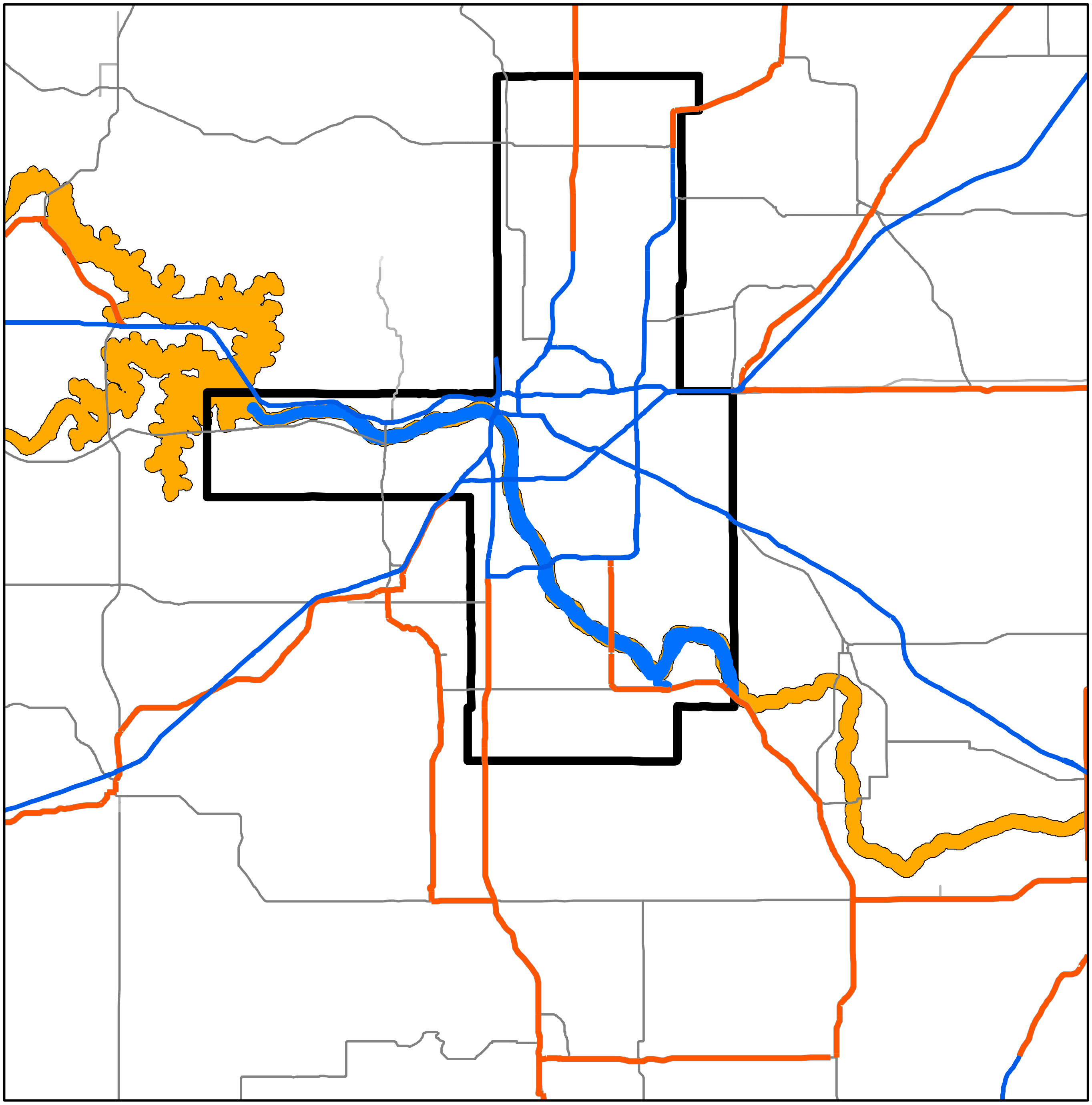
ARKANSAS RIVER CORRIDOR STUDY
MASTER PLAN
HYDROLOGIC & HYDRAULIC TECHNICAL MEMORANDUM



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MESHEK & ASSOCIATES, PLC

CIVIL & WATER RESOURCE ENGINEERING
GEOGRAPHIC INFORMATION SYSTEMS

Table of Contents

ARKANSAS RIVER CORRIDOR MASTER PLAN
HYDROLOGIC AND HYDRAULIC ANALYSES
TECHNICAL MEMORANDUM

Meshek & Associates, PLC

Introduction

The Arkansas River begins near Leadville, Colorado and flows generally southeast to its confluence with the Mississippi River near Greenville, Mississippi. The Arkansas River drains approximately 75,700 square miles upstream of the Tulsa, Oklahoma vicinity, of which nearly 50,000 square miles actually contribute to flows at Tulsa. The basin upstream of Tulsa is about 650 miles long and averages 150 miles wide. An overview of the Arkansas River basin above Tulsa is illustrated in Figure 1.

The Arkansas River Corridor Master Plan study includes the hydrologic and hydraulic analyses of the impacts of 2 proposed low water dams on the Arkansas River at Sand Springs and Jenks, Oklahoma, and modification to the existing Zink low water dam near 28th Street in Tulsa. The portion of the Arkansas River covered in this study includes approximately 81 river miles and flows through 3 counties. A map of the Arkansas River Corridor Master Plan study area is shown in Figure 2. The locations of the proposed and existing low water dams are shown in Figure 3.

The proposed low water dam at Jenks would have a pool set for elevation 596.0 and would inundate an area of about 502 acres. The static pool would be nearly 4 miles long and would average 4-6 feet in depth.

