluvial Valley by levees. Both Ellet and Humphreys and Abbot were, however, cognizant of the possibilities of a west bank floodway leaving the river below the mouth of the Arkansas and emptying ultimately into the Red River Basin.

As has been stated above, the report of the Delta Survey was submitted at a time when the national interest was preoccupied with the Civil War. The report resulted, therefore, in the formulation of no national flood control plan.

The end of the Civil War found the levee system in an almost ruinous condition. Little or no maintenance work had been done since the beginning of hostilities and the floods of 1862 and 1865 had caused widespread damage to the weak and inadequate lines. The Engineer Department, however, took prompt action to ascertain the extent of this damage. In December, 1865, Gen. A. A. Humphreys made an inspection of the lower river with a view to the repair or reconstruction of the levees. Humphreys had submitted the report of the Delta Survey but five years previously and therefore thoroughly understood this new task. His report (rendered in 1866) was confined to estimates of the amount and cost of work needed. This report has been discussed in Chapter VI.

The years 1867 and 1868 were marked by the closure of Bayou Plaquemine. This natural outlet of the river had shown signs of possible enlargement which would probably endanger the settled areas through which it flowed. It was therefore closed by a levee.

The disastrous flood of 1867 caused further damage to the levee system and that of 1874 was still more disastrous. A board was created in 1874 by the War Department to prepare a flood control plan. This board, known as the "Levee Commission", is mentioned in Chapters I, V, and VI. Its report is contained in the Annual Report of the Chief of Engineers for 1875. This report of the Levee Commission was quite apparently based upon that of the Delta Survey submitted some thirteen years before. The Commission stated that tributary diversions, cut-offs, and reservoirs were not adapted to the control of the Lower Mississippi River. The Commission admitted the efficacy of high-water outlets but asserted that the difficulties incident to their construction on the Lower Mississippi rendered their
use inadvisable. Specifically, the Commission recommended against a west bank outlet in the vicinity of Lake Providence. This recommendation was based on the fact that waters taken from the main river at Lake Providence must eventually be returned through Red River to the Mississippi or to its outlet, the Atchafalaya. This, the Commission believed, would raise flood stages below Old River. This opinion was based on studies of the flood of 1851. The Commission recommended strongly against the closure of the Atchafalaya River. On the contrary, it recommended the removal of the raft and other obstructions to flow. A careful survey including borings was recommended at Bayou Plaquemine. If this survey showed that there was no danger of its enlargement, the bayou should be reopened as an outlet. The closure of Bayou La Fourche had been recently recommended by a Board of State Engineers of the State of Louisiana. The Levee Commission disagreed with this recommendation although it did not advocate the artificial enlargement of the bayou. The creation of an outlet at Bayou Manchac on the east bank was considered wholly impracticable.

The Commission also considered the construction of an artificial outlet either at Bonnet Carre' or at Lake Borgne. So far as the Mississippi River itself was concerned, the Bonnet Carre' outlet was considered much the more useful of the two. An outlet of 250,000 cubic feet per second capacity located here would render the country below secure without the increase of existing levee grades; and would sensibly lower flood levels at least as far upstream as Baton Rouge. The objections to the Bonnet Carre' outlet advanced by the Commission were four: First, it would raise the water level in Lake Pontchartrain by at least 4 feet thus necessitating costly levees along the lake front. Second, it would destroy easy navigation of the lake. Third, it would increase the difficulties of railroad communication between New Orleans and the north. Fourth, the expense of the project would be very great. The construction difficulties at the Lake Borgne site were much less than those at Bonnet Carre'. The Commission considered, however, that the Lake Borgne outlet would be much less effective than one at Bonnet Carre'. The upstream effects of the former would run out a short distance above New Orleans. Borings made by Prof. C. G. Forshey, however, established the fact that there was no stratum of hard clay sufficiently near the ground surface to serve as a bottom of the Lake Borgne outlet channel. The Commission therefore feared that this outlet might deepen and widen so much that it would gradually pass out of control. Like the reports of Ellet and of the Delta Survey, the Commission report failed to cover the use of controlled floodways operative at flood stages only. All outlet proposals up to this time had contemplated uncontrolled floodways. In their failure to consider controlled works, these proposals were all incomplete.

As its name implies, the Levee Commission advocated levees as the most practicable means for the control of Mississippi River floods. The Commission disagreed diametrically with M. de Prony and Cuvier as regards the effect of levees upon the bed of the river. The contention of de Prony and Cuvier that levees raised the river bed had, in the opinion of the Commission, been refuted in the most conclusive manner by the great Italian hydraulic engineer, Lombardine. The Commission also disagreed with the theory that levees deepened the river chan-
nel. The report perpetuated the mistaken theory of Humphreys and Abbot that the bed of the Mississippi rested upon a stratum of hard clay resistant to erosion. Any enlargement which the levees might cause in the Mississippi River channel must therefore, in the Commission's opinion, take the form of increased width. The report summed up the practical effects of the levees as an increase in the high-water surface and a slight increase in bank caving. Like the Delta Survey, the Levee Commission referred its proposed levee grades to the flood of 1858. These grades were essentially those of the Delta Survey revised where later study had indicated revision advisable. This is illustrated in Table XLV below. A comparison between these grades and those of the present adopted flood control project is afforded by Table XXXIII.

**TABLE XLV**
Comparison of Levee Grades Recommended by the Delta Survey (1861) with Those Recommended by Levee Commission (1875)

Grades referred to high water of Flood of 1858

<table>
<thead>
<tr>
<th>Reach</th>
<th>Delta Survey</th>
<th>Levee Commission</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mouth of Ohio to Osceola, Ark. (165 Miles)</td>
<td>Levee grade to be 3 feet above high water at mouth of Ohio—gradually increasing to 7 feet above high water at Osceola.</td>
<td>Same as Delta Survey.</td>
</tr>
<tr>
<td>Osceola to Helena (141 Miles)</td>
<td>Levee grade to be 7 feet above high water throughout this reach.</td>
<td>Same as Delta Survey.</td>
</tr>
<tr>
<td>Helena to Island 71 (71 Miles)</td>
<td>Levee grade to increase gradually to 10 feet above high water at Island 71.</td>
<td>Same as Delta Survey.</td>
</tr>
<tr>
<td>Island 71 to Napoleon (22 Miles)</td>
<td>Levee grade to be reduced gradually to 8 feet above high water at Napoleon.</td>
<td>Same as Delta Survey.</td>
</tr>
<tr>
<td>Napoleon to Lake Providence (142 Miles)</td>
<td>Levee grade to increase gradually to 11 feet above high water at Lake Providence.</td>
<td>Same as Delta Survey.</td>
</tr>
<tr>
<td>Lake Providence to mouth of Yazoo River (54 Miles)</td>
<td>Levee grades to be gradually reduced to 6 feet above high water at mouth of the Yazoo River.</td>
<td>Same as Delta Survey.</td>
</tr>
<tr>
<td>Mouth of Yazoo River to Natchez (110 Miles)</td>
<td>Levee grade to be 6 feet above high water throughout this reach.</td>
<td>Same as Delta Survey.</td>
</tr>
<tr>
<td>Natchez to Red River Landing (67 Miles)</td>
<td>Levee grade to be 6 feet above high water throughout this reach.</td>
<td>Levee grade to increase gradually to 7 feet above high water at Red River Landing.</td>
</tr>
<tr>
<td>Red River Landing to Baton Rouge (60 Miles)</td>
<td>Levee grade to be 4 feet above high water throughout this reach.</td>
<td>Levee grade to be gradually reduced to 5 feet above high water.</td>
</tr>
<tr>
<td>Baton Rouge to Donaldsonville (56 Miles)</td>
<td>Levee grade to be 3 feet above high water throughout this reach.</td>
<td>Levee grade to be 5 feet above high water throughout this reach.</td>
</tr>
<tr>
<td>Below Donaldsonville</td>
<td>Levee grade to be 3 feet above high water.</td>
<td>At Carrollton, levee grade to be 4.7 feet above high water with proportionate reductions in levee grades below Carrollton.</td>
</tr>
</tbody>
</table>
As regards levee location, the Commission pointed out that the shape of the flood channel had an important bearing upon its efficiency. Salients in the levee line were therefore to be avoided both because they produced channel engorgements and because the salient levee angles were subject to serious erosion. Bank protection works were believed necessary for the improvement of the river. This bank protection would guard the levees against caving banks. In locations subject to serious caving, the Commission recommended that the main levees be set back a safe distance, and that valuable lands so exposed to flood be protected by smaller levee lines which would provide flood protection for about three years out of four. Consideration was invited to the proposition to use the levee lines as railroad rights of way.

