HYDROLOGIC AND HYDRAULIC DESIGN OF SMALL DAMS

by

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INTRODUCTION

The purpose of this paper is to present some of the more important criteria, methods, and techniques that were employed by the above authors in the hydrologic and hydraulic design of a system of small dams for flood control purposes. An attempt will be made to analyze the major design considerations for a system of small dams in the Bayou Bartholomew Basin, located in Arkansas and Louisiana. The paper will consider a specific design for a given basin, rather than the general design applicable to any system of small dams. It should be made clear at the outset that the resulting design to be presented could very easily be one of several, depending upon the criteria by which the engineer is governed and his own ingenuity. The ideas presented in this paper are considered appropriate and adequate for the Bayou Bartholomew study. The current study on Bayou Bartholomew is of a preliminary nature, and, therefore, no detailed studies or estimates, required of a final design, have been made.

The question may arise as to just what constitutes a small dam and when should it be used. There appears to be no definite, clear distinction in the literature between a small and a large dam. However, for the purposes of this paper, we may say that a small dam is any structure whose height above streambed does not exceed approximately 50 feet. The primary concern in planning for the development of a river basin is to select a system of reservoirs which, in conjunction with other works and measures, will best control and conserve the waters in the basin. Consideration should always be given to small, large, and intermediate size reservoir projects and combinations of these. In the Bayou Bartholomew Basin, as in many other basins, utilization of a large dam or a system of large dams is not possible because of topographic and other considerations. However, a group of small dams will provide flood protection for agricultural lands and small communities and contribute materially to reduction in flood damages in downstream areas, particularly from floods of frequent occurrence.

It has been found necessary in the design of small dams, due to limited engineering funds allocated for such projects, to use somewhat generalized criteria and simplified design methods. This procedure eliminates many of the detailed studies required for large dams and yet provides an adequate and economical design. Engineering judgment and use of sound engineering principles, however, are still vital to provide an adequate design.

Description of the Basin

Bayou Bartholomew is one of the principal east-bank tributaries of the Ouachita River. Its source is approximately 10 miles northwest of Pine Bluff, Arkansas, and flows through a tortuous channel to its junction with the Ouachita River at a point immediately upstream from Sterlington, Louisiana. The area of the Bayou Bartholomew watershed is 1,718 square miles, of which 1,074 square miles are upland area and the remaining 644 square miles are flood plain lands. The length of the valley is approximately 145 miles; whereas, the channel length approximates 370 miles. Figure 1 shows a general plan of the basin.

The Bayou Bartholomew Basin is generally rolling in the hill area and nearly level in the flood plain. There is a well-defined escarpment on the boundary between the hill land and the flood plain, and this escarpment roughly parallels the course of the bayou from 1 to 7 miles to the west. Elevations in the hill section range from about 100 feet, mean sea level, in the vicinity of Bastrop, Louisiana, to about 385 feet, mean sea level, in the vicinity of Star City, Arkansas. The alluvial valley portion of the watershed has a slope from north to south of approximately 0.5 foot per mile except in the upstream reaches, where the slope approximates 1 foot per mile.

Distribution of rainfall is fairly uniform over the watershed, with an average annual amount of 51 inches. Heavy winter and spring rains are characteristic of the area. Runoff factors for Bayou Bartholomew vary from 0 to 10 percent in summer and fall, and from 40 to 50 percent in the winter and spring. Approximately 80 percent of runoff in the watershed occurs during the winter and spring period.

The Bayou Bartholomew flood plain is subject to headwater flooding resulting from interior runoff from the flood plain proper and drainage from the hill section of the watershed. The flood problem results largely from the quick runoff from the hill section and the deteriorated condition of the Bayou Bartholomew channel. In addition, the lower part of the flood plain is subject to backwater flooding from the Ouachita River.

Proposed Plan

The proposed plan of improvement on Bayou Bartholomew consists of a system of ten small dams on the western escarpment controlling drainage areas ranging from 10.4 square miles to 192.8 square miles, with a total controlled area of 522.9 square miles. Although the major purpose of the system of small dams is flood control, additional benefits will be achieved from recreation and possibly from inclusion of storage for water supply in some of the reservoirs as the study develops. Storage was provided in each reservoir for conservation, flood control, and surcharge storage. Previous studies considered additional reservoir sites and systems ranging from 8 to 56 small dams. However, at present, the ten reservoir plan is considered more appropriate as each reservoir will provide a minimum of 5,000 acre-feet of storage for flood control. The proposed locations of the reservoirs are shown on Figure 1.

