



**US Army Corps
of Engineers**

Little Rock District

**CLEARWATER DAM
BLACK RIVER, MISSOURI
MAJOR REHABILITATION STUDY
HYDROLOGIC AND HYDRAULIC ANALYSES**

NOTES:

- 1) *All elevations listed in this report are referenced to NGVD.***
- 2) *Unless noted otherwise, river mile (RM) locations referred to in this report DO NOT correspond to "historic" RM locations on the Black River.***

FEBRUARY 2004

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1. Introduction

Clearwater Dam is located at river mile (RM) 257.4 of the Black River in Wayne County, Missouri, approximately 5.0 miles southwest of Piedmont, Missouri, and approximately 49.2 river miles upstream of Poplar Bluff, Missouri. Purpose of the hydrologic and hydraulic analyses associated with the major rehabilitation study is to provide downstream maximum flood heights and flood wave travel times that can be expected to result from seepage related dam breaches. Engineering and Construction Division, Hydrology and Hydraulics Section (EC-HH) performed analyses in support of this study. This report was developed by EC-HH as an appendix to the main study report. Described herein are the hydrologic and hydraulic analyses completed and the results obtained.

2. General

2.1 Scope of Work

Hydrologic and hydraulic analyses were performed for several hydrologic events that might result in a seepage related dam breach - pool at elevation 505.0 feet; 20% PMF resulting in peak pool elevation 581.4 feet with pool receded to elevation 574.7 feet at time of breach; and 100% PMF resulting in peak pool elevation 611.2 feet with pool receded to elevation 593.0 feet at time of breach. It is expected for pool elevations below about elevation 505.0 feet there would be enough warning time to avoid a dam failure by evacuating the pool to a safe level before a breach formed. Failure and non-failure analyses were performed for each event. Failure verses non-failure results were compared to determine the difference in flood heights and flood wave arrival time at various points of interest downstream of the dam.

2.2 Project History

The Flood Control Act of 1938 authorized the project and construction was completed in 1948. The dam is an earth embankment with a gated conduit for controlled flood releases and an uncontrolled emergency spillway in the right abutment. Significant modifications to the earth embankment and to the emergency spillway were constructed in 1989. The upstream side of the earth embankment was modified by addition of a stability berm at the toe of the embankment, and addition of a clay seepage berm which extends up to about EL 575 FT. A concrete parapet wall with top EL 611.0 FT was added to the top of the dam at the upstream side to provide 3 feet of freeboard above

the elevation of the top of the dam. The emergency spillway was significantly widened but construction was stopped before the 280 FT of additional design width was attained. Attainment of the design width would have provided a spillway crest length of 470 feet at EL 567.0 FT. 1989 post construction surveys indicate effective spillway crest length is about 330 feet at EL 567 FT, with an additional 110 feet on the right end ranging from EL 567 FT to about EL 571 FT. Rubble remains along the right bank of the spillway, restricting the spillway width to less than the design width through much of the spillway reach.

It is important to note the required emergency spillway design width developed for the 1989 construction effort may be incorrect due to changes in guidance and technology for the development of a reservoir inflow design flood (IDF). The IDF routed for determination of the required spillway width was developed in 1979 in accordance with Corps of Engineers Engineering Circular (EC) 1110-2-163 (25 August 1975), Southwest Division letter SWDED-XW (15 February 1979), and National Weather Service (NWS) Hydrometeorological Report No. 51 (HMR-51) dated June 1978. Manual methods were used for the distribution and centering of the spillway design storm over the contributing watershed. Three iterations were used to determine the critical storm center. After 1979 computer program HMR-52 was developed to provide a more efficient and sophisticated means of implementing the HMR-51 guidance for rainfall distribution and storm centering. HMR-52 provides an efficient means of performing many iterations to determine the critical storm center. Criteria for development of IDF's has also changed since 1979 in that current guidance (ER 1110-8-2 (FR), Inflow Design Floods for Dams and Reservoirs, 1 March 1991) suggests reservoir inflow unit hydrographs for IDF determinations should be peaked 25 to 50 percent to account for the fact that unit hydrographs are usually derived from smaller floods. Unit hydrograph peaking was not done for the 1979 IDF development effort. A new spillway adequacy study based on current criteria for development of the IDF and using current technology is being considered for separate submittal under the dam safety assurance program.

2.3 Clearwater Dam Pertinent Data

Pertinent data for the project are presented in Table C-1.

Table C-1
Clearwater Project Pertinent Data

Clearwater Dam:

Location: River Mile 257.4 on the Black River, Wayne
County, Missouri
Approximately 4.5 miles west of Piedmont, MO

Drainage Area: 898 square miles
One inch of runoff = 47,890 acre-feet

Dam:	Type: Earthen	
	Top of Dam, feet NGVD	608
	Streambed, feet NGVD	454
	Height, above streambed, feet	154
	Top of Parapet Wall, feet NGVD	611
	Crest Length, feet	4,225

Spillway:	Type: Side Saddle Uncontrolled	
	Crest Elevation, feet NGVD	567
	* Net Length, feet	470
	* Discharge @ Elevation 574 in CFS	22,750
	@ Elevation 602.5 in CFS	283,900
	* Design Discharge, CFS	353,000

Outlet Works:	Type: Concrete Lined Tunnel	
	Diameter, feet	23
	Gates (tractor type), number/size	3 / 9' X 20'
	Capacity @ Elevation 567 in CFS	25,000

Construction Dates:	Initiated:	1940
	Completed:	1948
	Modifications Initiated:	1988
	** Modifications Completed:	N/A

* With spillway excavated to design width
** Parapet wall, stability berm and dam seepage berm completed;
spillway widening effort stopped short of design width.