The existing Zink low water dam would be modified or re-constructed to have a pool raised from its existing elevation of 617.0 to an elevation of 620.0. The static pool would cover an area of around 580 acres, would be about 4.2 miles long, and would average 4 feet in depth.

The proposed Sand Springs low water dam would have a pool that would vary from 634 to 638 and would be used as a temporary storage facility to modulate flows downstream of the dam. At elevation 638.0, the static pool would cover an area of close to 1,420 acres, would be about 8.7 miles long, and would average 4-7 feet in depth.

FIGURE 1. ARKANSAS RIVER WATERSHED ABOVE STUDY AREA

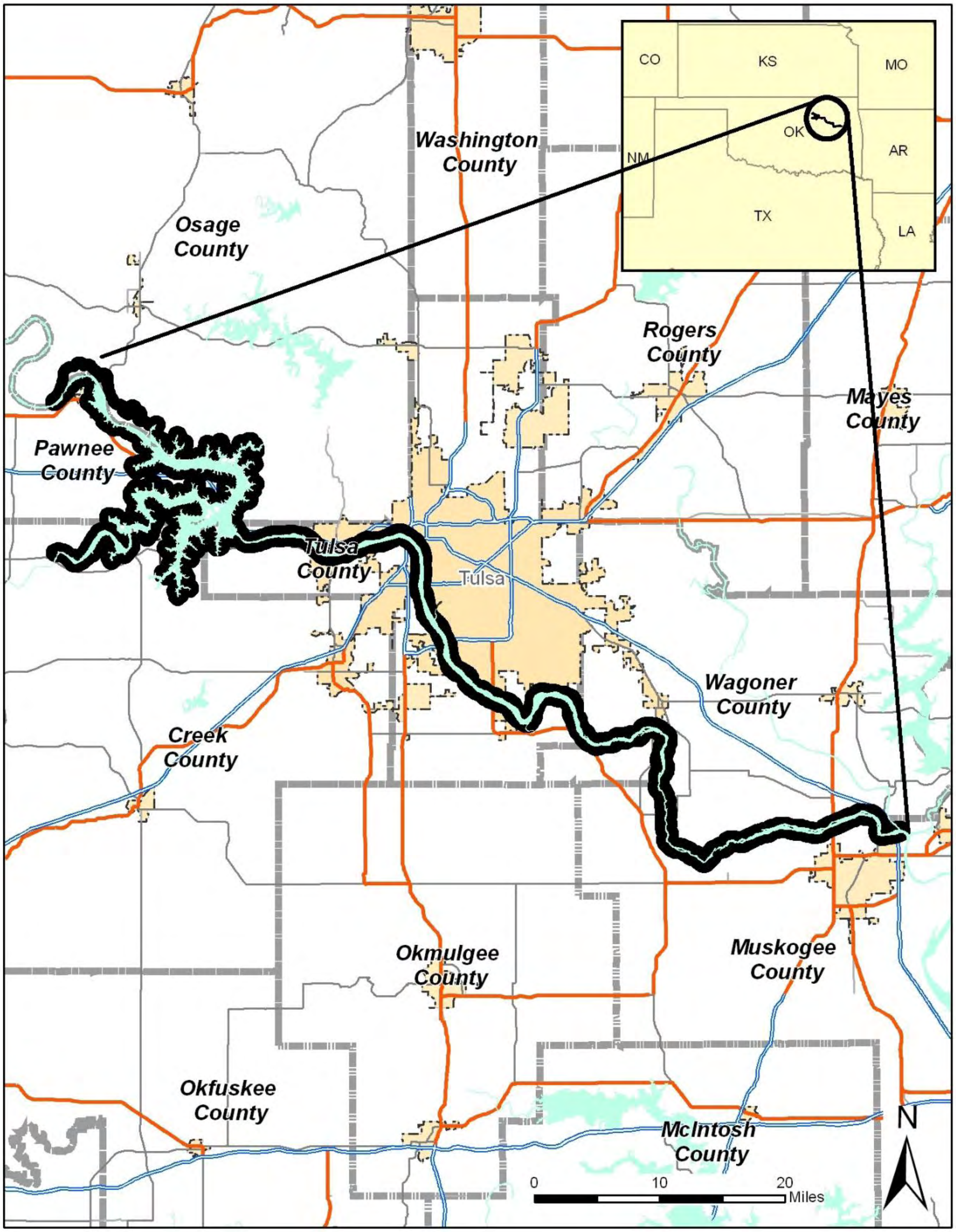


FIGURE 2. ARKANSAS RIVER CORRIDOR MASTER PLAN STUDY AREA

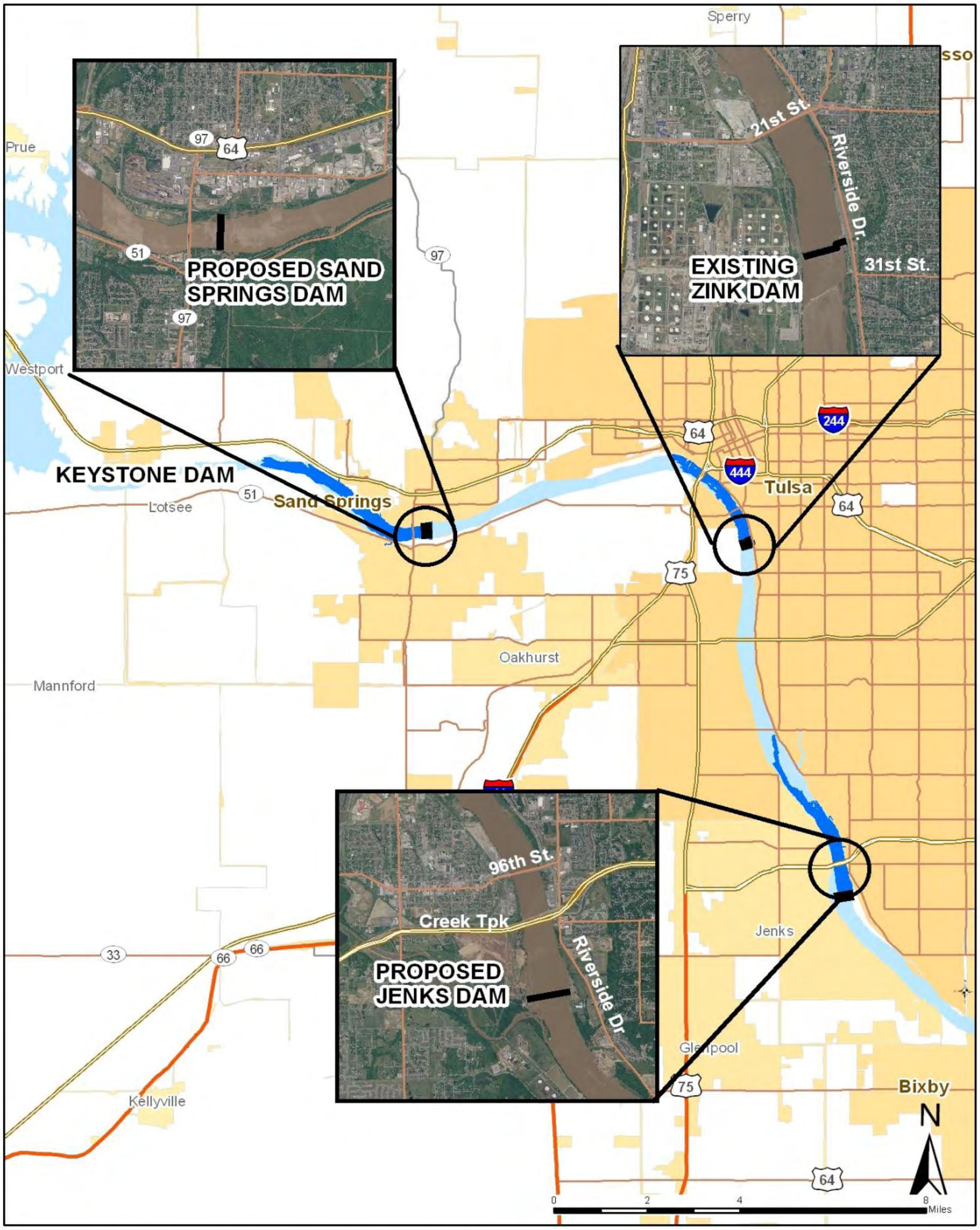


FIGURE 3. LOCATIONS OF LOW WATER DAMS

HYDROLOGIC MODELING

Existing Corps of Engineers Hydrologic Models

HEC-HMS and HEC-1 computer models were obtained from the Tulsa District, US Army Corps of Engineers for various tributary basins that flow into the Arkansas River in Tulsa County. Those models were developed and furnished by Scott Henderson, P.E. of the Hydrology-Hydraulics Branch as part of the Tulsa County Watershed Study prepared by the Corps in 2002. The basin models obtained from the Corps are listed in Table 1. Along with the HMS models, watershed outlines and stream centerline data were also obtained in GIS format. Since the flood of record occurred in October of 1986, the models were re-run with changes to the Control Specifications option so as to have starting and ending times of 04Oct86, 12:00 and 12Oct86, 22:00 respectively. Output hydrographs were created in the HEC-DSS (Data Storage System) and then added to the “master” DSS file **Tul_Ark_2009.dss.** Figure 4 shows the locations of the basins studied by the Corps and used in this study.

Existing City of Tulsa Hydrologic Models

HEC-1 and HMS computer models were obtained from Mr. Bill Robison, P.E. of the City of Tulsa for several tributary basins that flow into the Arkansas River, but only 3 of those basin models were required for use in this study. The models (previously developed for flood insurance studies or master drainage plans) obtained from the City and used for the Arkansas River Corridor Study are listed in Table 1. The HEC-1 and HMS models were re-run with changes only to the starting and ending computation dates to correspond to the 4-12 October 1986 timeframe. Output DSS paths for the mouth of the streams were created and then added to the “master” DSS file **Tul_Ark_2009.dss.** The 3 City of Tulsa basin models used in this study are shown in Figure 4.

Previously Developed Meshek Models

Meshek & Associates, PLC had previously developed HMS watershed models for 11 streams that flow into the Arkansas River. The models were re-run with changes only to the Control Specifications to have starting and ending times of 04Oct86, 12:00 and 12Oct86, 22:00 respectively. Output DSS paths were added to the “master” DSS file **Tul_Ark_2009.dss.** The 11 Meshek basins used in this study are shown in Figure 4.