The Commission recommended that an accurate and complete instrumental survey of the river be made.

To summarize, the flood control plan of the Levee Commission called for the continued operation of the Atchafalaya and La Fourche outlets. Bayou Plaquemine was also to be reopened as an outlet if surveys indicated that no dangerous enlargement of the outlet would follow. A levee system should be constructed along the main river from the head of the Alluvial Valley to the Gulf. Levee lines should also be constructed along the main tributaries. Double lines, as described above, should be constructed wherever advisable.

The Levee Commission recommended that this plan be carried out by an organization composed of six districts. Each district should be under the charge of a chief engineer who was clothed with ample legal power to carry on his work. He should be immune from local injunction or other interferences in time of flood and he should be empowered to compel the local inhabitants to turnout for flood fighting. The six district engineers should constitute a commission under a president. This commission should be charged with the coordination of the operations of the different districts and should be responsible to the authority from which it derived its existence. Necessary steps should be taken to insure the acquirement of rights of way, borrow pits, etc. The Levee Commission made no recommendation as to whether this flood control body should be a Federal agency or one created by cooperative action of the interested states. It made no recommendation as to how the cost of the project should be met. It did, however, voice the opinion that there was little hope of effective action except by Federal aid.

No effective action either by the states or by the Federal Government followed this report.

In the year 1878 a board of engineer officers was appointed to report upon a plan for the improvement of low-water navigation in the Mississippi. Later instructions were given this board to consider the effects of a permanent system of levees on the lower river upon navigation and flood control. The report of this board was submitted in January, 1879, and is contained in the Annual Report of the Chief of Engineers for that year. This report has been mentioned in Chapters I, V, VI, and X. There is not need to discuss it at length here. Suffice it to say that the board recognized that while levee lines had no influence upon low-water navigation, they did to some extent aid navigation at flood stages. This report gave color to the theory that levees might be considered as navigation improvement.
In the year 1879 the Mississippi River Commission was created as an agency of the Federal Government. The circumstances surrounding the creation of this Commission are discussed in previous chapters and there is no occasion to repeat them here. The importance of the act creating this Commission can not, however, be overestimated. The progress of flood control on the Lower Mississippi had in the past been slow and uncertain. The lack of cooperative action between the states, and the conflicting interests of adjacent communities had combined with the economic prostration of the southern states since the Civil War, to prevent the prosecution of any real and effective flood control measures. Although much thought had been given to the problem of Lower Mississippi flood control, and although voluminous and comprehensive studies on the question had been published, no central operating agency capable of consecutive coordinated action had existed in the past. This crying need was ultimately to be filled by the Mississippi River Commission. The Commission could not, however, assume this position at once. The reasons for this are easy to comprehend. The basic act, which organized the Commission, authorized the construction of neither navigation nor flood control works. Commission activities were at first limited to surveys; to examinations and investigations; and to the formulation of plans for river improvement including flood control. Funds for construction by the Commission became available only from year to year. Early Federal appropriation acts contained provisos limiting levee construction expenditures to those levees built in the interest of navigation. Flood control was a secondary issue. This restriction upon levee construction continued practically without interruption until the passage of the first flood control act in 1917. Then only was the construction of levees purely for flood control finally authorized. While appropriation acts carried this restriction, the Commission was effectively prevented from embarking on any comprehensive flood control project. Such flood protection as it did provide was incidental to navigation projects.

The preliminary report of the Mississippi River Commission submitted February 17, 1880 is contained in the Annual Report of the Chief of Engineers for 1881. Insofar as it related to flood control, the Commission report discussed two methods, outlets and levees. It was the opinion of the Commission that the construction of outlets would be dangerous on a silt charged stream such as the Mississippi. This opinion was based largely upon the observed effects of crevasses on the river and of outlets near its mouth. The diversion of any considerable portion of the flood discharge from the main channel into outlets would, the Commission believed, ultimately raise flood levels. On the other hand, the Commission asserted that levees exerted a great influence in deepening and enlarging the river channel. The Commission did not actually claim that levees were essential to navigation improvement but it voiced the opinion that the repair and maintenance of the existing levee lines would hasten the improvement of the channel. As a flood control measure, the Commission believed that levees were essential. It is interesting to note that the Commission rejected the theory, advanced by Humphreys and Abbot, that the bed of the river rested upon a hard clay stratum. No evidence to support this view was found but the evidence of borings directly contradicted it. It may also be noted that the Commission advised strongly against the construction of the Lake Borgne outlet.
With specific reference to the Atchafalaya River, the Commission recommended that nothing be done pending the completion of studies of that stream which Maj. W. H. H. Benyaurd, Corps of Engineers, was then making. As a matter of fact, the Benyaurd report was actually rendered on February 12, 1880, a few days before the first Commission report. The Benyaurd report was studied by a board of engineers detailed to consider it. The report of this board, dated April 20, 1880, is contained in the Annual Report of the Chief of Engineers for that year. The Benyaurd report contained two proposals for the partial separation of the Mississippi, Red, and Atchafalaya Rivers. The first proposal contemplated the separation of the Red from the Atchafalaya by the diversion of the first named stream into the Mississippi River through the upper branch of the old Mississippi River channel around Turnbull Island (see Plate X). The second proposal contemplated the separation of the Red and Atchafalaya Rivers from the Mississippi by the closure of Old River. The board of engineers rejected both of these proposals on the grounds that they would increase the danger of flood.

The first Mississippi River Commission report may be summarized as advocating the "levees only" plan for the control of the Lower Mississippi River floods. The Commission further advocated the control of the Atchafalaya outlet but did not recommend its closure.

As has been stated in Chapter VI, the Mississippi River Commission in 1880 appointed a committee on levees and outlets which rendered a report in November of that year. This report voiced the opinion that the Atchafalaya River was capable of accommodating the discharge from the drainage basins of the Red and Black Rivers. The restoration of the Mississippi River levees above Red River and the checking or closing of the outlet from the Mississippi into the Atchafalaya River would, the committee believed, reclaim considerable areas along the banks of the Atchafalaya, the Red, the Black, the Ouachita, and the Tensas Rivers as well as along a number of bayous and smaller streams. This committee report marks the beginning of a series of studies which have continued to very recent years. The annual reports of the Commission contain many discussions of the probable effects of the divorce of the Atchafalaya from the Mississippi. In general, proponents of the separation of the Red and Atchafalaya Rivers from the Mississippi River system usually based their opinions upon the following arguments: This separation would, they believed, be a positive insurance against the abandonment by the Mississippi River of its present channel below Old River in favor of a shorter route to the Gulf via the Atchafalaya River. They further stated that the Atchafalaya outlet tended to cause deterioration in the main river channel below Old River. They also held that the exclusion of the Mississippi River flood waters from the Atchafalaya channel would decrease flood danger in the Atchafalaya and Tensas Basins. So far as pertains to the main channel of the Mississippi River, these contentions have not been supported by experience. Shreves Cut-off, the removal of the Atchafalaya raft, and the extension of the Atchafalaya River levees have been followed by marked enlargement of the Atchafalaya River channel despite the construction of sill dams to prevent this enlargement. However, no serious channel deterioration in the main river below Old River has as yet resulted, nor
has the Mississippi River shown any tendency to abandon its present channel for the Atchafalaya River channel. Despite the comparatively steep slopes in the upper reaches of the Atchafalaya, the dispersion of its discharge through a series of ill-defined channels throughout its lower reaches effectually prevents the Mississippi from adopting the Atchafalaya channel. The extension southward of the Atchafalaya River levees will of course concentrate its discharge thereby improving the hydraulic properties of its channel. Should this levee system ever be carried to the Gulf of Mexico, there may be danger of the diversion of the Mississippi River discharge into the Atchafalaya. Suffice it to say now that the problem is not pressing and there will be ample time for such precautionary measures as experience may dictate if and when the need for the occur.