**Table C-1 (Cont.)
Clearwater Project Pertinent Data**

Clearwater Lake:

	Elev [ft, NGVD]	Lake Area [ac]	Lake Storage Capacity#			Spillway Capacity [cfs]
			Cumulative [ac-ft]	Runoff [in]	Incremental [ac-ft]	
Top of Parapet Wall	611	-	-	-	-	-
Top of Dam	608	-	-	-	-	-
Max. Design Water Surface##	607.7	18,485	1,008,150	21.05	594,450	353,000
Reservoir Easement Area Contour	572	11,210	469,840	-	-	-
Top of Flood Control Pool	567	10,400	413,700	8.64	391,780	-
Top of Conservation Pool###	494	1,630	21,920	0.46	21,920	-
Entrance Invert Elevation	467	-	-	-	-	-
Streambed at Dam	454	-	-	-	-	-

Prior to berm construction in 1989.

With spillway excavated to design width; at its current width the maximum probable water surface is about elevation 611.

Seasonal (May-Sep) top of conservation pool is EL 498 FT. In recent years a deviation to seasonal top of conversation pool EL 500 FT has been in effect.

2.4 Basin Description

The Black River drainage basin above Poplar Bluff, MO, lies in the foothills of the Ozark Mountains and consists of approximately 1246 sq-mi. 898 sq-mi lies above Clearwater Dam, and 348 sq-mi contributes flow to the Black River between Clearwater Dam and Poplar Bluff. The elongated basin is approximately 90 miles in length with width ranging from as little as about 5 miles in the lower end to as much as about 30 miles in the upper end. Elevations range from about 330 FT at Poplar Bluff to about 1300 FT in the upper reaches of the basin. Lateral tributaries to the Black River within this basin are numerous and generally of short length and steep grade.

2.5 Previous Studies

Previous hydrologic and hydraulic studies include:

- a) Dam Safety Hydrologic and Hydraulic Studies for Spillway Adequacy and Downstream Land Acquisition, August 1979. The spillway design flood (SDF) was developed in accordance with Corps of Engineers Engineering Circular (EC) 1110-2-163 (25 August 1975), Southwest Division letter SWDED-XW (15 February 1979), and National Weather Service (NWS) Hydrometeorological Report No. 51 (HMR-51) dated June 1978. Top of dam at the time of this study was EL 608.0 FT. With the top of dam artificially extended to contain the SDF, the maximum pool reached EL 618.6 FT, or 10.6 feet higher than the existing dam. The study concluded a spillway crest length of 540 feet was required to safely pass the SDF for top of dam EL 608.0 FT.
- b) Project Modification for the Dam Safety Assurance Program, Feature Design Memorandum No. 1, September 1983. This report concluded an additional 280 feet of spillway width (for a total crest length of 470 feet at EL 567.0 FT) was required to safely pass the SDF for effective top of dam EL 611.0 FT; top of earth embankment at EL 608.0 FT and top of concrete parapet wall at EL 611.0 FT to provide required freeboard.

2.6 Mapping

Mapping for the studied reach of the Black River was developed from digital U.S. Geological Survey (USGS) 1:24000 scale quadrangle maps. Map coordinates are referenced to Missouri State Plane Coordinate System (MO-SPCS), East Zone, North American Datum of 1983 (NAD83). The mapping (and study documentation) designates locations along the studied reach of the Black River by river mile, but due to differences in stationing techniques (historic paper/manual vs. modern digital) the river miles do not necessarily coincide with historic river mile designations for the Black River. The historically designated river mile for Clearwater Dam at RM 257.4 was assumed to represent the true distance along the Black River from its confluence with the White River near Newport in northeast Arkansas. The historic river mile designation at Clearwater Dam was used as the starting point for Black River stream stationing from Clearwater Dam downstream to just below Poplar Bluff, Missouri. In the hydraulic model, locations along the studied reach are referenced to both river mile, and stream stationing in feet. Stream stationing in feet may be divided by 5280 for direct conversion to corresponding river mile.

3. Hydrologic Analyses

3.1 Existing Condition and Seepage Dam Failure Scenario

A controlled lake release of 4000 cfs was used in conjunction with the pool at EL 505 FT for both existing condition analysis and as the antecedent condition for a dam breach with the pool at EL 505 FT. This is consistent with the reservoir regulation plan of operation for pool elevations in this low range. High pool elevation analyses performed for existing conditions and the seepage dam failure scenario were based on the Clearwater Lake IDF and estimated corresponding runoff for the 350 sq-mi of intervening drainage area between Clearwater Dam and a point just downstream of Poplar Bluff, Missouri. The IDF consists of an antecedent event, the standard project flood (SPF), followed five days by the probable maximum flood (PMF). The SPF effectively utilizes all available flood storage in the lake, thus the onset of the subsequent PMF occurs with the pool at approximately the top of the flood control pool. The analyses required modeling the PMF and a range of incrementally lesser magnitude flood events. Runoff hydrographs for the lesser magnitude flood events, or %PMF's, were derived directly from the Clearwater Lake IDF hydrograph and corresponding computed downstream runoff hydrographs by uniform, linear scaling of the respective PMF hydrograph ordinates to produce the "90% PMF" hydrographs, "80% PMF" hydrographs, etc. Each of the lesser magnitude events consists of the 100% SPF antecedent event followed by a percentage of the PMF event.

3.1.1 Clearwater Lake Inflow Design Flood

The Clearwater Lake IDF hydrograph was developed in 1979 (see 2.4, Previous Studies, part a)). The IDF has a peak discharge of 603,470 cfs with a runoff volume of 1,156,020 acre-feet (including base flow). The IDF hydrograph is shown on Plate C-1. Criteria for development of IDF's has changed since 1979 in that current guidance (ER 1110-8-2 (FR), Inflow Design Floods for Dams and Reservoirs, 1 March 1991) suggests reservoir inflow unit hydrographs for IDF determinations should be peaked 25 to 50 percent to account for the fact that unit hydrographs are usually derived from smaller floods. The Clearwater Lake IDF as developed in 1979 was assumed to be sufficiently accurate for purposes of assessing the adequacy of the dam for high pool elevations with respect to failure resulting from seepage.