Conservation Pool

Conservation storage includes storage provided for sediment accumulation within reservoir limits expected during the life of the project, and storage for recreation, fish and wildlife, and aesthetic purposes. Reservoir sedimentation studies for hydrologically similar areas within the Vicksburg District indicated an average sediment accumulation of 0.5 acre-foot per year per square mile of drainage area. For an assumed 50-year reservoir life, 25 acre-feet per square mile of drainage area should be provided for sediment storage. This results in an equivalent reservoir storage of 0.47 inches of runoff over the drainage area controlled by each dam. In order to provide additional reservoir storage for recreation and other conservation purposes, a total storage equivalent of about 1.5 inches of runoff over the drainage area was provided in the conservation pool where reservoir depth and storage at the site permitted. A minimum reservoir depth of approximately 6 feet was provided at the dam with the reservoir pool at the conservation level. Another consideration in setting the elevation of the conservation pool was to avoid the undesirable condition of having shallow water depths over a very large area. In order to maintain the reservoir pool at this conservation level, the crest of the weir in the flood regulating structure, to be described later, was set at the elevation of the conservation pool.

Flood Control Pool

Flood control storage was provided to regulate the reservoir design flood without flow over the spillway considering the flood regulating outlet to be operative. The reservoir design flood was selected as one resulting from a 10-day storm with an estimated frequency of once in fifty years. Rainfall values were taken from generalized rainfall isohyetal maps prepared by the U. S. Weather Bureau. USWB Technical Paper No. 40 gives rainfall values for durations from 30 minutes to 24 hours, for return periods of 1 to 100 years. Technical Paper No. 49 gives rainfall values for 2- to 10-day precipitation for return periods of 2- to 100-years. Accumulated rainfall values over the 10-day period were plotted against duration of rainfall, and incremental rainfall values were taken from this curve. Since the rainfall values taken from USWB isohyetal maps are point rainfall values, they were reduced from 0 to 14 percent according to size of drainage area to obtain average area-depth values. Infiltration rates were selected consistent with the region in question and yet conservative for design purposes. Time distribution of the rainfall is an important factor in determining the effects of a particular storm. For the synthetically derived storm from generalized rainfall values, the increments of rainfall excess were arranged in a critical sequence, after observance of the unit hydrograph, which would result in a maximum peak flow and represent the worse condition resulting from a storm of this magnitude.

The damsites are located on minor tributary streams where limited hydrologic information is available. It was necessary to attempt to represent the runoff at each damsite by some empirical relationship. In the design of small dams, the Synder unit hydrograph method has been used successfully, particularly in the study of runoff characteristics of drainage areas where streamflow records are not available. Empirical constants used in Synder's synthetic unit hydrograph method were derived from observed streamflow records in hydrologically similar areas.

It was not possible to realize a 50-year flood control storage at three of the reservoirs due to limited storage and economic considerations. A 5-year, 10-year, and 40-year flood control storage was provided for the three reservoirs. The equivalent flood control storage for the ten reservoirs ranged from 4.3 to 10.1 inches of runoff.

Capacity of Flood Regulating Outlet

The flood regulating outlet capacities were determined after considering channel capacities on Bayou Bartholomew at key locations where appreciable flooding occurs. These channel capacities were expressed in cubic feet per second per square mile (cfsm) of total contributing drainage area above each of the key locations. The locations selected were well within the reaches in which the flood control effects of the dams would be relatively large. From the lowest channel capacity expressed in cfsm, it was determined that an average maximum release rate of 7 cfsm from the ten reservoirs could be tolerated without producing flooding conditions. As a first step in determining the outlet capacity of each structure, it was assumed that the outlet discharge was 5 cfsm when the reservoir pool was at pool elevation corresponding to 25 percent of the full flood control storage. The reservoir design flood was then routed through the reservoir to determine the maximum level attained and the peak outflows. As one can readily see, there is a definite dependency between the size and thus the capacity of the outlet structure, the flood control storage provided, and the elevation of the spillway crest. Successive trials are necessary to furnish the best design. It was necessary to readjust the determined outlet capacities at several of the damsites for economic and practical considerations. However, the capacities were adjusted so as not to exceed the limiting channel capacity.

Spillway Capacity

For small dams capacity of the spillway is directly related to the magnitude of property damage and probable loss of project investment and human life in the event of failure during a severe flood. The proposed reservoirs in the Bayou Bartholomew Basin are located in a predominantly agricultural region where some risk of life and property is possible but not likely.

The spillway crest was set at the top of the flood control pool, determined from the maximum pool elevation attained after routing the reservoir design flood through the reservoir. The length of the spillway was initially assumed and the 100-year storm and the Standard Project Storm were routed through the reservoir according to criteria given below.

Generalized rainfall charts of the Weather Bureau were used to represent a storm having an estimated recurrence interval of 100 years. The synthetic 100-year flood was routed over the spillway assuming an initial pool elevation at the top of the conservation pool and the flood regulating outlet operative. Depth over the spillway was limited to 2 feet for this condition. The 100-year flood was again routed through the reservoir assuming the reservoir pool at an elevation corresponding to 50 percent flood control capacity and flood regulating outlet operative. For this condition, depth was limited to 4 feet over the spillway.