On March 25, 1881 a committee, afterwards known as the "Committee on the Red and Atchafalaya Rivers", was appointed by the Commission for the specific purpose of continuing the Atchafalaya River studies. This committee recommended the construction of a brush sill across Old River for the purpose of checking the enlargement of the Mississippi River outlet.

As has been stated elsewhere in this volume, the river and harbor act of March 3, 1881 appropriated funds for the river improvements recommended by the Mississippi River Commission. The scope of Commission activity was thus enlarged to include active river improvement operations. The Commission levee policies have been discussed in Chapter VI and will not be covered here. They may be summarized by the statement that levee grades were raised and levee design strengthened as experience showed the necessity therefor. Table XXXII shows the levee grades adopted from time to time by the Commission.

By 1884 the Commission studies of the Atchafalaya River problem had so far crystallized that in December of that year a plan for its partial solution was prepared. (See Plate X.) This plan proposed to prevent the further enlargement of the Atchafalaya by a series of six submerged dams or sills of crib and mattress construction, placed across the river channel at intervals not exceeding one-quarter of a mile. These sills had base widths of 300 feet and their crests were fixed just below low water except in the center of the channel where they were sufficiently depressed to permit river navigation at all stages. By 1889 the first two sills of the series had been built, the upper one being at the head of the Atchafalaya River proper at Simmesport. The four remaining sills were never built. By 1894 it was concluded that the two existing sills had accomplished the purpose for which designed and further construction was abandoned.

In the season 1888-1889 construction was undertaken on a dam across the channel of Red River west of Turnbull Island as a part of a project for the divorcement of the Red River from the Atchafalaya (see Plate X). It was built up to an elevation of 3 feet above low water when work was suspended for the time being. It will be recalled that after Shreves Cut-off (1831) Red River flowed into the Mississippi through the upper branch of Old River. By 1872, however, the Red had deserted this channel in favor of the lower branch of Old River. It will thus be seen that after 1872 Red River flowed directly into the Atchafalaya. The Old River channel between the Mississippi and the
Atchafalaya thereupon became a sort of equalizing channel through which the direction of flow changed with changes in the relative stages of the two streams which it connected. The construction of the Red River dam would, the Commission believed, force Red River to flow into the Mississippi through Sugarhouse Chute and Upper Old River. After Red River had opened this channel, the dam was to be raised to a grade 10 feet above low water. The Atchafalaya was therefore to be relieved of Red River discharge except at high stages when flow over the Red River dam would permit the escape of excess flood waters down the Atchafalaya. Under the plan, the lower branch of Old River would become merely an outlet for Mississippi flood waters.

A considerable amount of dredging was undertaken to open the upper branch of Old River and to maintain navigation in the lower branch of Old River during the next few years. By 1897 the project had been modified to include the excavation of an artificial channel across Carr Point for Red River and the closure of the Upper Old River channel below the point of departure of this artificial channel. Notwithstanding continued efforts, however, Red River could not be forced to abandon its old channel and the project was ultimately abandoned. The dam across Red River was now a menace to navigation and was finally removed by explosives.

The severe flood of 1897 attracted national attention with the result that a report on flood control was submitted by the Senate Commerce Committee of the Federal Congress. This report, called the Nelson report after Senator Knute Nelson, the chairman of the committee, is published in Senate Report No. 1433, 55th Congress, 3rd Session. The Nelson report condemned reservoirs and outlets as a means of flood control and recommended the construction of a complete and adequate levee system extending from Cairo to the Head of the Passes. This report did not result in the passage of an effective flood control law. It is, nevertheless, significant as an endorsement of the "levees only" method then in force.

Between 1898 and 1906 the valley enjoyed comparative freedom from floods, the flood of 1903 being the only one of importance which occurred during the period. The act of June 4, 1906 extended the Commission's jurisdiction over levees to the head of the St. Francis Basin. As has been stated in Chapter VI, this enlargement of jurisdiction was a necessary corollary of the earlier adoption by the Commission of the policy of protecting the lower St. Francis Basin. This policy was ineffective unless adequate protection of the upper portions of this basin was possible.

In 1913 the jurisdiction of the Commission for levee construction was extended to Rock Island. In this year, after the occurrence of a disastrous flood, the Mississippi River Commission was directed by the President of the United States to submit a report on flood control. This report was submitted on May 16 of that year and is generally known as the Townsend report after Col. C. McD. Townsend, Corps of Engineers, at that time President of the Commission. This report is published in Senate Document No. 204, 63rd Congress, 1st Session. The report considered six methods of flood control,—reforestation, reservoirs, cut-offs, outlets, floodways, and levees. As to the reforestation, the report held that, while forests might have some influence upon river
discharges during mid stages, their effects were negligible at extreme high or low-water stages. Furthermore, that the enormous expense of the lands required for reforestation was altogether out of proportion to the benefits which reforestation might cause. The report stated that reservoirs offered much promise as a flood control expedient in mountainous country, where short high dams could be constructed to form reservoirs of great depth and volume. In the rolling country which constituted the greater part of the Mississippi Valley, however, an effective reservoir system could be obtained only by enormous expenditures altogether disproportionate to the benefits which it would confer. Cut-offs were not recommended for flood control. It was believed that while they afforded slight relief in the reaches immediately above the cut-offs, they increased flood heights below. Outlets were condemned on the grounds that they would afford local relief only and that they would cause channel deterioration in the main channel below. Outlets were also subject to criticism in that they would require levee protection as well as the main channel. There was, moreover, always danger that the river would desert its own channel for that of an outlet. Floodways were rejected. The report stated that a floodway channel capable of carrying the flood discharge of the Mississippi would necessarily have to be as large as the main river channel itself.

The Townsend report recommended the construction of an adequate levee system. It was a declaration in favor of the "levees only" theory.

The Townsend report merits comment on two points; cut-offs and floodways. Present day opinion does not entirely agree with the report as to the effect of cut-offs. The cut-off is followed by a decrease in flood stages upstream to the limit of cut-off influence but there appears to be no corresponding permanent rise in flood stages below. When such a stage rise is observed, it is the result of an increase in discharge. This has been discussed in Chapter II.

In this report, the Commission evidently held the concept that a floodway should be a channel parallel to the main river channel and capable of carrying the entire flood discharge, or that discharge in excess of bankfull stage. The Commission stated that the flood discharge of the river approximated 2,000,000 cubic feet per second while its bankfull discharge capacity approximated one-half that amount. The floodway channel would therefore necessarily be of the same size as the main river channel. The theory behind the Boeuf and Atchafalaya floodways contemplated by the present flood control plan rests on a different basis. These floodways are adjuncts to the levee system. They are not designed to carry the entire flood discharge and do not become operative until the flood capacity of the leveed channel is taxed to the danger point. They are not channels at all in the strict meaning of the word. They are limited areas within the natural overflow side basins of the Mississippi River which are protected from inundation by the present plan to the same extent that they have been in the past. The levees at the heads of these areas have, however, not been strengthened as have the main river lines elsewhere. They thus constitute weak zones which will fail when the capacity of the main channel is dangerously taxed. The side levees of these floodways limit the overflow to the minimum practicable zone.
The Mississippi River Commission submitted another report in 1913 on the problem of the Red and Atchafalaya Rivers. This report had been directed by Congress in 1910. It was based very largely on observations made during the flood of 1912. The Commission asserted that had the Red and Atchafalaya Rivers been divorced from the Mississippi during the 1912 flood, the Mississippi flood heights below Old River would have thereby increased by about 4 feet. Such an increase in flood stages would not be permanent, however, as the river would enlarge its cross section progressively to accommodate the increased flood discharge resulting from the closure of the Atchafalaya outlet. The plan advanced for the separation of these rivers called for the construction of a dam across Old River. This dam was to be connected by levees to the Mississippi levee line on the one side and to the Atchafalaya River levee line on the other. Navigation was to be provided for by a lock and canal between the Mississippi and the Red. This was to be located near Union Point, 23 miles above the mouth of Old River.