3.1.2 Intervening Inflow below Clearwater Dam

A method of estimating runoff for the intervening drainage area between Clearwater Dam and Poplar Bluff, Missouri, which could be expected to correspond to the Clearwater Lake IDF and respective %PMF flood events of incrementally lesser magnitude, was needed. Storms that would produce very large inflow to Clearwater Lake may be expected to be widespread, resulting in significant runoff for the intervening drainage area. Significant intervening inflow occurring coincident with a Clearwater Lake flood release may be expected to affect flood heights and the speed with which a flood wave migrates down the valley. The intervening drainage area was divided into three sub-basins (Upper, Middle, and Lower). HEC-HMS version 2.1.3 (Oct 2001) was used to build a rainfall-runoff model used to produce a runoff hydrograph for each that might be expected to result from occurrence of the storm events from which the Clearwater Lake IDF was developed (Standard Project Storm (SPS) followed by the Spillway Design Storm (SDS)).

The Clearwater Lake IDF analysis included SPS and SDS hydrographs representing precipitation falling directly on the lake at flood pool elevation. This rainfall data was used directly as representative of rainfall over the intervening drainage area. This assumption is reasonably accurate for the Upper sub-basin due to its close proximity to Clearwater Lake, and is conservative for the more distant Middle and Lower sub-basins. The initial loss (0.50 inches) and infiltration loss rate (0.03 in/hr) used in the development of the Clearwater Lake IDF were used directly. Snyder unit hydrograph parameters for each of the sub-basins were estimated based on an existing analysis of the actual Snyder unit hydrograph parameters for known flood events at many gaged basins in the Little Rock District. Drainage areas were determined from USGS 1:24000 scale topographic maps. Unit hydrograph coefficients and basin characteristics are shown in Table C-2.

TABLE C-2 Unit Hydrograph Coefficients and Basin Characteristics for Drainage Area below Clearwater Dam								
Sub-Basin	D.A. [sq-mi]	L [mi]	Lca [mi]	Sst [ft/ft]	Ct	640Cp	Cp	Tp [hrs]
Upper	87	18.10	3.93	0.0027	1.3	350	0.547	4.67
Middle	146	31.61	16.38	0.0017	1.7	310	0.484	11.08
Lower	117	33.23	17.01	0.0004	4.3	300	0.469	28.78

The basin model used for computing the intervening inflow hydrographs was named "CW Dam to PB EX", contained in HEC-HMS project file "Clearwater Dam".

3.1.3 Clearwater Lake Flood Routings

HEC-HMS version 2.1.3 (Oct 2001) was used to build a reservoir model and perform the respective flood routings for existing conditions. The IDF and lesser magnitude %PMF events were routed through Clearwater Lake, resulting in lake release hydrographs for the respective events. HEC-HMS uses the Modified-Puls method to perform reservoir routings. Authorized top of conservation pool is EL 494.0 FT with authorized seasonal (summer months) adjustment to EL 498.0 FT. Top of seasonal conservation pool EL 500.0 FT has been in effect via approved deviation for at least six years and is expected to continue indefinitely. Therefore, the pool was conservatively assumed to be at EL 500.0 FT at the start of the routings. No conduit releases were made during the routings as concurrent flooding downstream likely would not permit such releases. Pool elevation-storage data was drawn from the Clearwater Lake Water Control Manual (1995). The reservoir model used for the existing condition routings was named "CW Res Only EX", contained in HEC-HMS project file "Clearwater Dam".

3.1.3.1 Existing Condition Routings

Existing condition routings used a pool elevation-discharge rating for the spillway based on existing conditions. The existing condition spillway discharge rating is shown on Plate C-2. Existing condition routing of the IDF resulted in a peak pool elevation of 611.2 FT (0.2 feet higher than the top of the parapet wall) with a corresponding peak spillway discharge of 315,000 cfs. Pool elevation remained above top of design pool EL 608.0 FT for 12 hours during which time pool elevation slightly exceeded top of parapet wall EL 611.0 FT for 3 hours. Overflow of the dam was ignored for purposes of this study. Summary results of the routings are shown in Table C-3.

TABLE C-3 Summary Results of Clearwater Lake Existing Condition Flood Routings (Started at Pool EL 500.0 FT)			
%PMF Event	Peak Inflow* [cfs]	Peak Outflow [cfs]	Peak Pool EL [ft]
100	603,000	315,000	611.2
90	543,000	280,000	608.1
80	483,000	247,000	604.9
70	423,000	215,000	601.8
60	362,000	185,000	598.6
50	302,000	152,000	594.9
40	242,000	118,000	590.8
30	182,000	82,800	586.3
20	121,000	51,800	581.4
10	61,000	33,700	578.1

* Peak inflow for the %PMF event following the full SPF which produced an antecedent peak inflow of 303,000 cfs.

3.1.3.2 Dam Breach Routings

HEC-1 version 4.1 (1998) was used to build a reservoir model and perform the dam breach analyses associated with the seepage dam failure scenario. HEC-HMS currently does not feature dam break analysis functionality. HEC-1 input files for the model runs are named "SDB20.DAT" (20% PMF event) thru "SDB100.DAT" (100% PMF event) and "EDB_505.DAT" (pool at EL 505.0 FT) with corresponding output files named "SDBXXX.OUT".

Geotechnical engineers (EC-DG) provided estimates of conditions expected to result in a seepage induced dam breach and the expected breach geometry. It was estimated a seepage related dam breach might occur as a result of low pool elevations of relatively long duration or high pool elevations of relatively short duration. Pool EL 505 FT is equaled or exceeded about 9.3% of the time on an average annual basis and was selected for the low pool elevation breach analysis. The Clearwater Lake pool elevation-duration curve is shown on Plate C-1.

For high pool elevations of relatively short duration it was estimated a dam breach would begin to form if the pool exceeded EL 575.0 FT, about the top of the seepage berm, for a total of 95 hours during the course of an event. Results of the existing condition flood routings indicated flood events equaling or exceeding approximately the 20% PMF event would produce this condition. Erosion to full estimated breach geometry was estimated to occur in one hour. Fully developed breach geometry was estimated as a trapezoidal shape with a 100 foot bottom

width at EL 478.0 FT and 1H:1V side slopes extending to the top of the earthen embankment at EL 608.0 FT. The pool elevation hydrograph resulting from the existing condition (without failure) flood routing was reviewed to determine the hour at which a breach was expected to begin forming, and the pool elevation at that hour. The HEC-1 dam break model was started at this hour and pool elevation. The model used the simulation start time to reference the applicable reservoir inflow hydrograph at the appropriate time step and used the inflow hydrograph from that point forward in the dam break simulation. All breaches started with the pool in recession.