TABLE 1
INTERVENING STREAMS AND HYDROLOGIC MODELS

STREAM NAME	SOURCE	ARKANSAS RIVER STATON In feet above start of study	MODEL TYPE	COMPUTATION INTERVAL (minutes)
Brush Creek	Meshek (New)	427549	HMS	1
RB1a	Meshek (New)	427545	HMS	1
RB1	Meshek (New)	425169	HMS	1
Little Sand Creek	Corps of Engr	423127	HMS	1
Sand Creek	Meshek	420000	HMS	1
LB1	Meshek (New)	413827	HMS	1
Mud Creek	Meshek (New)	411589	HMS	1
LB2	Meshek (New)	410667	HMS	1
Shell Creek	Corps of Engr	403112	HMS	1
Euchee Creek	Corps of Engr	395476	HMS	1
Franklin Creek	Corps of Engr	389855	HMS	1
Fisher-Anderson Creeks	Corps of Engr	386127	HMS	5
Main St.	Meshek	383534	HMS	1
SAND SPRINGS DAM		**382170**		
Pratt Creek	Meshek	381842	HMS	1
RB2	Meshek (New)	381420	HMS	1
Squirrel Hollow	Meshek	380047	HMS	1
Redfork Creek	Meshek	377829	HMS	1
Pecan Woodland	Meshek	375070	HMS	1
Big Heart Creek	Corps of Engr	369362	HMS	1
Berryhill Creek	Corps of Engr	365659	HMS	1
Parkview Creek	City of Tulsa	352698	HEC-1	1
Oak Creek	Meshek (New)	350779	HMS	1
Downtown Area	Meshek (New)	346270	HMS	1
Elm Creek	Swift	343801	HMS	1
Swan Creek	City of Tulsa	341813	HMS	10
ZINK DAM		**339440**		
Crow Creek	City of Tulsa	337952	HMS	10
Cherry-Redfork Creeks	Corps and Meshek	330033	HMS	1
Mooser Creek	Swift	326058	HEC-1	1
Perryman Ditch	Swift	322620	HEC-1	1
Joe Creek	Corps of Engr	311317	HEC-1	10
Fred Creek	Swift	307811	HMS	1
RL Jones AP	Meshek	306246	HMS	5
Vensel Creek	Swift	300184	HEC-1	1
JENKS DAM		**297318**		
Polecat Creek	Meshek	295373	HMS	5

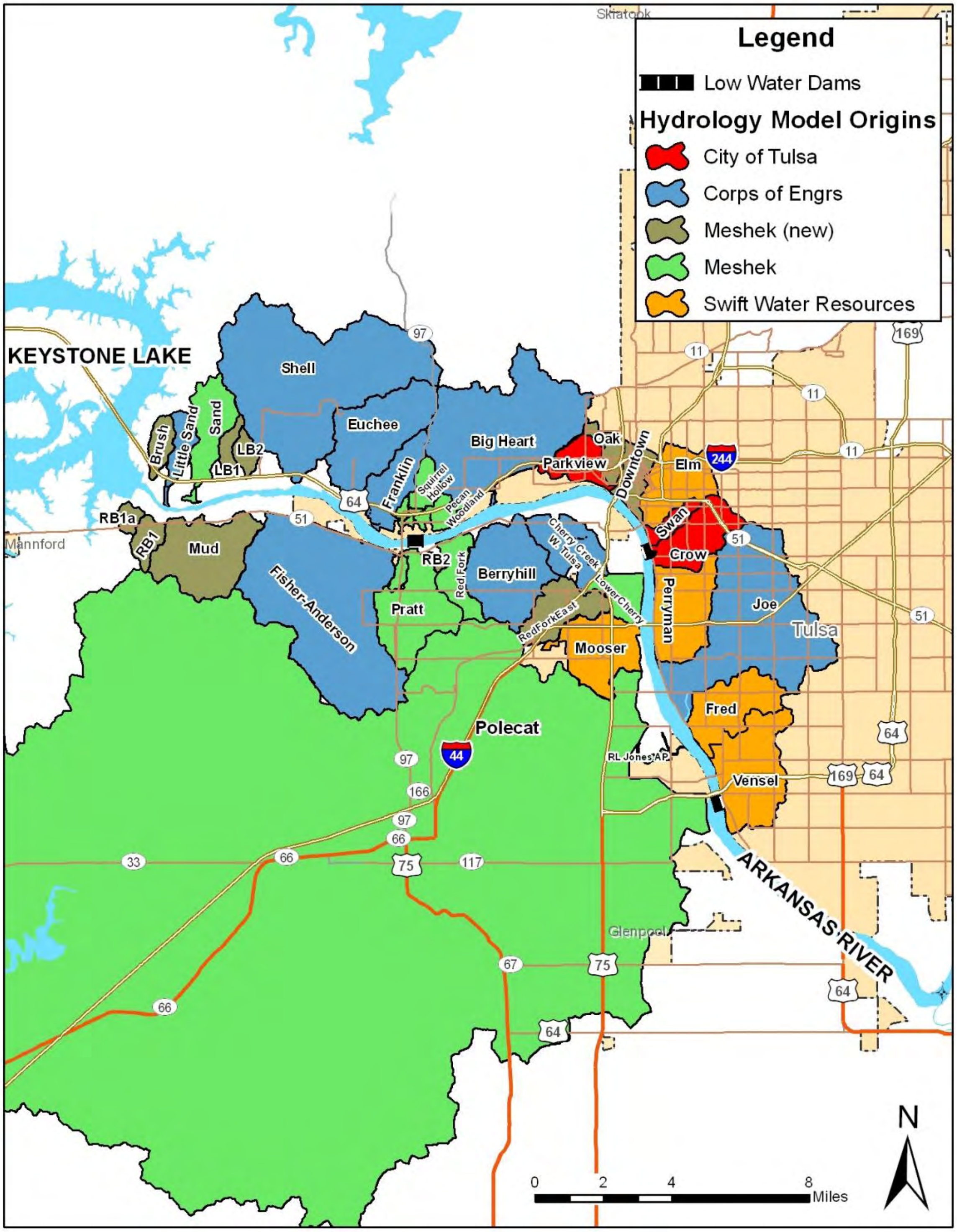


FIGURE 4. TRIBUTARY WATERSHEDS MODELED IN STUDY

Existing Swift Water Resources Models

Swift Water Resources of Tulsa had previously developed HMS and HEC-1 models for 5 streams that flow into the Arkansas River. The models, developed as part of previous flood insurance studies or master drainage plan studies for the City of Tulsa, were re-run with changes only to the Control Specifications to have starting and ending times of 04Oct86, 12:00 and 12Oct86, 22:00,respectively. Output DSS paths for these models were added to the "master" DSS file **Tul_Ark_2009.dss.** Figure 4 shows the locations of the basin models developed by Swift Water Resources.