The Commission, however, advanced the belief that the cessation of dredging in Old River would lead to the silting up of that channel and to the eventual separation of the two river systems by natural means. This could be accelerated by the construction of permeable dikes in Old River. This gradual closure of the Atchafalaya outlet would, it was believed, permit the Mississippi gradually to enlarge its channel below thus rendering unnecessary the material increase in main river levee grades below Old River.

In 1914 the Mississippi River Commission directed a study to be made as to the possible construction of a spillway to be located at or near New Orleans for the protection of that city from floods. This study included borings at Bonnet Carre', Kenner, and Lake Borgne on the east bank of the river, and also at Willow Bend, Waggaman, and Jesuit's Bend on the west bank. The report of this study was submitted to the Commission on October 8, 1914. It stated that a flood stage above 21 feet on the Carrollton gage would be dangerous to the commercial and business interests of the City of New Orleans; and that a weir 6,000 feet long, capable of diverting 230,000 cubic feet per second, would be sufficient to prevent floods from exceeding that stage on the Carrollton gage. Objection was, however, raised to east bank sites for such a spillway; principally on the ground of silting in Lake Pontchartrain or Lake Borgne, and of the menace to New Orleans from backwater and through the interruption of the continuity of the levee line by the spillway structure. The chief objection to the west bank sites was the large area which would be overflowed by the operation of a spillway. The conclusions of the Mississippi River Commission were unfavorable to the construction of the spillway and no steps were taken towards its construction.

On March 1, 1917 the first flood control act was passed. This act has been discussed in Chapters I and VI. It is only necessary to repeat here that the act constituted an important change in policy in that it authorized levee construction for flood control. It will be recalled that hitherto levee construction at Federal expense had been limited to that needed for navigation improvement.

On August 12, 1919 the Mississippi River Commission added another report to the growing volume of technical literature on the Atchafalaya problem (see House Document No. 288, 66th Congress, 1st Ses-
This report was rendered pursuant to instructions contained in the first flood control act. It contained estimates for levee construction for the protection of areas within the Atchafalaya River, Bayou des Glaises, Red River, and Black River Basins. As to the divorcement of the Atchafalaya from the Mississippi, this report stated that the greatest measure of protection to the Red and Atchafalaya Basins would be afforded by the separation of the Atchafalaya from the Mississippi and the diversion of the entire Red River discharge into the latter river. This plan was, however, admittedly open to the objection that it would increase the Mississippi flood discharge below Old River. The report discounted fears for the undue enlargement of the Atchafalaya, however. Although this river was admitted to be increasing its channel cross section, the rate of this increase was found to be too slow to encourage apprehensions that the Mississippi might abandon its channel for that of the Atchafalaya at any early date. The deterioration in the Mississippi channel below Old River was also found to be of little significance. The Commission reported that the channel cross sections at low-water and mid stages had diminished to some extent; but that there had been little change in channel cross section at bank full stage. (See Chapter II.)

The works which were actually constructed for the solution of the Atchafalaya problem consisted of two sills placed near the head of the Atchafalaya and a dam across Old River west of Turnbull Island. As has been stated previously, this dam was later destroyed. The value of the two sills in the Atchafalaya River has been the subject of considerable controversy. Since 1926 the United States has not maintained them and they are therefore at present deteriorating. It is difficult to draw a definite conclusion as to the actual value of these works. Their construction has not checked the enlargement of the Atchafalaya River below them. Whether they have retarded this enlargement can not be stated. Both sills have caused extensive scour in the river bed immediately below their downstream toes. These scoured holes have a tendency to enlarge upstream which has necessitated the placing of apron mats to protect the sills. Since maintenance has ceased, the sills will probably ultimately be undermined from the downstream side. The upper sill is being maintained to some extent by the Louisiana and Arkansas Railroad which has a railroad bridge across the Atchafalaya immediately upstream of the sill. It is probable that railroad construction has, to a certain extent, aggravated conditions at the upper sill and is hastening its ultimate failure. The west approach to this railroad bridge is on a fill which extends out from the west bank Atchafalaya River levee. This fill has produced an engorgement of flood flow in the channel of the Atchafalaya River and has no doubt increased scour to a marked extent at this point.

The year 1921 marked the completion of the so-called Cypress Creek closure. This closure has been mentioned in Chapter VI. Cypress Creek gap is a term which, for some years prior to 1921, was applied to a gap between the west bank Mississippi River levees and the south bank Arkansas River levees. The west bank Mississippi River levee line originally bent back away from the main stream, and extended up Cypress Creek as shown on Plate XLI. In 1900 a gap approximately seven miles in length existed between the Mississippi and Arkansas River levee lines at Cypress Creek. This gap was in effect the head of a natural
floodway which became operative whenever the stage at Arkansas City exceeded 50 feet (2 feet above flood stage). Flood waters pouring through this gap inundated extensive areas in the Tensas Basin and finally found their way through natural drainage lines into the Red River Basin. In the flood of 1916, the discharge down the Tensas Basin approximated 336,000 cubic feet per second. The closure of the gap had long been urged by local interests. Work on the closure was undertaken by local interests in 1916 and by February, 1919 was completed down to the north bank of Cypress Creek, reducing the gap to a length of approximately 1,200 feet. At this time operations were discontinued as the Mississippi River Commission objected to the complete closure of the gap. These objections were due to the interruption to local drainage which the closure would cause, and which might lead to heavy damage claims against the Federal Government. Upon the provision of adequate drainage measures, the Commission withdrew its objection and the closure was effected in January, 1921. The expense of this closure was borne by local interests.

Cypress Creek closure denied to the Mississippi its natural overflow outlet down the Tensas Basin and was of course followed by a super-elevation of flood heights in the main channel of the river below the closure. The closure of this gap has created a serious danger spot on the river. This was demonstrated beyond all question in the flood of 1927 when the Mound Landing crevasse occurred on the Mississippi River and when the levees on the south bank of the Arkansas failed. It is interesting to note that the Boeuf floodway proposed by the present flood control plan restores to the river this natural floodway in modified form. Whereas before 1921 the Cypress Creek gap permitted the discharge of flood waters down the Tensas Basin whenever the river had exceeded a very moderate flood stage, the present flood control plan permits the river to use this floodway only when main river stages become dangerously high.

In 1923 the second flood control act was passed on March 4 of that year. This act extended the jurisdiction of the Commission to include flood control works between the Head of the Passes and Rock Island, Illinois, and up the tributaries and outlets of the Mississippi River so far as these tributaries and outlets might be affected by Mississippi River flood waters. This act was in effect the culmination of a series of extensions of Commission jurisdiction. It has been discussed elsewhere in this volume.

The flood of 1922 caused much apprehension at New Orleans as to the possibility of a levee crevasse along the city front. In 1924 the Louisiana State Legislature authorized measures for the reduction of flood levels and for the better protection of the city. In compliance with the terms of this act, a project for a flood outlet from the Mississippi River into Breton Sound was prepared by local interests. This outlet was to be effected by the removal of about eleven miles of levee line at Pointe a la Hache on the east bank of the Mississippi. The upper end of this proposed outlet was located about 50 miles below the City of New Orleans. It was expected that the operation of this outlet would keep the flood stage at the Carrollton gage at New Orleans to not more than 20 feet for a flood of the proportions of that of 1922. This would amount to a reduction of more than 1-foot on this gage as the maximum flood stage of 1922 at Carrollton was actually 21.27 feet.
In 1925 authority for the construction of this relief outlet was granted and the outlet was finished in 1926. As actually constructed, it consists of a break in the levees about eleven miles long and becomes operative when the river passes flood stage at Pointe a la Hache, or at about 10 feet on the Carrollton gage.