Dam break analyses for the seepage dam failure scenario used the existing condition spillway rating. The dam breach simulation option in the HEC-1 software does not allow direct input of a user defined spillway rating curve. Uncontrolled spillways must be defined in terms of the standard weir equation and the program internally computes spillway outflow. The following values used in the dam break simulations were determined to produce a spillway rating very closely approximating the existing condition rating developed with the HEC-RAS backwater model: $C = 1.9365$, $L = 380$ FT, $H_{exp} = 1.5990$. Overtopping of the dam itself was not a factor for any of the dam breach simulations performed.

Summary results of the dam breach simulations for the seepage dam failure scenario are shown in Table C-4.

Event	Peak Outflow [cfs]	Pool EL at Start of Breach [ft]	Inflow at Start of Breach [cfs]	Outflow at Start of Breach [cfs]
100% PMF	827,000	593.0	6,880	134,000
80% PMF	723,000	587.5	3,640	92,300
60% PMF	643,000	582.7	1,700	60,200
40% PMF	578,000	578.5	898	36,300
20% PMF	523,000	574.7	732	19,400
Pool at EL 505.0 FT	51,000	505.0	700	4,000

3.1.4 Downstream Flood Routings

Routing of the Clearwater Lake outflow hydrograph and associated runoff hydrographs for the intervening drainage area, for each respective flood event analyzed, was performed in the unsteady flow hydraulic model of the Clearwater Dam to Poplar Bluff reach of the Black River.

3.2 Pool Elevation-Frequency

The current Clearwater Lake pool elevation-frequency curve is based on the results of SUPER Model W95X02 simulation of the 1940-1992 period of record inflows with the 1993 reservoir control plan of operation. Computed annual maximum pool elevations were plotted using the median plotting position formula (aka "Chegodayev's" formula) and used as a guide for the hand drawn curve. Confidence limits about this curve were developed using the HEC-FDA program which incorporates the procedures described in ETL 1110-2-537, "Uncertainty Estimates for Non-Analytic Frequency Curves" (31 October 1997). Summary results of the analysis are shown in Table C-5. A plot of the curve with its 68% (1 standard deviation) and 95% (2 standard deviations) confidence limits is shown on Plate C-10.

TABLE C-5
Clearwater Lake Pool Elevation-Frequency Data

Exceedance Probability	Pool EL [ft]	Std. Dev. [ft]	95% Lower Bound [ft]	68% Lower Bound [ft]	68% Upper Bound [ft]	95% Upper Bound [ft]
0.9999	497.0	3.2	490.6	493.8	500.2	503.4
0.9990	497.4	3.2	491.0	494.2	500.6	503.8
0.9900	497.9	3.2	491.5	494.7	501.1	504.3
0.9500	498.3	3.2	491.9	495.1	501.6	504.8
0.9200	498.5	3.2	492.1	495.3	501.7	504.9
0.9000	498.9	3.2	492.5	495.7	502.2	505.4
0.8700	499.5	3.2	493.1	496.3	502.7	505.9
0.8600	500.3	3.2	493.9	497.1	503.5	506.7
0.8000	505.2	3.2	498.8	502.0	508.4	511.6
0.7000	510.0	2.6	504.9	507.4	512.6	515.1
0.6000	513.2	2.4	508.5	510.8	515.6	517.9
0.5000	516.8	2.6	511.5	514.2	519.4	522.1
0.4000	521.0	2.9	515.1	518.1	523.9	526.9
0.3000	525.5	3.5	518.6	522.0	529.0	532.4
0.2000	532.1	4.5	523.2	527.6	536.6	541.0
0.1000	542.8	6.2	530.5	536.6	549.0	555.2
0.0500	552.2	7.4	537.4	544.8	559.6	567.0
0.0400	555.3	7.8	539.6	547.4	563.1	570.9
0.0200	564.0	9.0	546.1	555.0	573.0	582.0
0.0100	572.0	10.0	551.9	562.0	582.0	592.1
0.0040	581.6	11.3	559.0	570.3	592.9	604.2
0.0020	588.2	12.2	563.9	576.0	600.4	612.5
0.0010	594.4	13.0	568.4	581.4	607.4	620.4

4. Hydraulic Analysis

4.1 Black River Unsteady Flow Model

The HEC-RAS version 3.0.1 (March 2001) software was used to build an unsteady flow hydraulic model of the approximately 55 mile reach of the Black River extending from RM 257.27 (0.13 miles below Clearwater Dam) to RM 201.79 downstream of Poplar Bluff, Missouri. The model was developed for modeling high magnitude flood flows where overbank conveyance is dominant. Model results downstream of RM 205.98 (STA 1087585 in Figure 1) are considered unreliable due to the complex nature of the flow patterns in the several mile wide right bank floodplain.

All roadway embankments and levees in the study reach were assumed to survive overtopping; remaining intact at full height

for all flood flows modeled. Hydraulic model layout in the vicinity of Poplar Bluff is of particular interest and is shown below in Figure 1.