The Standard Project Storm has been widely used by the Corps of Engineers as a guide in the design of certain type projects. The Standard Project Storm estimate was determined using generalized methods and represents the most severe flood-producing rainfall depth-area-duration relationship and isohyetal pattern of any storm that is considered reasonable characteristic of the region in which the drainage basin is located. The Standard Project Flood was routed through the reservoir assuming an initial pool elevation at 50 percent flood control capacity and the flood-regulating outlet operative. It was necessary under these conditions for the spillway to pass the Standard Project Storm with a minimum of 3 feet of freeboard below the top of the dam.

The spillway length was selected after considering the above conditions and criteria. Successive trials assuming different spillway lengths were necessary to satisfy the given criteria and furnish the best design from an economic and practical viewpoint. In any event, the spillway length was set so as to provide a minimum capacity equal to 25 percent of the peak 3-hour average inflow rate of the Standard Project Flood.

Figure 2 gives a recapitulation of Corps of Engineers spillway design criteria for other classes of small dams.

Freeboard

Surcharge pool elevations and storage were determined after routing the 100-year frequency flood and the Standard Project Flood. In all cases, the maximum surcharge pool elevation resulted from routing of the Standard Project Flood. A minimum freeboard of 3 feet was provided between the top of surcharge pool and top of dam. Freeboard allowances for wave runup and wind tide effects were computed based on a 40 mph overland wind velocity oriented in the critical wave-producing direction. Only one of the small dams required a freeboard greater than 3 feet.

Flood Regulating Outlet Design

The flood regulating outlet selected for each of the small dams was a drop-inlet type structure with covered top riser connected to the downstream side of the dam by a concrete pipe through the earth embankment. Figure 5 shows the major features of a typical structure for Prairie Creek Dam. Structures similar to this have been used quite successfully in this area. The discharge through the structure is controlled first by weir flow and later by full pipe flow. The intake structure itself is located within the reservoir a short distance from the toe of the dam.

The inlet invert elevation was set after consideration of the profile along the proposed axis of the dam, and generally was about two feet below the average ground elevation. The outlet invert was set on the downstream side of the dam 1 to 2 feet lower than the inlet invert, depending upon the length of the pipe and the slope of the terrain. The design head for determining the concrete outlet pipe size was taken at the pool elevation corresponding to 25 percent of the full flood control storage. The size of the concrete pipe was selected using an average release rate of 5 cfsm at the calculated design head. Pipe friction losses were evaluated using Manning's "n" of 0.15. Allowance was made for losses due to entrance conditions and trash rack, elbow, and velocity head.

A weir set at the conservation pool level was provided on two sides of the intake structure. The total length of the weir was chosen so as to provide a smooth transition from weir flow to pipe flow conditions by insuring that the transition would occur before the reservoir pool level reached the top cover of the riser. The minimum depth of cover over the weir was 2 feet, or not less than one-half the pipe diameter for pipes with a diameter greater than 4 feet, and ranged from 2 to 5 feet for the ten structures. A minimum water area, L x h, was made equal to 3.5 to 4 times the area of the pipe to maintain reasonable entrance velocities and reduce losses. Figure 3 shows the outlet structure discharge rating curve for Prairie Creek Dam.

Trash racks having a bar spacing of not more than 3/8 times the diameter of the pipe were placed on two sides and one foot along the top edge of the structure to protect the structure and prevent clogging of the pipe from floating debris. A manhole was provided in the top of the structure for access and a slide gate having a diameter of approximately one-half the pipe diameter was provided to allow draining of the reservoir.

Stilling Basin Design

A stilling basin was provided below the outfall of the flood regulating structure to dissipate the energy of the outflow by formation of a hydraulic jump, and to curtail erosion of the downstream channel. Figure 5 shows the plan and profile of a typical stilling basin for Prairie Creek Dam. A transition having vertical side walls was extended from the pipe outfall to the stilling basin floor. The angle of this flared section between the projected conduit axis and the stilling basin side wall is defined by the equation:

$$\phi = \tan^{-1} \left(\frac{1}{\Delta L}\right)$$

The value of ΔL is the flare ratio and represents the distance along the axis in the direction of flow for unit divergence. Actual model tests on circular conduits have indicated that for a straight wall a flare ratio of twice the Froude number is satisfactory. Use of this relationship resulted in a flare ratio varying from 3 to 4.

The lower nappe of the flow profile from the outlet portal invert to the stilling basin floor was analyzed by the theoretical equation for jet trajectory:

$$Y = - \left(X\tan\theta + \frac{gX^2}{2V^2\cos^2\theta}\right)$$

where X and Y are the horizontal and vertical coordinates measured from the beginning of the curve in feet, θ is the angle with the horizontal of the approach invert at the beginning of the vertical curve, in degrees, and g is the gravitational constant, in feet per second per second. The velocity, V, is that which occurs at the beginning of the curve, and as a conservative measure to prevent separation of flow from the floor was taken as 1.25 times the average velocity.