It was feared that the construction of the Pointe a la Hache relief outlet might cause deterioration in the river channel below. Surveys made between January 8 and August 5, 1927 from a point about four miles above the upper end of the spillway and extending 5.2 miles below its lower end, reveal that during that period approximately 12,000,000 cubic yards of material had been deposited in the channel in excess of all scour. During the flood of 1927, the maximum discharge through the outlet amounted to about 302,000 cubic feet per second. Computations indicate that the maximum effect of the Pointe a la Hache spillway during the 1927 flood prior to the Caernarvon crevasse amounted to a lowering of stage of about 0.62-foot on the Carrollton gage. Had the Caernarvon crevasse not been created, the probable maximum effect of this relief outlet would have amounted to a maximum lowering of 0.87-foot on the Carrollton gage. Assuming the 1927 flood totally confined and without the Caernarvon crevasse, it is estimated that the Pointe a la Hache relief outlet would have lowered flood stages at New Orleans by a maximum of about 1.18 feet.

The construction of the Pointe a la Hache relief outlet did not allay anxiety for the protection of New Orleans. It is obvious that the safety of the City of New Orleans is intimately connected with the Atchafalaya River which constitutes a relief outlet for the river above the city. Plans for the protection of New Orleans and vicinity should therefore extend upstream to Old River. By the act of April 17, 1926, Congress directed an investigation as to the practicability of controlling Mississippi River floods between Pointe Breeze (about 9 miles above the mouth of Old River) and Fort Jackson (20 miles above the Head of the Passes) by means of spillways. Plans were to be prepared to prevent flood stages from exceeding 16, 17, 18, 19, and 20 feet respectively on the Carrollton gage; and 46, 47, and 48 feet on the Simmesport gage (at the head of the Atchafalaya). The law stipulated, however, that no diversion of Mississippi River waters into Mississippi Sound was to be considered. This stipulation removed from consideration the Lake Borgne outlet proposal first advanced by Mr. Ellet. It will be noted that these investigations were made by a board appointed by the Chief of Engineers. The Mississippi River Commission was not concerned in the studies although engineering data in the hands of the Commission were made available for the use of the board.

This board was commonly known as the Spillway Board. As finally constituted, its membership was Col. W. P. Wooten, Lt. Col. G. B. Pillsbury, Lt. Col. J. L. Schley, Maj. H. S. Bennion, Maj. J. C. Gotwals, and Maj. W. H. Holcombe, all of the Corps of Engineers. The flood of 1927 occurred while the Spillway Board was conducting its studies. Its report, which was based largely on that flood, was submitted November 12, 1927. This report is published in House Document No. 95, 70th Congress, 1st Session. The Spillway Board fixed 2,750,000 cubic feet per second as a maximum probable Mississippi River discharge at the mouth of Old River. This was 250,000 cubic feet per
second less than the discharge assumed at that point for the present flood control project. The board considered three separate plans which were designated as follows:

a. Levees without spillway relief.
b. The Morganza Plan.
c. The Comprehensive Plan.

The "levees without spillway relief" plan was submitted as a basis by which the other two plans might be compared. This plan called for the enlargement of levee lines on the Mississippi River (below Old River); on the Atchafalaya River; on Old River; and on Bayou des Glaises. The construction involved was estimated to cost $77,000,000. This estimate did not include the cost of raising the levees and wharves along the New Orleans city water front. The estimated cost of this additional construction was fixed at $40,000,000.

The Morganza Plan was so named because it provided for a floodway on the west bank of the Mississippi River at Morganza. In order to limit maximum flood stages to 46 feet on the Simmesport gage and 20 feet on the Carrollton gage, this plan proposed the closure of Old River; the construction of floodways at Bonnet Carre', Caernarvon, and Morganza; and the enlargement of existing Mississippi levees between Old River and Bonnet Carre'. The three above mentioned floodways were to be controlled by fixed spillways located at their heads. These flood control measures were estimated to cost $87,300,000. To hold flood levels below 18 feet on the Carrollton gage and below 46 feet on the Simmesport gage would, the board believed, require an additional west bank floodway discharging through Lake Salvador into Barataria Bay. This increased the estimated cost to $94,500,000. The board did not recommend the adoption of the Morganza Plan. The successful operation of the plan was predicated upon the exclusion of Mississippi River flood waters from the Tensas Basin. This the board did not feel was assured under the then existing flood control project.

The Comprehensive Plan contemplated the enlargement of existing levee lines; the construction of spillway controlled floodways at Bonnet Carre' and Caernarvon; and the construction of an uncontrolled floodway in the Atchafalaya Basin. This last named floodway was to be obtained by setting back the west bank Atchafalaya River levees by about seven miles. The estimated cost of the Comprehensive Plan was fixed at $63,600,000. The Spillway Board believed the Comprehensive Plan to be sound but did not consider its immediate prosecution to be economically justified. It recommended therefore, that only such construction be undertaken as would provide protection against a flood equal in magnitude to that of 1927 with a levee freeboard of 3 feet.

The recommendations of the Spillway Board were not adopted. The report of the board was submitted at a time when the Office of the Chief of Engineers was actively engaged in the preparation of that flood control plan which was afterwards adopted and which is now being prosecuted. The studies of the Spillway Board were, however, available as an aid in the preparation of this plan.

In the spring of 1927, the disastrous flood of that year occurred. This flood has been mentioned elsewhere in this volume; there is no occasion to describe it here. It was of prolonged duration. The first
flood waves crested at Cairo on January 7 and last wave crested at Carrollton on April 25. It inundated approximately 23,000 square miles and caused a total of thirteen crevasses on the main river levees. As has been stated elsewhere, this flood resulted in the adoption of the present flood control act.

On June 11, 1927, a board of engineers was appointed by the Chief of Engineers to report upon the practicability and desirability of reservoirs as a means of Mississippi River flood control. This board, as finally constituted, consisted of Col. W. Kelly, Maj. T. M. Robins, Maj. E. L. Daley, Maj. D. I. Sultan, Maj. R. T. Coiner, Maj. E. Graves, and Capt. A. B. Jones, all of the Corps of Engineers. The report of this board was submitted December 1, 1927 and is contained in Committee Document No. 2, 70th Congress, 1st Session. This board which was commonly known as the "Reservoir Board", operated under broad instructions. Its program embraced the following:

a. A study of the existing reservoir systems in the Mississippi River system and their influence upon Mississippi River floods.

b. The capacity, cost, and effect of headwater reservoir systems on the Mississippi River tributaries when operated primarily in the interests of flood control.

c. The potential reservoir capacities in the basins of the main river and of its principal tributaries.