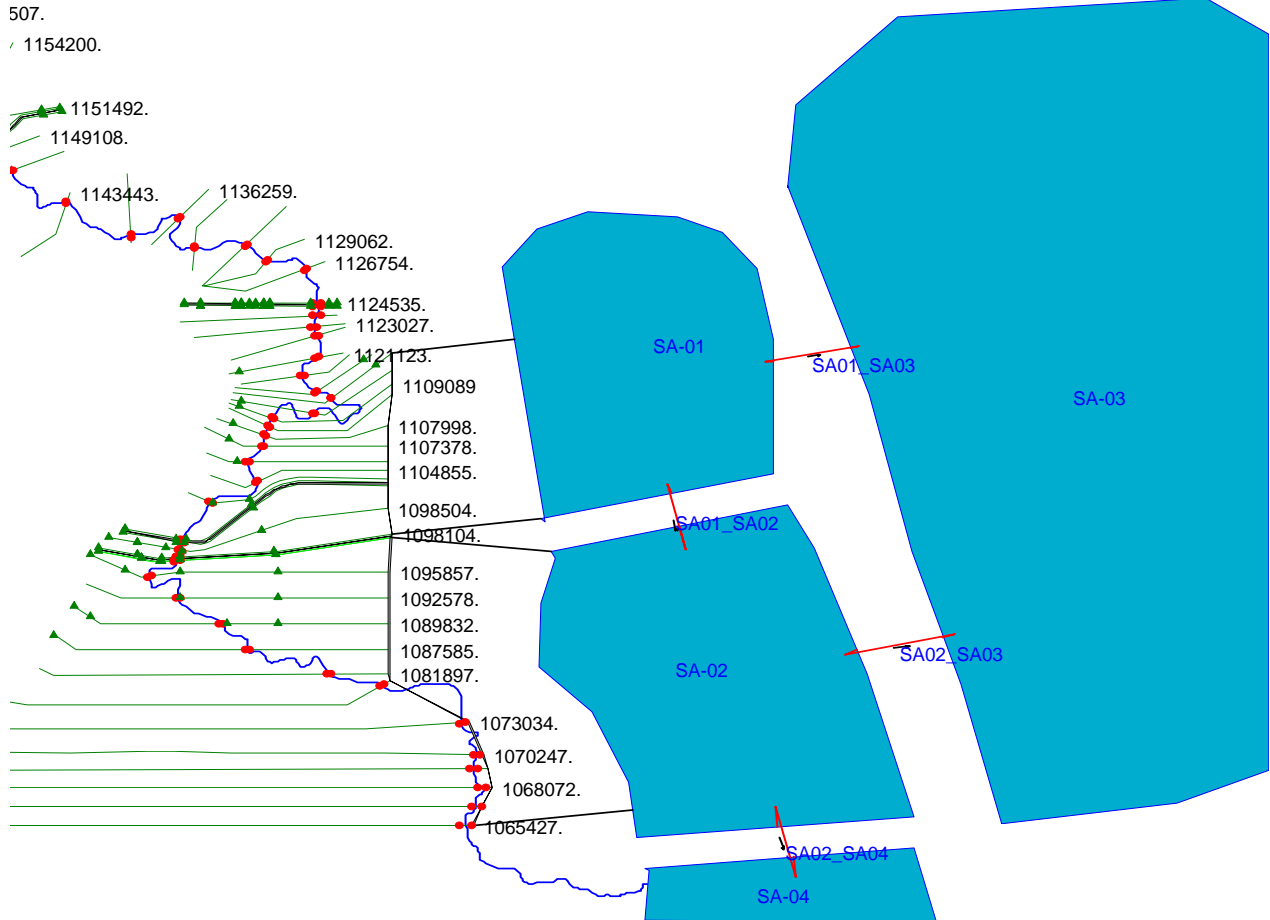


FIGURE C-1 - Hydraulic Model Layout at Poplar Bluff

The North Inter-River Levee located off the left bank of the Black River, in the vicinity of Poplar Bluff, was modeled as a lateral weir over which flow was allowed to pass into the floodplain between the North Inter-River Levee and the St. Francis River Levee. The inter-river floodplain upstream of the Missouri-Pacific (MO-PAC) railroad was modeled as Storage Area 1 (SA-01). The inter-river floodplain downstream of the MO-PAC railroad was modeled as Storage Area 2 (SA-02). Inter-River Levee overflow occurring upstream of the MO-PAC railroad was allowed to pass into SA-01, from which it could pass downstream over the MO-PAC railroad into SA-02 and/or out of the system over the St. Francis River Levee into "infinite" Storage Area 3 (SA-03). North Inter-River Levee overflow occurring downstream

of the MO-PAC railroad was allowed to pass into SA-02, from which it could pass out of the system over the St. Francis River Levee into "infinite" SA-03 and/or out of the system into infinite Storage Area 4 (SA-04). Levees and the railroad embankment separating the storage areas were modeled as broad crested weirs over which flow could pass in either direction. The boundary between SA-02 and SA-04 was modeled as a broad crested weir with $C=1.8$, based on the natural ground profile along the downstream boundary.

Model results for SA-01 and SA-02 represent estimates of the maximum ponded stage achieved and do not reflect the variation in water surface elevation which can be expected during the course of a flood event as flood waters pass through these areas.

The right bank levee and floodwall extending downstream from model RM 207.3 (historic RM 210.2) in south Poplar Bluff were not included in the hydraulic model since reasonable estimates of flood impacts to south Poplar Bluff resulting from large flood events could be made without expending the significant effort required to accurately model this hydraulically complex area. CESWL Report *Black River, Poplar Bluff, MO (Section 205) - Detailed Project Report and Environmental Assessment (November 1995)* concluded the portion of the structure above the downstream Poplar Bluff city limits can be expected to provide protection from the Black River for up to the 100-YR flood event. Below the Poplar Bluff city limits overtopping and/or breaching of the levee may occur for the 100-YR or lesser magnitude flood events, resulting in backwater flooding of south Poplar Bluff. The report defined the 100-YR total discharge as 48,200 cfs at Business Highway 60. Of the total discharge 14,000 cfs is expected to pass down the Black River with the remaining 34,200 cfs expected to pass through the relief reach bounded on the west by the Butler County Drainage District No. 12 Levee in east Poplar Bluff, and on the east by the Inter-River Levee. Based on the results of the analyses presented in the November 1995 report, south Poplar Bluff is assumed to be impacted by flood events for which the peak discharge at Business Highway 60 exceeds 48,200 cfs.

Model results below model RM 205.98 (historic RM 208.90) should not be considered reliable. The model was extended downstream from this location only to provide a downstream boundary for the model computations.