For reasons of economy in construction, a series of steps were used to closely represent the theoretically calculated trajectory. The height of each step was limited from 1 to 2 feet. The length of the transition, width of stilling basin and elevation of stilling basin floor were chosen so that the hydraulic jump for the reservoir design flow would occur at the upper end of the stilling basin. Streamflow records were not available, and it was necessary to estimate tail water conditions. The length of the stilling basin was made equal to 3.5 times the conjugate depth for the design flow. Stilling basin lengths ranged from 14 to 40 feet. A two-foot wide by two-foot high end sill extending the full width of the basin was placed at the lower end of the stilling basin to deflect the flow upward and help prevent eroding and undermining of the lower end of the basin.

Riprap protection was provided downstream of each stilling basin on a 1 on 6 adverse slope from the end sill to a point 1 foot below the elevation of the stilling basin floor. In addition to protecting against channel and bank erosion downstream of the stilling basin, this type of riprapped configuration provides a secondary stilling effect. The hydraulic design of rock riprap involves the relationships of many variables; and there is a current need for additional field and laboratory investigations to permit determination of the most economic design. Therefore, it is not deemed necessary or appropriate in this paper to consider in detail the design of the riprap below the stilling basins. Layer thicknesses and stone gradations were selected according to current Corps of Engineers criteria and were based on the critical velocity over the end sill resulting from the peak flow of the reservoir design flood. Although use of this critical velocity in designing the riprap protection tends to be conservative, it is considered appropriate for a lack of sufficient data on tail water conditions.

Spillway Design

The spillway for each reservoir is to be a broad-crested earth spillway with grass cover, where velocities are not excessive, and was designed to safely pass large flows away from the earth embankment. In all cases, the spillway is to be located away from the dam taking advantage of natural valleys to return the flow to the downstream channel without threatening the safety of the dam itself. The spillways are to be formed through cuts in existing material with sufficient bulk to provide safety against breaching of the spillway during the occurrence of very large floods. It is anticipated that the spillways for the dams having a 50-year flood storage capacity will be used on the average of once during the life of the project. Therefore, the spillways were designed on the basis that some scour and erosion would be permissible.

The spillways were designed using the equation for flow over a broad-crested weir;

$$Q = CLH^{3/2}$$

where C is the coefficient of discharge, L is the spillway length, and H is the total head on the spillway. Velocity of approach was considered negligible and H was taken as the static head on the crest. A "C" value of 3.09 was used in computing the spillway rating curve. A 50-foot wide concrete-lined crest was placed on the spillways of the dams having a 5- and a 10-year frequency flood storage capacity and on those having a velocity over the spillway in excess of about 6 feet per second due to the 100-year frequency flood, and a concrete chute spillway was necessary for the largest dam. Spillway lengths ranged from 300 to 1,000 feet.

Figure 3 shows the spillway rating curve for Prairie Creek Dam and Figure 4 shows the Spillway Design Hydrograph (Standard Project Flood).

Conclusion

This paper has considered very briefly some of the major factors in the hydrologic and hydraulic design of a system of small dams in the Bayou Bartholomew Basin. Use of generalized criteria and simplified design methods furnishes an adequate, economical design and provides a means of comparison of projects in different localities. The reader is referred to publications listed in the bibliography for a more detailed explanation of criteria, methods, and procedures.

Present studies on Bayou Bartholomew are underway to determine the effectiveness of the system of small dams in reducing flooding stages on Bayou Bartholomew and its tributaries. Economic studies will provide estimates of the benefits to be derived from the proposed plan; and then an evaluation of the project can be made.

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CORPS OF ENGINEERS U. S. ARMY