The Reservoir Board found that the effect of existing reservoirs upon the maximum flood heights of the Mississippi River was negligible. These existing reservoirs included eight reservoirs constructed on the western tributaries of the Mississippi River by the U. S. Reclamation Service and operated for irrigation; the headwater reservoir system of the Mississippi River itself; and the Keokuk Reservoir, a hydro-electric power installation at Keokuk, Iowa, on the Mississippi River. The Miami Conservancy Project reservoir system for local flood control on the Miami River, a tributary of the Ohio, was found upon study to be capable of a detrimental as well as beneficial effect upon Mississippi River flood heights. It could therefore not be considered as a factor in the reduction of Mississippi River flood stages. The board reported that the above mentioned reservoirs operated primarily for Mississippi Valley flood protection, would reduce Mississippi River flood discharges by an aggregate of only 2,200 cubic feet per second. This figure is insignificant in a problem as great as that of Mississippi River flood control. The board stated that reservoirs having an aggregate capacity of 97,800,000 acre-feet of water could be constructed in the headwaters of the Mississippi River tributaries at an aggregate estimated cost of $1,292,000,000. This headwater reservoir storage would give a dependable reduction of known maximum flood heights of 5.7 feet at Cairo, 6.9 feet at the mouth of the Arkansas River, and 5.4 feet at the mouth of Old River. Reservoirs with a capacity of 64,000,000 acre-feet could be constructed in the valleys of the Mississippi River and its principal tributaries at a cost of $525,000,000. These reservoirs would give a dependable reduction in maximum flood heights of about 3 feet at Cairo, 10 feet at the mouth of the Arkansas River, and 7 feet at the mouth of Old River. Such protection would not, however, in the opinion of the board, eliminate the need for floodways below the mouth of the Arkansas.
After the flood of 1927, the National Congress appointed committees on flood control to draft a flood control law. The House Committee held extended hearings at which a number of different flood control proposals were advanced. Two of these proposed plans are noteworthy; that of the Mississippi River Commission and that of the Chief of Engineers. Congressional deliberations finally developed largely into a study of the comparative merits of these two plans. In reality these plans were in many respects similar. They differed materially only in the total assumed volumes of discharge and in the proportions of this discharge carried in the leved channel and in floodways.

The plan of the Mississippi River Commission is published in Committee Document No. 1, Committee on Flood Control, House of Representatives, 70th Congress, 1st Session. This plan marked a change in Commission policy. Prior to the 1927 flood, the Commission had maintained that a levee system of reasonable grade, and constructed at a reasonable cost, would suffice to afford flood protection. The flood of 1927 demonstrated, however, that such a levee system would not suffice. The Commission therefore recognized the necessity of providing additional relief measures. In formulating its plan, the Commission gave consideration to the following flood control methods: reforestation, tributary diversions, outlets, reservoirs, channel improvement, floodways, and levees.

Reforestation was not, in the opinion of the Commission, an efficacious method of flood prevention. The return of the land to forests would not reduce surface run-off. Moreover, the process was too slow to provide relief within a reasonable time and its cost would be so great as to be entirely uneconomic.

The tributary diversions considered by the Commission were that of the Tennessee River into the Warrior River system and that of the Red River into the Gulf. The complete diversion of the Tennessee was considered impracticable on account of the regimen of the Warrior River system and the limited capacity of its flood channels. A partial diversion limited to 88,000 cubic feet per second was therefore considered. The proposed Red River diversion was fixed at a maximum of 300,000 cubic feet per second, a discharge greater than that of the Red River. This diversion would therefore draw flood waters from the lower end of the Tensas Basin. Studies of these two possible tributary diversions, however, indicated their cost rendered them uneconomic. The Commission therefore did not recommend them.

As to outlets, the Commission stated that only one natural outlet was in existence, namely, the Old River-Atchafalaya outlet. The Commission did not report on the construction of artificial outlets.

In its study of reservoirs, the Commission classified possible reservoirs under three general headings: headwater reservoirs above Cairo; headwater reservoirs below Cairo; and reservoirs within the Alluvial Valley. The Commission rejected the headwater reservoir system above Cairo as utterly uneconomic. Headwater reservoirs on the tributaries below Cairo were, in the Commission's opinion, practicable on the St. Francis, White, Arkansas, Yazoo, and Red Rivers. The Commission found that the aggregate cost of reservoirs on the first three rivers was prohibitive. Reservoirs on the Yazoo and the Red were not apparently of much value. Reservoirs in the Alluvial Valley would merely substitute
controlled storage for the uncontrolled storage then existing in backwater areas in the basins of tributary streams. The advantages of such controlled storage were so slight in comparison with their cost that the Commission did not believe them to be economic. Nevertheless, the Commission recommended further studies of Alluvial Valley reservoirs and also of headwater reservoirs on the St. Francis, White, and Arkansas.

Channel improvement measures discussed by the Commission included dredging and cut-offs. Dredging was not considered to possess any guarantee of success. Indeed, experience with channel dredging for navigation indicated that it would prove not only costly but entirely ineffective. Cut-offs would improve conditions in the upper reaches of the Alluvial Valley but would greatly increase flood stages in the lower reaches where the channel did not possess sufficient capacity to dispose of the accelerated flood discharge caused by the increased slopes in the channel above.

The flood control plan advocated by the Commission is illustrated on Plate LXIX. It consisted of a strengthened and improved levee system supplemented by floodways and designed to afford protection against a flood discharge of 2,250,000 cubic feet per second at Cairo; 2,850,000 cubic feet per second at Arkansas City; and 2,650,000 cubic feet per second at the latitude of Old River. The levees were to be trapezoidal in cross section with a freeboard of 5 feet above maximum flow line between Cape Girardeau and New Orleans. Below New Orleans this freeboard decreased to 3 feet at the end of the levee line. The Commission stated that it might be advisable to construct so-called safety valve spillways in these levees at selected locations where drainage was afforded by natural drainage lines in the side basins. These safety valve spillways were, in effect, waste weirs through which excess flood waters would discharge when stages became so high that the levee line was seriously threatened.

Floodways above the mouth of the Arkansas River were not deemed to be practicable although the Commission considered further study on this point to be advisable. Four floodways were called for as follows: a Boeuf Basin floodway; an Atchafalaya floodway; a floodway at Bonnet Carre'; and one at Caernarvon. The Boeuf Basin floodway extended from Cypress Creek (below the mouth of the Arkansas River) down the Boeuf Basin emptying into the Red River backwater area. Its maximum discharge capacity was fixed at 600,000 cubic feet per second. It was to be 3 miles wide and partially cleared. A spillway was to be located at its head. The Atchafalaya floodway extended down the basin of the Atchafalaya River and was to be formed by the radical enlargement of the Atchafalaya by setting back the levees on one or both banks of the river as might be found necessary. The maximum discharge capacity of this outlet was fixed at from 900,000 to 1,000,000 cubic feet per second. The Bonnet Carre' and Caernarvon floodways were both to be closed by controlled spillways. The capacities of these floodways were fixed at 250,000 cubic feet per second each. The Bonnet Carre' floodway discharged into Lake Pontchartrain and the Caernarvon floodway into Breton Sound. It may be noted that the Commission asserted that a comprehensive plan for bank protection was an indispensable part of the flood control project.
As to costs, the Commission reiterated the policy of local participation in costs as expressed in previous flood control acts, i.e., that local interests should bear one-third of levee construction costs; should provide rights of way; should assume the payment of damages; and should maintain the completed levees. The policy that local interests should also bear one-third the expense of bank protection on tributary streams was advocated. The Commission, however, qualified the application of these policies under certain conditions and also stated that the entire cost of other flood control structures (including floodways) should be borne by the United States. The cost of the project was fixed at $775,000,000 including $91,000,000 for rights of way and damages.

The Mississippi River Commission plan may be summarized as a strengthened and improved levee system supplemented by four floodways located in the Boeuf Basin, the Atchafalaya Basin, at Bonnet Carre', and at Caernarvon respectively. These floodways were to be controlled by spillways. The estimated cost of the plan was $775,000,000.

The plan submitted by the Chief of Engineers (the so-called Jadwin Plan) is contained in his report as published in House Document No. 90, 70th Congress, 1st Session; and is illustrated on Plate LXX. This plan contemplated a maximum probable flood discharge of 3,000,000 cubic feet per second below the mouth of Reel River. Like the Commission plan, the Jadwin report considered but rejected reforestation, reservoirs, and channel improvement as methods of flood relief. In addition, set-back levee lines and dredged side channels were rejected.

The report stated that forests might have some local effect in reducing torrential floods in mountainous or hilly regions. The Mississippi floods were, however, slowly rising, continued outpourings of drainage from a vast region. Reforestation could therefore not be considered as a part of a comprehensive project for flood control on that river.