4.1.1 Existing Conveyance System

The Black River flows generally southeastward through the study reach. The main channel varies from about 100 FT to 500 FT in width through most of the study reach with an average slope of about 2.8 ft/mi. Main channel slope varies from as steep as about 3.8 ft/mi in the upper end of the study reach to as little as about 1 ft/mi in the lower end of the study reach in the vicinity of Poplar Bluff. The main channel is generally clear and clean with tree lined banks through most of the study reach. A Manning's roughness coefficient of 0.03 was used in the hydraulic model for the main channel throughout the study reach.

Overbank conveyance characteristics vary widely through the study reach. Upstream of the fall line at about RM 212 overbanks vary in width from as little as about 100 feet to as wide as about 5000 FT between the steeply rising sides of the valley. Downstream of the fall line the Black River passes into the relatively flat historic floodplain of the Mississippi River and overbank widths stretch to several miles when considering high magnitude flood flows. Overbank cover generally varies from open pasture and cultivated farmland to areas of forest with medium to thick undergrowth. Manning's roughness coefficients varying from 0.05 to 0.12 were used in the hydraulic model for the overbanks.

4.1.2 Model Geometry

4.1.2.1 Valley Cross-Sections

Valley cross-section geometry was developed from U.S. Geological Survey (USGS) 30 meter grid digital elevation models (DEM's). Accuracy of the thirty meter DEM's are comparable to the corresponding 1:24000 scale, 20-foot contour interval, USGS topographic maps from which they are developed, but may exhibit less accuracy in flat areas.

4.1.2.2 Channel Cross-Sections

Surveyed channel cross-sections at selected locations along the portion of the modeled reach extending from about RM 252 downstream to about RM 218 were collected in 1981. The survey data is contained in CESWL survey field books 81W2-1, 81W2-2, and 81W2-5. This data was used directly where applicable, and used as guidance for estimating channel geometry at other locations in the reach.

4.1.2.3 Road Crossings

Twelve road crossings were identified in the study reach. Design drawings for eight of the twelve crossings were obtained from the Missouri Department of Transportation (MDOT) and used as the basis for incorporating those crossings into the model. Drawings were not readily available for the Missouri-Pacific (MO-PAC) railroad crossing at RM 215.48 and it was not included in the model. A county road crossing located approximately one mile south of Williamsville at RM 232.62 was not included in the model because the absence of a significant roadway embankment across the mile wide floodplain indicated inclusion of the crossing would have little impact on results of the hydraulic model. Similar judgment was used to exclude the Bartlett Street crossing located at RM 208.1 just downstream of U.S. Business 60 in Poplar Bluff. Geometry for the MO-PAC railroad crossing at RM 207.96 in Poplar Bluff was estimated from visual observation, 1:24000 USGS quad maps, and levee design drawings.

4.1.2.4 Levees

Top of levee elevations were drawn from levee design drawings. Elevations for the levee in east Poplar Bluff were drawn from "Butler County D.D. NO. 12, East Poplar Bluff, MO", drawing number 13/35 (L-4), dated January 1960, revised June 1991. Elevations for the Inter-River Levee located about 2 miles east of Poplar Bluff were drawn from "North Inter-River Drainage District and the Ring Levee Drainage District", drawing 13/36 (L-5A), dated January 1940, revised September 1989. Both drawings were found in CESWL's "Natural Disaster Response Plan", OM 500-1-1.

Top of levee elevations for the St. Francis River right bank levee were drawn from a design drawing supplied by the Memphis District, dated 1961-1962.

4.1.2.5 Storage Areas

Elevation-storage curves for SA-01 and SA-02 were developed from USGS 30 meter DEM data.

4.2 Existing Condition Discharge Rating

HEC-RAS version 3.0.1 (March 2001) was used to build a steady state backwater model of the spillway reach from which the existing condition pool elevation-discharge rating was developed. Model geometry for the concrete ogee weir structure, and for the portion of the cross-sections extending from the original spillway alignment centerline toward the left bank was based on Clearwater Dam original design drawing "Spillway Plan and Section", January 1946. Geometry for the portion of the cross-sections extending from the original spillway alignment centerline toward the right bank were based on post-construction field survey data collected in 1989. Beyond the limits of these data sets cross-section geometry was estimated from USGS 1:24000 scale topographic maps.

Model cross-sections were sub-divided into left overbank, main channel, and right overbank for assignment of Manning's roughness coefficients due to the irregularity of the right bank and the presence of trees on both banks. The main channel was defined as extending from toe of slope to toe of slope. Some moderate scour/erosion of the main channel bed during the course of a flood event was assumed. N-values for the main channel ranged from 0.025 (in Clearwater Lake) to 0.05, with 0.04 used through most of the reach. An n-value of 0.04 is commensurate with that for a mountain stream with bed composed of gravels, cobbles, and a few boulders [Chow, 1959]. Overbank n-values ranged up to 0.07. Expansion coefficients ranged from 0.3 to 0.5, and contraction coefficients ranged from 0.1 to 0.3. N-values and/or expansion and contraction coefficients were increased in areas where significant turbulence could be expected and/or sudden changes in velocity were observed in the model.

The model extended from downstream of the lower end of the spillway, just below Highway H-H, to a point in Clearwater Lake about 1000 feet beyond the upstream end of the spillway approach channel. Steady state profiles were computed for flows ranging from 1000 cfs to 450,000 cfs. The resulting existing condition pool elevation-discharge rating table is shown in Table C-6. The plotted curve is shown on Plate C-3.

TABLE C-6
Existing Condition Spillway Pool-Elevation Discharge Rating

Spillway Discharge [cfs]	Pool EL [ft]
0	567.0
1000	568.1
2000	568.8
4000	569.9
6000	570.9
10000	572.3
15000	573.8
20000	575.1
30000	577.4
40000	579.4
60000	582.8
80000	585.9
100000	588.6
140000	593.5
180000	598.1
220000	602.3
260000	606.2
300000	609.9
350000	614.2
400000	618.4
450000	622.4

5. Results of Analyses

Maximum increases in flood heights for failure verses non-failure, for each of the three dam breach analyses for which downstream hydraulic models were developed are shown in Table C-7. Graphs of the data are shown on Plates C-8 and C-9. The increases in with failure verses without failure flood heights for the 100% PMF are smaller than for the 20% PMF because uncontrolled spillway releases preceding the dam breach are larger for the 100% PMF and the increase in flow capacity per incremental increase in depth of flow for the river is greater for higher stages.