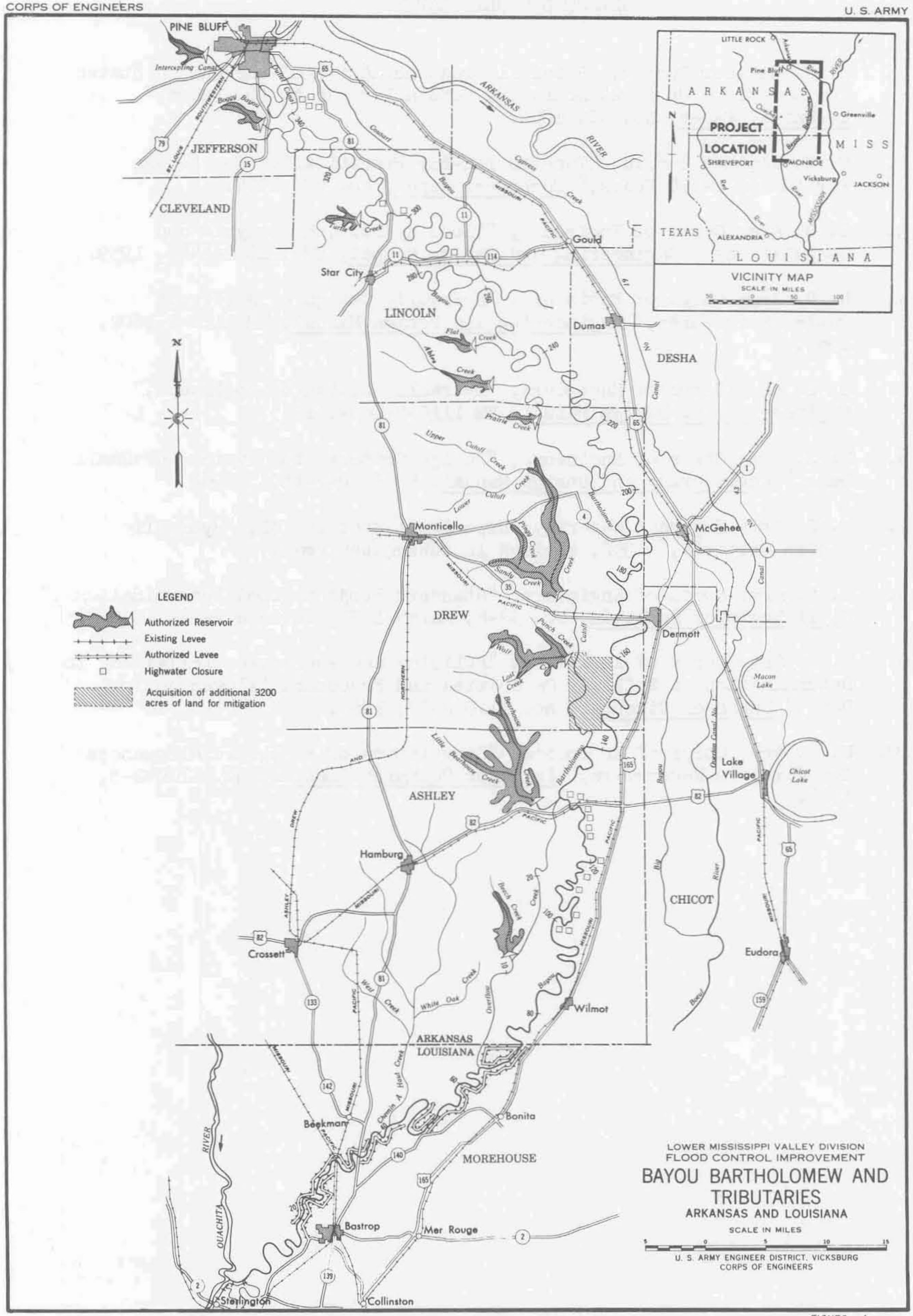


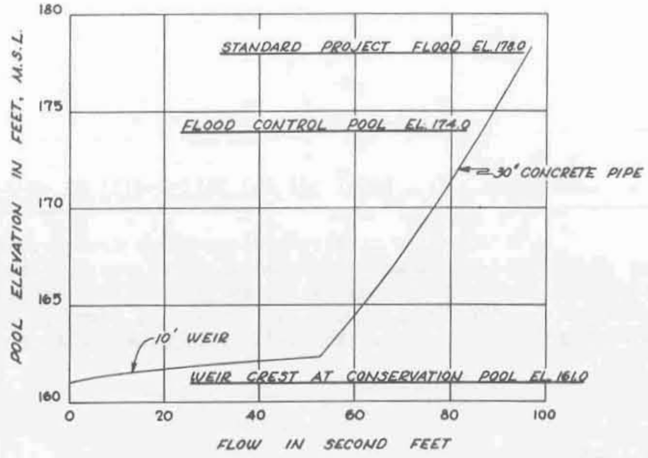
FIGURE 1

FIGURE 2

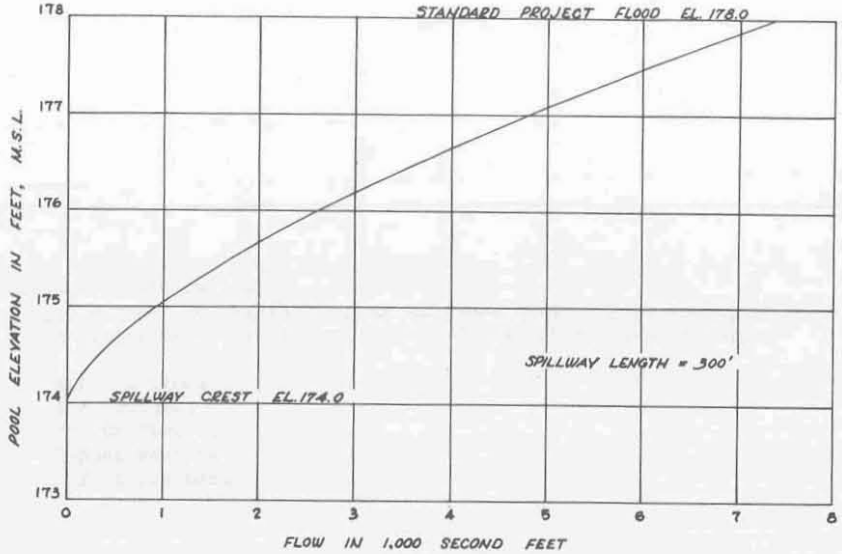
SPILLWAY DESIGN FOR SMALL DAMS

Class* of dam	Design criteria and spillway capacity*: Flood frequency	Pool level at beginning of flood	Flood regulating outlet	Limiting depth of flow over spillway Ft.	Freeboard Ft.
1. Low dams of 5' to 10' or less having impoundments of 5 to 20 acre-feet and located where downstream effects of dam failure would be minor.		Conservation	Operating		May be as low as 1 foot.
2. Dam located where there would be no apparent risk to life or severe property damages, but conditions do not favor adoption of Class 1 dam.	50-year	Conservation	Operating	2	Crest of dam not less than 3 ft. above spillway crest or 2 ft. above the peak reservoir stage during spillway design flood.