New Meshek Models

Meshek developed single basin HMS models for an additional 10 streams that flow into the Arkansas River. Those models were developed using elevation data obtained from the National Elevation Dataset (NED) and have 1 minute computation intervals. The models used the same starting and ending dates and times as the previously described models. Output DSS paths for these models were added to the "master" DSS file **Tul_Ark_2009.dss.** The locations of the basins for the new Meshek HMS models are shown in Figure 4.

Hydrologic Studies Results

The peak 100-year discharges for the tributary streams at their confluence with the Arkansas River are shown in Table 2. The start date and time for the hydrologic models was set as 12:00, October 4th. The 3 columns on the right side of the table will be explained later in this report.

Previous Arkansas River Hydrology

The Tulsa District US Army Corps of Engineers developed discharge frequency relationships for the Arkansas River in Tulsa as part of the 1980 Tulsa County Flood Insurance Study, prepared for FEMA. The 1% chance (100-year) flood discharge developed in that study was 170,000 cubic feet per second (cfs). Then, in 2002, the Tulsa District developed a discharge frequency reservoir release curve for Keystone Dam outflows during the 2002 Tulsa County Watershed Study. That curve is shown in Figure 5. The 100-year flood peak discharge developed in the 2002 study is now 205,000 cfs. The Corps states that the increase is due to an additional 27 years of record, in which 3 significant floods have occurred. Additionally, observed stage and flow hydrographs were obtained from the Corps of Engineers for the October 1986 flood at the Keystone Dam and the Tulsa, OK 11th Street stream gage.

TABLE 2
HYDROLOGIC ANALYSES RESULTS

TRIBUTARY STREAM NAME	ARKANSAS RIVER STATION	TRIB 1% CHANCE (100-YR) PEAK FLOW	DATE OF TRIB PEAK	TIME OF TRIB PEAK	ARK RIVER PEAK DATE	ARK RIVER PEAK TIME	ADJUSTMENT TO TRIB HYDROGRAPH
Brush	427549	1865	5-Oct	0:45	5-Oct	1:00	NONE
RB1A	427545	1353	5-Oct	0:26	5-Oct	0:40	NONE
RB1	425169	2283	5-Oct	0:34	5-Oct	1:00	SHIFT +26 MIN
Little Sand	423127	2594	5-Oct	0:41	5-Oct	1:20	SHIFT +40 MIN
Sand	420000	2142	5-Oct	1:09	5-Oct	1:20	SHIFT +11 MIN
LB1	413827	1272	5-Oct	4:51	5-Oct	1:40	SHIFT -210 MIN
Mud	411859	6616	5-Oct	1:30	5-Oct	1:40	NONE
LB2	410667	1362	5-Oct	0:37	5-Oct	2:00	SHIFT +83 MIN
Shell	403112	7567	5-Oct	3:26	5-Oct	2:20	SHIFT - 60 MIN
Euchee	395476	7732	5-Oct	1:50	5-Oct	2:40	SHIFT + 50 MIN
Franklin	389855	4405	5-Oct	0:39	5-Oct	3:00	SHIFT + 141 MIN
Fisher Anderson	386127	15964	5-Oct	3:25	5-Oct	3:20	NONE
Main St.	383534	702	5-Oct	0:15	5-Oct	3:30	SHIFT +195 MIN
SAND SPRINGS DAM	**382170**						
Pratt	381842	5958	5-Oct	1:18	5-Oct	3:40	SHIFT +142 MIN
RB2	381420	778	5-Oct	0:18	5-Oct	4:00	SHIFT +222 MIN
Squirrel Hollow	380047	1850	5-Oct	0:16	5-Oct	4:00	SHIFT +224 MIN
Redfork	377829	2231	5-Oct	0:52	5-Oct	4:00	SHIFT +188 MIN
Pecan Woodland	375070	805	5-Oct	0:08	5-Oct	4:10	SHIFT +242 MIN
Big Heart	369362	14514	5-Oct	1:45	5-Oct	4:20	SHIFT + 155MIN
Berryhill	365659	9657	5-Oct	1:31	5-Oct	4:40	SHIFT + 190 MIN
Parkview	352698	1286	5-Oct	0:24	5-Oct	5:00	SHIFT + 276 MIN
Oak	350779	2249	5-Oct	0:37	5-Oct	5:00	SHIFT + 263 MIN
Downtown	346270	4475	5-Oct	0:21	5-Oct	5:10	SHIFT + 289 MIN
Elm	343801	6111	5-Oct	0:30	5-Oct	5:20	SHIFT +290 MIN
Swan	341813	2025	5-Oct	0:30	5-Oct	5:20	SHIFT + 290 MIN
ZINK DAM	**339414**				5-Oct		
Crow	337952	4753	4-Oct	16:10	5-Oct	5:40	SHIFT + 820 MIN
Cherry Red Fork	330033	7762	5-Oct	0:35	5-Oct	6:00	SHIFT + 325 MIN
Mooser	326058	9608	4-Oct	17:56	5-Oct	6:00	SHIFT + 716 MIN
Perryman	322620	3482	4-Oct	14:43	5-Oct	6:20	SHIFT + 937 MIN
Joe	311317	19086	5-Oct	1:30	5-Oct	6:40	SHIFT +310 MIN
Fred	307811	6807	5-Oct	0:53	5-Oct	6:40	SHIFT +347 MIN
RL Jones AP	306246	188	5-Oct	4:50	5-Oct	6:40	SHIFT +110 MIN
Vensel	300184	8766	4-Oct	17:31	5-Oct	7:00	SHIFT +811 MIN
JENKS DAM	**297420**						