TABLE C-7

Maximum Increase in Flood Heights [FT]: Dam Failure vs. Existing Condition W/O Failure

Distance From Dam [mi]	Location Description	Dam Breach w/ Pool At EL 505 FT	Dam Breach w/ Pool At EL 574.7 FT (20% PMF)	Dam Breach w/ Pool At EL 593.0 FT (100% PMF)
0.8	Vicinity of Dam	7.1	24.1	16.5
4.9	Piedmont/McKenzie Creek	10.3	27.7	15.5
7.0	Hwy 34	9.8	21.7	12.2
7.8	Leeper	9.8	19.5	10.8
9.5	Hwy 49 at Mill Springs	9.2	20.8	11.4
19.2	Hwy 49 at Browns Crossing	10.5	12.2	7.2
22.5	Williamsville	8.7	18.6	7.9
30.0	Hwy 67 near Hendrickson	12.1	20.5	6.2
31.0	Hendrickson	11.0	17.3	5.9
39.6	Hwy W at Hilliard	9.7	13.6	4.3
44.4	Hwy 60 Bypass near Poplar Bluff	7.7	9.8	2.2
45.7	Begin Inter-River Levee at Poplar Bluff (EL +/- 343 FT)	7.3	5.0	1.6
49.2	Bus. Hwy 60 at Poplar Bluff	9.0	4.9	1.1
49.4	MO-PAC RR at Poplar Bluff	8.1	3.4	0.7
51.0	South Poplar Bluff	7.3	1.0	0.8
	Pond Area 1 - Northeast of Poplar Bluff	0.0	5.4	0.6
	Pond Area 2 - East of Poplar Bluff	0.0	4.7	0.6

5.1 High Pool Seepage Dam Failure Scenario

The 20% PMF was determined to be the threshold event for high pool seepage dam failure based on resident time (about 95 hours) of pool elevation exceeding EL 575.0 FT. Peak discharge for the 20% PMF flood event without failure is about 51,800 cfs, versus a with failure peak discharge of about 523,000 cfs. Failure occurred at pool EL 574.7 FT with the pool in recession from peak pool EL 581.4 FT. Downstream maximum water surface elevations and flood wave travel times for the 20% PMF flood event are shown in Table C-8. Peak discharge for the 100% PMF event without failure is about 315,000 cfs, versus a with failure peak discharge of about 827,000 cfs. Failure occurred at pool EL 593.0 FT with the pool in recession from peak pool EL 611.2 FT. Downstream maximum water surface elevations and flood wave travel times for the 100% PMF event are shown in Table C-9. Dam breach outflow hydrographs for the 20% PMF and the 100% PMF events are shown on Plate C-4. Water surface profiles for the 20% PMF and the 100% PMF events are shown on Plate C-5.

TABLE C-8
 20% PMF Seepage Failure Dam Breach Simulation
 Peak Flood Elevations and Times to Peak at Locations below Clearwater Dam

Distance From Dam [mi]	Location	Without Failure			With Failure		
		Elapsed Time ¹ [hrs]	Peak WSEL [ft,NGVD]	Peak Q [cfs]	Elapsed Time ² [hrs]	Peak WSEL [ft,NGVD]	Peak Q [cfs]
0.8	Vicinity of Dam	44.5	468.0	52,724	3.6	492.1	494,205
4.9	Piedmont/McKenzie Creek	45.0	456.6	54,166	3.9	484.3	414,436
7.0	Hwy 34	45.5	445.9	54,111	4.5	467.6	411,843
7.8	Leeper	45.5	442.9	54,092	4.8	462.4	410,303
9.5	Hwy 49 at Mill Springs	46.5	435.1	54,012	5.5	455.9	401,039
19.2	Hwy 49 at Browns Crossing	47.5	403.7	69,472	8.6	415.9	368,265
22.5	Williamsville	49.0	390.1	67,974	9.7	408.7	351,977
30.0	Hwy 67 near Hendrickson	52.0	369.9	62,905	10.7	390.4	337,011
31.0	Hendrickson	52.0	367.3	62,571	11.6	384.6	335,953
39.6	Hwy W at Hilliard	55.0	354.4	69,058	14.3	368.0	300,976
44.4	Hwy 60 Bypass near Poplar Bluff	56.5	345.3	66,436	14.9	355.1	298,002
45.7	Begin Inter-River Levee at Poplar Bluff (EL +/-343 FT)	56.5	343.9	64,810	15.2	348.9	297,581
49.2	Bus. Hwy 60 at Poplar Bluff	56.5	342.2	56,436	15.4	347.1	89,102
49.4	MO-PAC RR at Poplar Bluff	57.5	337.4	56,422	15.9	340.8	78,789
51.0	South Poplar Bluff	58.5	332.5	56,389	17.4	333.5	78,277
	Pond Area 1 - Northeast of Poplar Bluff	66.0	323.2	8,196*	20.0	328.6	218,213*
	Pond Area 2 - East of Poplar Bluff	66.0	315.7	3,508*	22.0	320.4	174,839*

1 - From beginning of PMF spillway release to time of maximum water surface elevation.

2 - From beginning of dam breach to time of maximum water surface elevation.

* Peak inflow to pond area. Inflow to Pond Area 1 is from Black River overtopping Inter-River Levee. Inflow to Pond Area 2 is from Pond Area 1 across MO-PAC RR.