3. Dam located in agricultural area at some distance above human habitation and some risk to life and property is possible but not likely, provided an active alert is maintained during storm periods.	100-year	Conservation	Operating	2	
	100-year	50% of flood control storage	Operating	4	
	SPF in accordance with EM 1110-2-1411 (CIVENG BUL No. 52-8)	50% of flood control storage	Operating		Minimum of 3 ft.
	Min. required capacity	25% of the peak 3-hour average inflow rate of the Standard Project Flood			
4. Dam located immediately upstream from an area of human habitation subject to flooding, or there is any doubt as to the ability for evacuation during an emergency, or where potential heavy damage to property is evident.	Spillway dimensions and structural designs will be as required by the various other manuals in the EM 1110-2 series, to safely pass the spillway design flood computed from the probable maximum precipitation.				

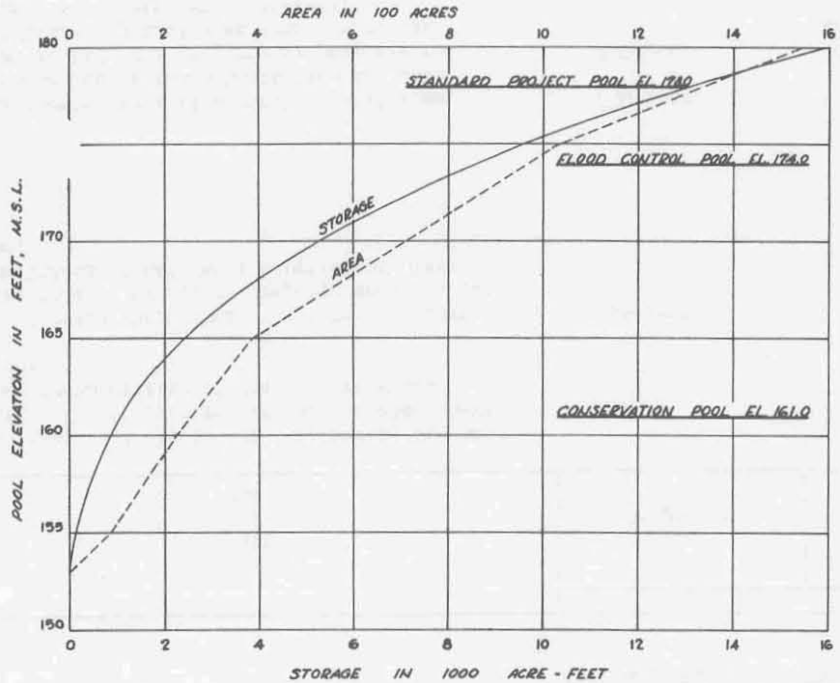
* Reference EM 1110-2-1101 (19 Feb 1968)