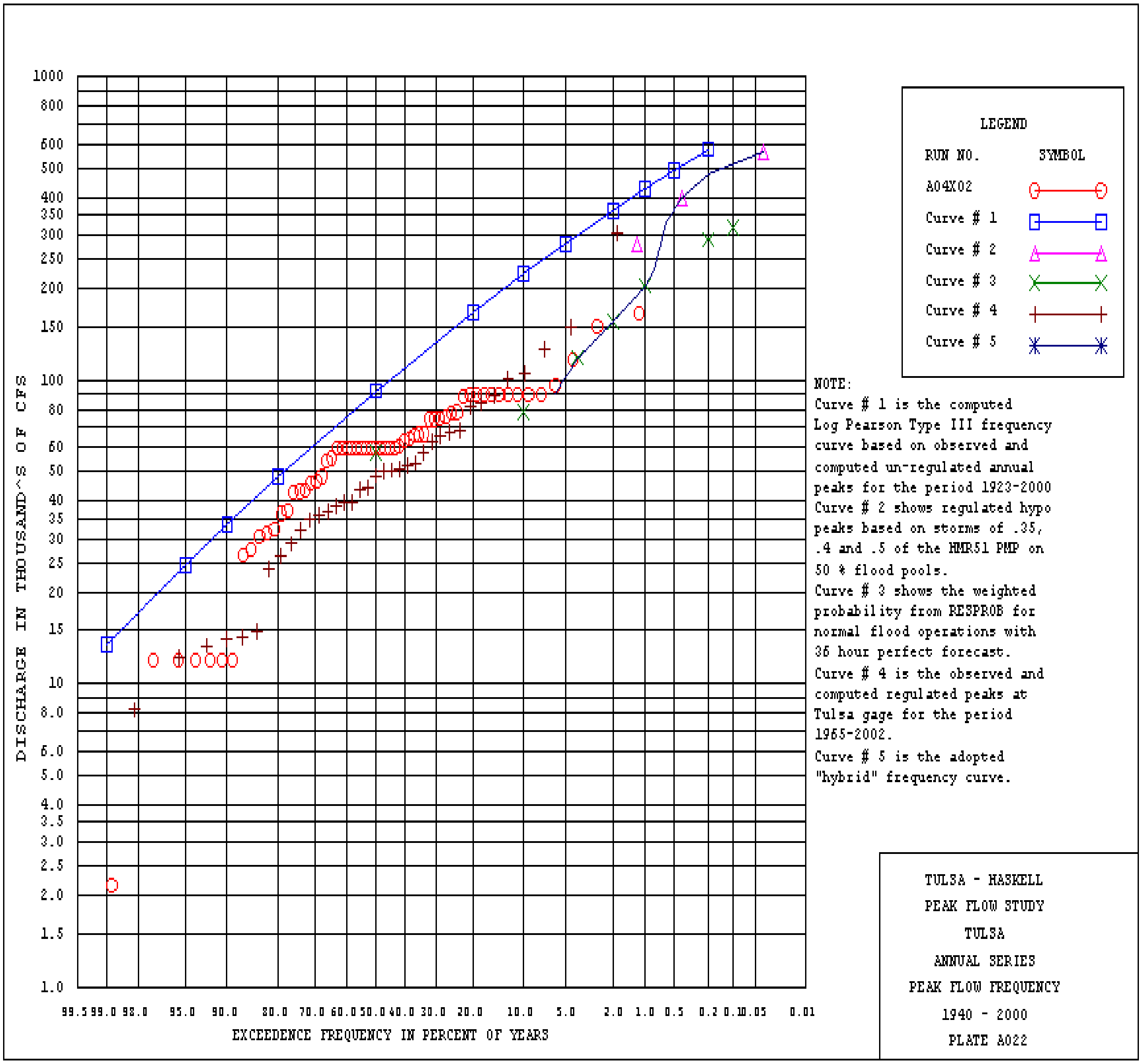


FIGURE 5. PEAK DISCHARGE FREQUENCY CURVE – ARKANSAS RIVER BELOW KEYSTONE DAM

HYDRAULIC (BACKWATER) MODELING

Existing Corps of Engineers Arkansas River HEC-RAS Steady State Backwater Model

Meshek obtained the Arkansas River HEC-RAS (version 3.1.3) steady state backwater computer model prepared by the Corps of Engineers for the Tulsa County Watershed Study of 2002. That models' floodplain and channel geometric properties were developed from elevation data (2-foot contour interval) prepared by Aerial Data Service of Tulsa in 2002 and was supplemented by field surveyed channel cross section data. The steady state model produces backwater profiles based on a peak flow condition and does not vary with time. Backwater profiles for the 10-, 50-, 100- and 500-year frequency floods were developed during that study. Meshek reviewed the model and its results and has duplicated the backwater profiles. Those profiles are shown at the end of this report in Appendix C as Panels 01P and 02P.

Existing Corps of Engineers Arkansas River HEC-RAS Unsteady State Backwater Model

Meshek also obtained the Arkansas River HEC-RAS unsteady state computer model (version 4.0) prepared by the Corps of Engineers in 2007. The unsteady model is a dynamic model and uses flow hydrographs as upstream boundary conditions. The model was prepared by Russ Wyckoff, P.E. in order to analyze the downstream impacts for various release scenarios, including the October 1986 flood. The model was calibrated by the Corps to the October 1986 flood release and reconstitutes the observed flows and stages at the Tulsa 11th Street gage effectively. That model was slightly modified (additional cross sections inserted and Keystone outflow hydrographs developed) and used for the various scenarios discussed in the following sections.