In discussing reservoirs, the report summarized the conclusions reached by the Reservoir Board of 1927, and also discussed the possibility of converting the St. Francis Basin into a series of flood reservoirs by means of levees extending across the basin and equipped with controlled openings to govern the inflow and outflow of flood waters. The report rejected reservoirs primarily on the grounds of prohibitive cost and inadequate capacity completely to control lower river floods.

Three methods of channel improvement were considered. These were dredging, channel straightening, and clearing. Dredging was rejected on account of its prohibitive cost and uncertain results. Straightening by cut-offs was regarded as too uncertain and dangerous to warrant adoption. Clearing between levee lines was conceded to be of limited effect in lowering flood heights. This method would, however, increase overbank flood velocities with consequent danger of cut-offs and of scour along the levee lines. Clearing was expensive in first cost and in maintenance. As a flood control measure it was not deemed practicable.

The report estimated that it would be necessary to set the levee lines back about six miles in order to decrease flood stages by about 8 feet. This was estimated to cost $1,000,000,000. Moreover, except in the northern part of the Alluvial Valley, such a set-back would abandon to the river the most valuable lands of the valley and would necessitate
the removal of population centers to areas remote from navigation facilities. Although a general levee set-back was not considered feasible, local set-backs would in many cases prove advantageous and therefore would be made whenever necessary in the execution of the plan which was recommended.

Dredged side channels would, the report stated, prove enormously costly and would silt up rapidly. A partially dredged by-pass channel between Cape Girardeau and White River capable of carrying 300,000 cubic feet per second was estimated to cost $337,000,000 as compared with $27,000,000, the cost of providing equivalent protection by means of levees. As will be noted later, however, leveed floodways were considered practicable and did in fact form part of the proposed flood control plan.

"Levees only" was rejected by the report as being physically impracticable. The existing levee lines were thought to be near the limit of practicable construction as to height and size. Moreover, the raising of the levee line to the grades necessary for a "levees only" plan would not only be extremely costly but would also materially increase the potential damages resulting from crevasses.

The plan recommended is shown on Plate LXX. This plan contemplates the enlargement of the cross section of the existing levee system and the raising of the levee grades on an average of about 3 feet throughout. Four floodways, Birds Point-New Madrid, Boeuf Basin, Atchafalaya Basin, and Bonnet Carre' are provided to care for such flood waters as can not be cared for by the enlarged levee system. The flood stages assumed are those of the maximum probable flood. The levee freeboard over these maximum stages is to be 1-foot.

The Birds Point-New Madrid floodway is to be formed by the construction of a set-back guide levee between Birds Point on the west bank of the Mississippi immediately below Cairo, and New Madrid. The plan does not call for the destruction of the existing riverside levee line between these two points. This line is to be maintained except that the grade of its crown is to be reduced by 5 feet for such distances as to allow the floodway to become operative at a flood stage of 55 feet on the Cairo gage. The maximum project stage at Cairo is assumed at 59 feet on the Cairo gage, which is considered the maximum for which the Cairo, Illinois, levees are safe. The water taken from the river at the upper end of the Birds Point-New Madrid floodway is to be returned at its lower end. The floodway is therefore merely an enlargement of the river channel to prevent excessive stages in this reach of the river.

The Boeuf floodway extends from the vicinity of Cypress Creek on the west bank of the river (below the mouth of the Arkansas) down the Boeuf Basin and empties into the Red River backwater area. It averages about 13 miles in width and is to be confined by side guide levee lines. The Boeuf floodway is uncleared. It is closed at its head by the existing west bank riverside levee which is to be maintained at its present height and cross section. The fuse plug feature of this levee is introduced by raising and strengthening the adjacent levee lines. The Boeuf spillway capacity is estimated to be 900,000 cubic feet per second. Its operation limits the estimated flood discharge in the main river channel below the mouth of the Arkansas to 1,950,000 cubic feet per second. It will be noted that this floodway is calculated to
become operative only when main river flood stages become so high that the fuse plug levee at its head will fail. Since this levee is to be maintained at 1914 standards, it is evident that the Jadwin plan provides to the lands within the floodway the same protection against inundation by floods that had been previously accorded them. Under the plan, waters discharged down the Boeuf floodway will find their way back into the Mississippi through Old River only. This is possible only when the main river stage at the mouth of Old River is low enough to permit the discharge of Old River into the Mississippi. In other words, the reentry of the Boeuf floodway discharge into the Mississippi is possible only when the Mississippi channel can safely accommodate it.

The Atchafalaya floodway is to be constructed by building side guide levee lines down the Atchafalaya Basin. The existing Atchafalaya riverside levees are to be maintained as are the existing levees along the south banks of Old River and Bayou des Glaises at the head of the basin. These levee lines will thus serve to confine ordinary floods. The fuse plug feature is, however, introduced by raising and strengthening levee lines adjacent to the Old River and Bayou des Glaises levees. As in the case of the Boeuf floodway, the lands within the Atchafalaya are to be afforded the same protection as they have previously enjoyed. To insure the complete operation of the floodway, the Bayou des Glaises levee immediately north of the west floodway is to be set back. The maximum flood capacity of the Atchafalaya floodway is fixed at 1,500,000 cubic feet per second, thus limiting to 1,500,000 cubic feet per second the main river flood discharge below Old River.

The Bonnet Carre' floodway is intended for the protection of New Orleans. Its discharge capacity is fixed at 250,000 cubic feet per second, thus reducing the maximum main river flood discharge past New Orleans to 1,250,000 cubic feet per second (or to a maximum stage of 20 feet on the Carrollton gage). The floodway is closed at its head by a fixed spillway. It empties into Lake Pontchartrain. A detailed description of this floodway as it was actually constructed appears later in this chapter. The location of this floodway at Bonnet Carre' was recommended only after a study of the Caernarvon floodway site. This location was rejected on the grounds that the operation of a floodway there would dangerously increase flood velocities past New Orleans thereby inducing scour along the levee lines and interfering with navigation.

The report called for a 9-foot navigation channel 300 feet wide between Cairo and Baton Rouge. This was to be obtained by the regularization method at a cost of $110,000,000 of which $10,000,000 was provided for temporary dredging.

A comprehensive bank protection program to stabilize the river was included in the plan. The plan also called for the production of a general utility topographic map of the entire Alluvial Valley and for the construction of a hydraulic laboratory.

A change was recommended in the flood control organization. Hitherto control of operations had rested in the Commission which reported direct to the Secretary of War. The report recommended that control be centered in the Chief of Engineers under the direction of the
Secretary of War. Under this scheme the President of the Commission became the executive officer directly charged with the prosecution of the work. The local district engineers reported to him direct. The Commission ceased to be an executive agency and became an advisory body.

The cost of the proposed flood project was estimated at $296,400,000. It was recommended that local interests be required to provide rights of way, bear the costs of damage, and contribute 20 per cent to the cost of construction.

The plans of the Mississippi River Commission and of the Chief of Engineers represent a definite step in the development of Mississippi River engineering. They mark the official abandonment of the "levees only" theory. The need was recognized for auxiliary means to keep the maximum main river discharge within the capacity of a levee system of reasonable dimensions. The adoption of floodways as part of the proposed plans did not, however, mark radical or new departures in engineering thought. As has been seen above, escape outlets for excess flood waters had been proposed many times before. Furthermore, on the Sacramento River they had been used in 1910 as the main feature in the flood control plan. One great difference exists, however, between the early and the more recent proposals for the Mississippi River on this phase of flood engineering. Early reports discussed primarily relief channels which were operative at all stages. In other words, early investigators generally confined their studies to either natural or artificial "all-stage" outlets. It remained for the more recent studies to develop the concept of floodways confined in leveed channels and operative at dangerous flood stages only.