CONDUIT RATING CURVE



SPILLWAY RATING CURVE



AREA - CAPACITY CURVE

RATING CURVES
AREA CAPACITY CURVE

PRAIRIE CREEK DAM

FIGURE 3

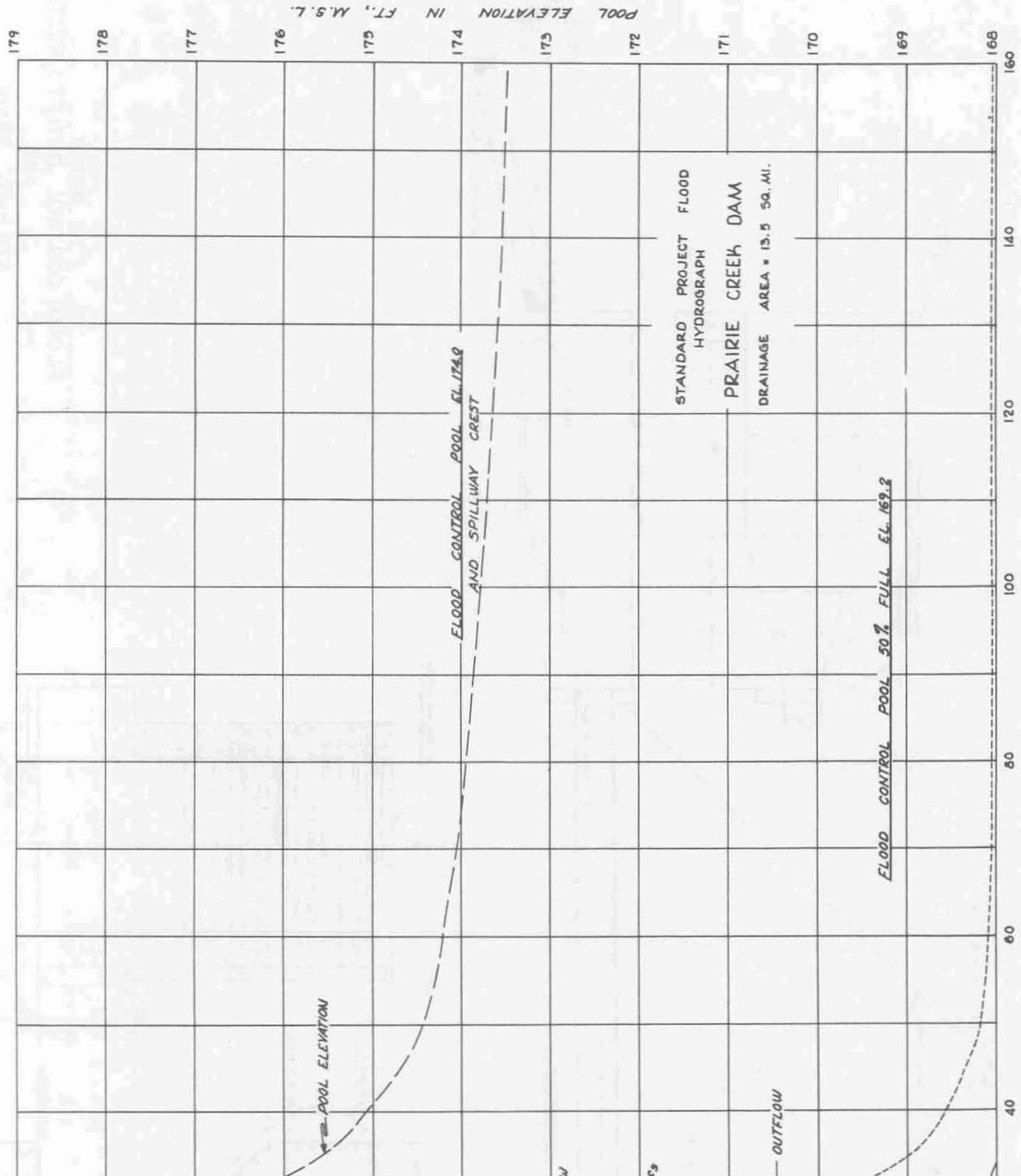
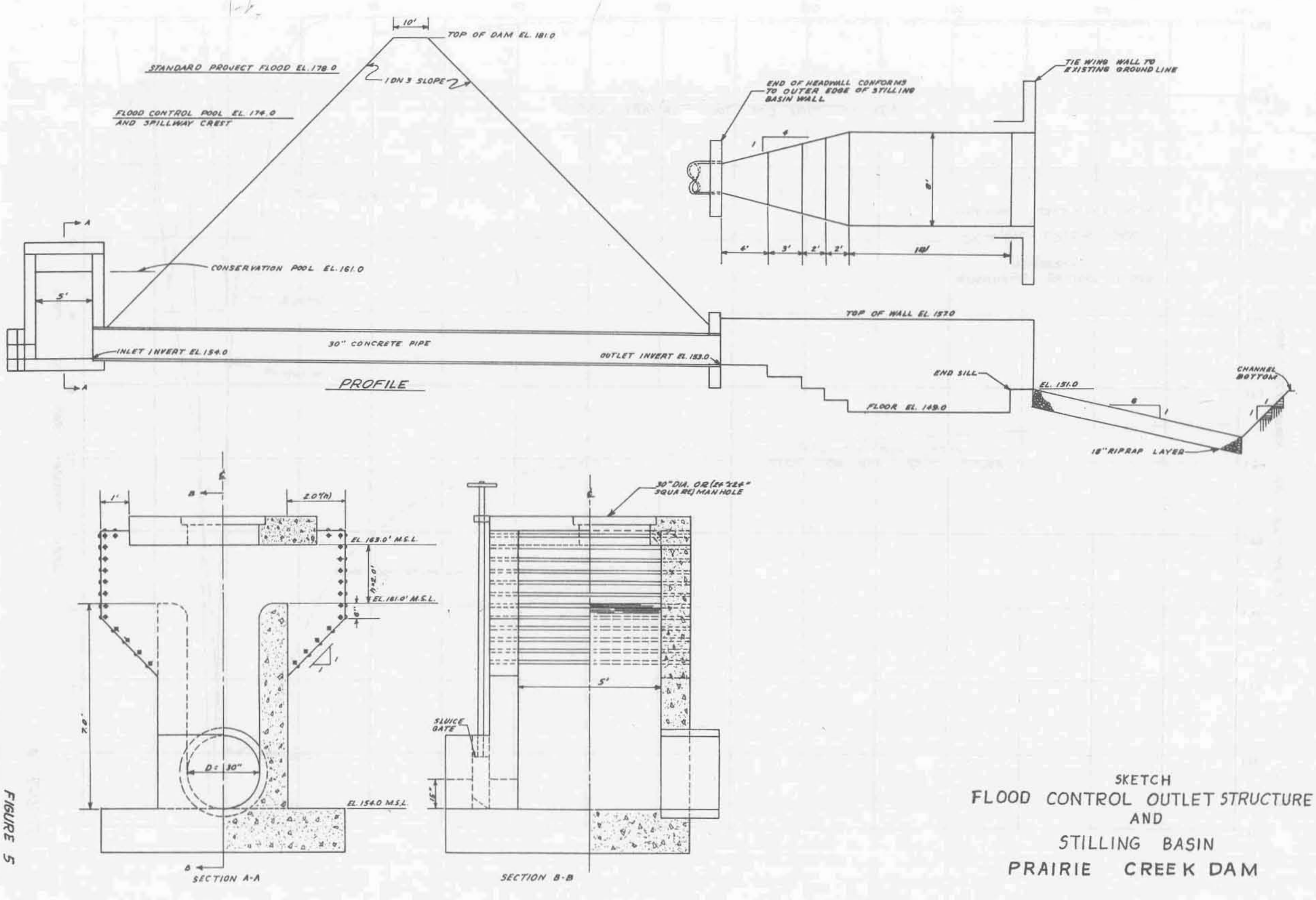