Verification of Unsteady State Model

The Corps of Engineers unsteady state HEC-RAS model was further verified by Meshek to 5 historical time periods for varying flow regimes. The models were verified using the flow and stage hydrographs at the USGS stream gage (Gage No. 07164500) located at the 11th St. Bridge in Tulsa (river station 349255.5). The periods used for the verification were:

October 1-30, 1986	Peak Flow = 306,000 cfs
August 19-21, 2008	Peak Flow = 12,200 cfs
September 14-29, 2008	Peak Flow = 66,630 cfs
October 15-25, 2008	Peak Flow = 23,000 cfs
May 23-29, 2009	Peak Flow = 49,740 cfs

Plots of the verification results are shown in Appendix A.

Tennessee Valley Authority (TVA) HEC-RAS Model

The TVA developed a backwater model using the existing Corps of Engineers HEC-RAS Steady State model. Several scenarios were developed to analyze the impacts of seven proposed low water dams. Those scenarios and the impacts on the 1% chance (100-year) flood elevation are shown in the following table.

TABLE 3
TVA BACKWATER ANALYSES COMPARISON

Cross Section	Plan ID	Plan Description	1% Chance (100-Year) Water Surface Elev. (ft)	Delta WSEL from Existing Conditions
Just Upstream of Jenks Dam Location				
299023.8	Plan 01	Existing Conditions	612.35	
299023.8	Plan 03	Structures Modeled as Geometry Ground Points	612.40	0.05
299023.8	Plan 06	Structures Modeled as Inline Weirs & Gates	612.56	0.21
299023.8	Plan 07	Dams Modeled as Changes in Cross Section Shape and Wetted Perimeter	612.30	-0.05
299023.8	Plan 08	Cross Sections Added at Dam Locations, but No Structures Added	612.33	-0.02
Just Upstream of Zink Dam Location				
340498	Plan 01	Existing Conditions	631.12	
340498	Plan 03	Structures Modeled as Geometry Ground Points	631.14	0.02
340498	Plan 06	Structures Modeled as Inline Weirs & Gates	630.93	-0.19
340498	Plan 07	Dams Modeled as Changes in Wetted Perimeter Only	631.14	0.02
340498	Plan 08	Cross Sections Added at Dam Locations, but No Structures Added	631.14	0.02
Just Upstream of Sand Springs Dam Location				
384445.6	Plan 01	Existing Conditions	649.27	
384445.6	Plan 03	Structures Modeled as Geometry Ground Points	649.32	0.05
384445.6	Plan 06	Structures Modeled as Inline Weirs & Gates	649.52	0.25
384445.6	Plan 07	Dams Modeled as Changes in Wetted Perimeter Only	649.29	0.02
384445.6	Plan 08	Cross Sections Added at Dam Locations, but No Structures Added	649.29	0.02

ALTERNATIVE SCENARIOS – UNSTEADY STATE HEC-RAS MODEL

Several flow scenarios were analyzed using the HEC-RAS unsteady state backwater model furnished by the Corps of Engineers and further modified by Meshek. Table 4 lists those scenarios and their pertinent information. A full description of the various scenarios and the results are given in the following sections.

TABLE 4
UNSTEADY RAS MODELING SCENERIOS

SCENARIO NAME	ARKANSAS RIVER FLOW (CFS)	COMMENTS
KEYS_900	900 cfs constant release made for Low Flow Requirements	No lateral inflow hydrographs added. Pilot Channel of 2'wide x 10'deep added for model stability. Channel "n" values modified to account for shallow flow depth
KEYS_3K	3,000 cfs constant release (one hydropower unit online at one half capacity)	No lateral inflow hydrographs added. Pilot Channel of 2'wide x 10'deep added for model stability. Channel "n" values modified to account for shallow flow depth
KEYS_6K	6,000 cfs constant release (one hydropower unit online)	No lateral inflow hydrographs added. Pilot Channel of 2'wide x 10'deep added for model stability. Channel "n" values modified to account for shallow flow depth
KEYS_12K_HYDPWR	Hydropower releases of 12,000 cfs made at 2 specified intervals	No lateral inflow hydrographs added. Simulation of hydropower releases: One in the morning for 4 hours and one in the evening for 3 hours.
KEYS_12K_HYDP1WR	Single 4 hour Hydropower releases of 12,000 cfs	No lateral inflow hydrographs added. Simulation of hydropower releases: One in the morning for 4 hours
KEYS_12K_INTERNALS	12, 000 constant release Hydropower Generation	Lateral tributary inflows added with no shift in hydrograph timing.
KEYS_12K_MOD_INTL	12, 000 constant release Hydropower Generation	Lateral tributary inflows added with a shift in hydrograph timing to coincide with river peaks.
KEYS_90K	90,000 peak release 10% Chance (10-year) flood	Peak flow of 90,000 cfs release from Keystone Dam using shape of Oct86 flood hydrograph – no lateral inflows added
KEYS_155K	155,000 peak release 2% Chance (50-year) flood	Peak flow of 155,000 cfs release from Keystone Dam using shape of Oct86 flood hydrograph – no lateral inflows added
KEYS_205K	205,000 peak 1% Chance (100-year) flood	Peak flow of 205,000 cfs release from Keystone Dam using shape of Oct86 flood hydrograph – no lateral inflows added
KEYS_300K	300,000 Keystone Dam release of Oct 86 flood	Peak flow of 300,000 cfs release from Keystone Dam using shape of Oct86 flood hydrograph – no lateral inflows added
KEYS_350K	350,000 peak Levee design flood	Peak flow of 350,000 cfs release from Keystone Dam using shape of Oct86 flood hydrograph – no lateral inflows added