TABLE C-9
 100% PMF Seepage Failure Dam Breach Simulation
 Peak Flood Elevations and Times to Peak at Locations below Clearwater Dam

Distance From Dam [mi]	Location	Without Failure			With Failure		
		Elapsed Time ¹ [hrs]	Peak WSEL [ft,NGVD]	Peak Q [cfs]	Elapsed Time ² [hrs]	Peak WSEL [ft,NGVD]	Peak Q [cfs]
0.8	Vicinity of Dam	49.5	487.1	315,684	2.9	503.6	788,792
4.9	Piedmont/McKenzie Creek	49.5	479.7	326,748	3.1	495.2	662,977
7.0	Hwy 34	50.0	464.2	326,528	3.6	476.4	657,808
7.8	Leeper	50.0	459.6	326,391	4.0	470.4	654,793
9.5	Hwy 49 at Mill Springs	50.5	453.5	325,963	4.5	464.9	636,864
19.2	Hwy 49 at Browns Crossing	51.0	416.9	374,947	7.0	424.1	591,638
22.5	Williamsville	51.5	410.3	373,639	7.5	418.2	567,316
30.0	Hwy 67 near Hendrickson	53.0	392.2	371,183	8.8	398.4	541,320
31.0	Hendrickson	53.5	387.1	370,981	9.2	393.0	539,701
39.6	Hwy W at Hilliard	55.0	371.6	394,848	10.8	375.9	528,658
44.4	Hwy 60 Bypass near Poplar Bluff	55.5	357.1	394,329	11.2	359.3	526,521
45.7	Begin Inter-River Levee at Poplar Bluff (EL +/-343 FT)	55.5	350.4	394,241	11.3	352.0	526,406
49.2	Bus. Hwy 60 at Poplar Bluff	55.5	348.5	88,128	11.1	349.6	119,998
49.4	MO-PAC RR at Poplar Bluff	56.0	341.0	79,841	11.6	341.7	101,919
51.0	South Poplar Bluff	56.5	333.5	79,835	12.4	334.3	100,345
	Pond Area 1 - Northeast of Poplar Bluff	54.0	329.7	314,424*	14.0	330.3	424,455*
	Pond Area 2 - East of Poplar Bluff	57.0	321.7	311,532*	15.0	322.3	411,672*

1 - From beginning of PMF spillway release to time of maximum water surface elevation.

2 - From beginning of dam breach to time of maximum water surface elevation.

* Peak inflow to pond area. Inflow to Pond Area 1 is from Black River overtopping Inter-River Levee. Inflow to Pond Area 2 is from Pond Area 1 across MO-PAC RR.

5.2 Low Pool Seepage Dam Failure Scenario

Relatively long duration low pool elevations equaling or exceeding about EL 505.0 FT may result in a seepage related dam breach. Pool elevations below about EL 567.0 FT, the emergency spillway crest elevation, can be expected to result in a Clearwater Lake maximum release of about 4000 cfs as the lake is drawn down to normal pool elevation with a corresponding flow rate of about 4500 cfs at Poplar Bluff. The actual rate of release is determined by downstream conditions and controlled to minimize downstream flooding. A flow rate of 4500 cfs was used throughout the downstream reach for the without failure water surface profile. Peak discharge for the pool at elevation 505.0 FT with failure is about 51,000 cfs. Downstream maximum water surface elevations and flood wave travel times are shown in Table C-10. The dam breach outflow hydrograph is shown on Plate C-6. Water surface profiles are shown on Plate C-7.

TABLE C-10
Pool at EL 505.0 FT Seepage Failure Dam Breach Simulation
Peak Flood Elevations and Times to Peak at Locations below Clearwater Dam

Distance From Dam [mi]	Location	Without Failure			With Failure		
			WSEL [ft,NGVD]	Q [cfs]	Elapsed Time ¹ [hrs]	Peak WSEL [ft,NGVD]	Peak Q [cfs]
0.8	Vicinity of Dam	-	459.9	4,500	2.8	467.0	45,134
4.9	Piedmont/McKenzie Creek	-	443.6	4,500	5.1	453.9	38,293
7.0	Hwy 34	-	434.0	4,500	6.1	443.8	37,449
7.8	Leeper	-	431.1	4,500	6.4	440.9	36,966
9.5	Hwy 49 at Mill Springs	-	423.4	4,500	7.6	432.6	36,096
19.2	Hwy 49 at Browns Crossing	-	388.2	4,500	13.1	398.7	29,953
22.5	Williamsville	-	377.1	4,500	15.5	385.8	29,003
30.0	Hwy 67 near Hendrickson	-	351.4	4,500	19.9	363.5	26,396
31.0	Hendrickson	-	348.7	4,500	20.2	359.7	26,292
39.6	Hwy W at Hilliard	-	337.3	4,500	26.3	347.0	22,157
44.4	Hwy 60 Bypass near Poplar Bluff	-	331.5	4,500	29.4	339.2	21,129
45.7	Begin Inter-River Levee at Poplar Bluff (EL +/-343 FT)	-	330.6	4,500	30.2	337.9	20,973
49.2	Bus. Hwy 60 at Poplar Bluff	-	324.8	4,500	31.9	333.8	20,325
49.4	MO-PAC RR at Poplar Bluff	-	323.8	4,500	32.3	331.9	20,264
51.0	South Poplar Bluff	-	321.4	4,500	34.4	328.6	19,765
	Pond Area 1 - Northeast of Poplar Bluff	-	N/A	N/A	N/A	N/A	N/A
	Pond Area 2 - East of Poplar Bluff	-	N/A	N/A	N/A	N/A	N/A

1 - From beginning of dam breach to time of maximum water surface elevation. Not applicable for without failure controlled release.

N/A - Black River did not overtop the Inter-River Levee.

6. References

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3. Clearwater Dam, Black River, MO - Project Modification for the Dam Safety Assurance Program Reconnaissance Report; Revised May 1986; CESWL.
4. Dam Safety Assurance Program; 12 September 1997; ER 1110-2-1155; USACE.
5. Inflow Design Floods for Dams and Reservoirs; 1 March 1991; ER 1110-8-2(FR); USACE.
6. HEC-RAS River Analysis System Hydraulic Reference Manual; Version 3.0, January 2001; CPD-69; USACE Hydrologic Engineering Center.
7. Duration and Frequency of Hydrologic Events in the Little Rock District; January 1999 (Rev April 2000); CESWL-EC-HH.
8. HEC-1 Flood Hydrograph Package User's Manual; June 1998; CPD-1A (Version 4.1); USACE Hydrologic Engineering Center.
9. Black River, Poplar Bluff, MO - Section 205 - Detailed Project Report and Environmental Assessment; November 1995; CESWL.