Congressional consideration of the flood control problem soon centered upon a comparison of these two plans. Although they differed but little in their engineering features, the estimated costs of the two plans differed widely. This difference lay, namely, in the expensive spillway structures included in the Mississippi River Commission plan and in the large flowage damage estimates provided for by it. Each plan had its steadfast supporters and the deliberations of the Flood Control Committee were lengthy and detailed. Ultimately the flood control act of May 15, 1928 was passed adopting the plan of the Chief of Engineers and directing its prosecution. The law provided for a board of three members; the Chief of Engineers, the President of the Mississippi River Commission, and a civil engineer to be appointed by the President of the United States. This board was charged with the consideration of the engineering differences between the plan recommended by the Commission and the adopted plan, and to report upon them to the President of the United States. The decisions of the President upon these differences were thereupon to become part of the adopted project. The act reaffirmed the principle of local cooperation in construction costs but provided for the payment by the United States of the entire cost of the general project. This provision was adopted on account of the heavy flood expenses to which the local interests had already been put, both for flood control works and in the form of flood damages. Local interests were, however, charged with maintenance of levee lines, and the provision of certain rights of way. Flowage rights were to be purchased by the United States only in cases of the diversion of additional destructive flood waters from the channel of the Mississippi. The expenditure of United States funds on flood works previously authorized on the
main river and on its tributaries was authorized, provided the local interests contributed one-third the cost of construction, provided rights of way, and maintained completed works. This provision applied mainly to the main river above Cape Girardeau and to tributaries below Rock Island within the influence of Mississippi River backwater. Certain provisions were made for flood rescue and emergency flood work. The act modified the jurisdiction and functions of the Mississippi River Commission as recommended in the Jadwin plan. One interesting provision of the law is that requiring a study of the possibilities of reservoirs for flood control. This study will, when completed, be broader and more comprehensive than any reservoir studies previously made for the Mississippi River system. The act authorized the appropriation of $325,000,000 for the carrying out of the adopted plan. Of this amount $10,000,000 was for the previously authorized work referred to above and $5,000,000 was for emergency flood work.

Immediately after the enactment of the flood control act of May 15, 1928, the board to make the comparative study of the Commission and Jadwin plans was formed. This board consisted of Maj. Gen. Edgar Jadwin, Chief of Engineers, Brig. Gen. T. H. Jackson, President of the Mississippi River Commission, and Mr. C. W. Sturtevant, civil engineer. The report of this board was submitted on August 8, 1928. It has been published in printed form by the Mississippi River Commission. The findings of the board were completely in favor of the adopted plan. No change in that plan was recommended.

The passage of the present flood control act has been followed by the vigorous prosecution of the adopted project. The Birds Point-New Madrid floodway and the Bonnet Carre' floodway are now (December 31, 1931) practically completed and are capable of operation in case of emergency. The enlargement of the main river levee lines and of levee lines on the south banks of the Arkansas and Red Rivers has been vigorously pushed. The set-back levee of the Bayou des Glaises loop is complete and numerous set-backs have been made on the Atchafalaya River levees. The navigation plan has been developed on comprehensive lines and has been vigorously prosecuted. The topographic map of the Alluvial Valley is now practically complete. The hydraulic laboratory called for by the adopted project was completed at Vicksburg, Mississippi, in 1930 and is in active operation. The new project received a test in the flood of 1929 but thanks to the rapid strides made between the adoption of the act and the flood, the main river levee system withstood the flood without crevasses.

Although active construction on the adopted project is now well under way, studies as to possible revision have been more or less continuous since its adoption. One report by the Chief of Engineers is noteworthy. This report was submitted on February 28, 1931, and is printed in House Document No. 798, 71st Congress, 3rd Session. As annexes to the report of the Chief of Engineers, appear reports by the Division Engineer, Lower Mississippi Valley Division (President, Mississippi River Commission); the Mississippi River Commission; and the Board of Engineers for Rivers and Harbors. The report is voluminous and considers a number of alternate plans. With certain minor modifications, the adopted plan was recommended for continuance. This report has not yet (December 31, 1931) received Congressional action.
The narrative of flood control operations on the Lower Mississippi River would be incomplete without a description of the Bonnet Carre' floodway. As has been stated above, this floodway admits a maximum discharge of 250,000 cubic feet per second from the Mississippi River into Lake Pontchartrain. A floodway at Bonnet Carre' for the protection of New Orleans is no new scheme. The earliest known suggestion for such a protection measure appears in Darby's "Geographic Account of Louisiana" published in 1816. The first engineering investigation of this site on behalf of the Federal Government was made by Ellet. Since that time discussions of the proposal appear in many Federal engineering reports. They may be found principally in the reports of the Delta Survey (Humphreys and Abbot) in 1861; of the Levee Commission in 1875; and of the Spillway Board in 1927. The recommendation contained in this last report formed a basis for the general type of floodway provided for in the adopted plan.

Pursuant to the adoption of the present flood control plan, a site was located about 32.8 miles above Canal Street, New Orleans (see Plate LXXI). The crest of the natural bank at Bonnet Carre' is about elevation 14 feet mean Gulf level. The ground slopes away from the river towards Lake Pontchartrain to an elevation of about 2 feet mean Gulf level at a distance of two miles from the river bank and to an elevation of 1-foot mean Gulf level near the lake shore. The high lands in the first two miles were cultivated before the construction of the floodway while the remainder of the floodway is flat, swampy, and densely wooded with cypress, gum, ash, and cottonwood. The floodway site is crossed by the main lines of the Illinois Central Railroad, the Louisiana Railroad and Navigation Company, and the Yazoo and Mississippi Valley Railroad. The Jefferson Highway from New Orleans to points north also crosses the floodway site.

A temporary laboratory was established at the spillway site late in 1928 and tests were made to determine the most effective and economical cross section of the structure; probable backwater heights; the proper protection against scour; foundation requirements; and the measures necessary to guard against excessive seepage. The necessary hydraulic computations were also made.

The floodway itself is 7,700 feet wide at the spillway structure and widens out to 12,000 feet between the center lines of the guide levees at the lake. The floodway is 5.7 miles long. The heights of the side guide levees range from 12 to 19.3 feet, averaging about 17 feet. Grades and cross sections of these levees are sufficient to provide a very substantial factor of safety. The floodway is closed at its upper end by a spillway set about 1,000 feet back from the river bank. This spillway is 7,698 feet in length between abutments and is divided into three hundred and fifty weirs by piers spaced 22 feet on centers. Each weir has a clear opening of 20 feet. One-half of the weir crests are at an elevation of 16 feet on the Carrollton (New Orleans) gage, the other half are at an elevation of 18 feet on that gage. The spillway is controlled by a needle dam. An operation track extends over the spillway on the spillway piers. On this track a traveling winch operates for the manipulation of the needles. The girders which support the upper ends of the needles are movable so that in an emergency all the needles in one bay between adjacent piers may be released simultaneously. Immediately below the spillway is a stilling basin in which
three lines of staggered baffles or "dentated sills" are placed to break up the velocity of the discharge over the spillway. Below this stilling basin there is an articulated concrete mat made up of concrete blocks 3 feet square laid on a gravel and riprap base to prevent scour downstream of the spillway.

Based on past flood records, the spillway will probably be operated for periods of from one to three months on an average of once in five years. In operation the flow through the spillway will be limited to that necessary to keep the Mississippi River stage from surpassing 20 feet on the Carrollton (New Orleans) gage. This means that as the river rises needles must be removed in sufficient numbers to take off the crest of the flood. Similarly, as the river falls these needles will be progressively replaced. It is expected that the maximum flow of 250,000 cubic feet per second through the floodway will provide an ample margin of safety for New Orleans while at the same time no inconvenience will be caused due to the raising of Lake Pontchartrain levels. Computations indicate that the maximum discharge will raise the lake only 2 feet which is much less than the variation in lake level caused by storm winds in the summer and fall. Studies indicate that the rate of silting in Lake Pontchartrain as a result of the operation of the Bonnet Carre' spillway will be too slow to occasion the use of protective or corrective measures.