STANDARD PROJECT FLOOD
HYDROGRAPH
PRAIRIE CREEK DAM
DRAINAGE AREA = 13.5 SQ. MI.
FLOOD CONTROL POOL EL. 174.0
AND SPILLWAY CREST
FLOOD CONTROL POOL 50% FULL EL. 169.2
POOL ELEVATION
INFLOW
OUTFLOW
21,400 cfs
7600 cfs
EL. 178.0
FLOW IN 1,000 SECOND FEET
POOL ELEVATION IN FT., M.S.L.
TIME FROM BEGINNING OF RAINFALL IN HOURS
STANDARD PROJECT FLOOD HYDROGRAPH

FIGURE 4



SKETCH
FLOOD CONTROL OUTLET STRUCTURE
AND
STILLING BASIN
PRAIRIE CREEK DAM

FIGURE 5

PERTINENT DATA

PRAIRIE CREEK DAM

Item	Unit	544 Prairie Creek
DRAINAGE AREA	sq mi	13.5
ONE-INCH RUNOFF	acre-ft	720
CONSERVATION POOL		
Elevation	ft msl	161.0
Area	acres	260
Storage	acre-ft	1,100
Equivalent runoff	inches	1.5
FLOOD CONTROL POOL		
Elevation	ft msl	174.0
Area	acres	960
Storage	acre-ft	7,500
Equivalent runoff	inches	10.4
Design Pool frequency	years	50
Release rate at top FC pool	cfsm	6
SURCHARGE POOL		
Standard project flood		
Elevation	ft msl	178.0
Area	acres	1,330
Storage	acre-ft	4,400
Equivalent runoff	inches	6.1
100-year frequency flood		
Elevation	ft msl	175.4
Area	acres	1,070
Storage	acre-ft	1,300
Equivalent runoff	inches	1.8
DAM		
Crest elevation	ft msl	181.0
Upstream slope protection		Veg.
Freeboard		
Standard project flood	ft	3.0
100-year frequency flood	ft	5.6
Height above streambed	ft	28
OUTLET WORKS		
Number of conduits		1
Size		
Diameter	inches	30
Inlet invert elevation	ft msl	154.0
Outlet invert elevation	ft msl	153.0
Intake structure		
Sluice gate-size	inches	15
Riser-size	ft	2.5 x 5
Capacity at top FC pool	cfs	85
SPILLWAY		
Type		Veg.
Crest elevation	ft msl	174.0
Crest length	ft	300
Peak discharge		
Standard project flood	cfs	7,400
100-year frequency	cfs	1,500

FIGURE 6