SCENARIO – KEYS_900

A low flow simulation was made to simulate low flow conditions (minimum releases) from Keystone Dam to just below the proposed site of the Jenks low water dam. A constant release of 900 cfs from Keystone Dam was assumed and no lateral inflows from the intervening tributaries were added. Because the unsteady state backwater model tends to become unstable for very shallow flow conditions, a virtual pilot channel of 2' wide by 10' deep was added to the channel geometry. The additional 20 square feet of flow area at each cross section would not alter the conveyance capacity of the river to any significant degree. In addition to the pilot channel, additional cross sections were added at the proposed Sand Springs and Jenks low water dam locations that would better depict the change in hydraulic grade once the dams were actually designed. Finally, Manning's "n" values were adjusted in the channel to reflect the shallow flow conditions and interpolated cross sections were added at two locations several miles downstream of the Jenks low water dam location where channel streambed grade changes were significant. The 900 cfs maximum water surface profile is shown on Panel 03P of Appendix C.

SCENARIO – KEYS_3K

A scenario of one hydropower unit running at one half capacity was evaluated, producing a constant flow of 3,000 cfs from Keystone Dam. No lateral inflows from the intervening tributaries were added. The same changes to the channel geometry as used in the KEYS_900 scenario were used in this analysis. The 3,000 cfs maximum water surface profile is shown on Panel 03P of Appendix C.

SCENARIO – KEYS_6K

A scenario of one hydropower unit running at full capacity was evaluated, producing a constant flow of 6,000 cfs from Keystone Dam. No lateral inflows from the intervening tributaries were added. The same changes to the channel geometry as used in the KEYS-900a scenario were used in this analysis. The 6,000 cfs maximum water surface profile is shown on Panel 03P of Appendix C.

SCENARIO – KEYS_12K_HYDPWR

This alternative simulated a hydropower generation of 2 units generating for 4 hours in the morning and for 3 hours in the evening, a typical generation schedule. The same changes to the channel geometry as used in the KEYS_900 scenario were used in this analysis. The 12,000 cfs maximum water surface profile for this scenario is shown on Panel 03P of Appendix C.

SCENARIO – KEYS_12K_INTERNALS

Between Keystone Dam and the proposed Jenks Dam location, 30 streams flow into the Arkansas River and drain approximately 140 square miles. Because of the significant intervening drainage area above the proposed and existing low water dam locations, analyses were performed to evaluate the effects of the lateral inflows coupled with a constant Keystone Dam release of 12,000 cfs (full hydropower generation release). Hydrographs developed in the previously mentioned HMS and HEC-1 models for a 1% chance (100-year frequency) flood event on each of the 30 tributary streams were added as lateral inflow hydrographs. No adjustments to the timing of the lateral inflow hydrographs were made. The hydrographs for the Arkansas River and the intervening tributaries at the 3 low water dam locations is shown on Figures B-1, B-3 and B-5 in Appendix B. The maximum water surface profile for this scenario is shown on Panel 03P of Appendix C.

SCENARIO – KEYS_12K_MOD_INTL

This scenario is similar to the KEYS_12K_INTERNALS alternative, except that the lateral inflow hydrographs for the tributary basins have been shifted in time so that the tributary peak flows coincide with the Arkansas River hydrograph flood peak at the confluence with the tributaries. This would depict a "worst case scenario" for a normal hydropower release from Keystone Dam and a 100-year frequency flood occurring over the intervening basins down to the Jenks low water dam. The amount of time that each lateral tributary hydrograph was shifted is shown in Table 2. The resultant hydrographs for the Arkansas River and the intervening tributaries at the 3 low water dam locations are shown on Figures B-2, B-4 and B-6 in Appendix B. The maximum water surface profile for this scenario is shown on Panel 03P of Appendix C.

SCENARIO – KEYS_90K

This scenario uses a 10% chance (10-year frequency) flood peak of 90,000 cfs established by the Corps of Engineers in the Tulsa County Watershed Study of 2002. The discharges developed in the 2002 study are for instantaneous flood peaks and no hydrographs for those floods were developed. Since the unsteady state model uses flood flow hydrographs as upstream boundary conditions, the October 1986 flood hydrograph was used to form the shape of the hydrograph. Values in the October 1986 hydrograph were multiplied by a factor of 0.3 to develop the 90,000 peak hydrograph. The maximum water surface profile for this scenario is shown on Panel 03P of Appendix C.

SCENARIO – KEYS_155K

This scenario uses a 2% chance (50-year frequency) flood peak of 155,000 cfs established by the Corps of Engineers in the Tulsa County Watershed Study of 2002. The discharges developed in the 2002 study are for instantaneous flood peaks and no hydrographs for those floods were developed. Since the unsteady state model uses flood flow hydrographs as upstream boundary conditions, the October 1986 flood hydrograph was used to form the shape of the hydrograph. Values in the October 1986 hydrograph were multiplied by a factor of 0.517 to develop the 155,000 peak hydrograph. The maximum water surface profile for this scenario is shown on Panel 03P of Appendix C.

SCENARIO – KEYS_205K

This scenario uses a 1% chance (100-year frequency) flood peak of 205,000 cfs established by the Corps of Engineers in the Tulsa County Watershed Study of 2002. The discharges developed in the 2002 study are for instantaneous flood peaks and no hydrographs for those floods were developed. Since the unsteady state model uses flood flow hydrographs as upstream boundary conditions, the October 1986 flood hydrograph was used to form the shape of the hydrograph. Values in the October 1986 hydrograph were multiplied by a factor of 0.6833 to develop the 205,000 peak hydrograph. The maximum water surface profile for this scenario is shown on Panel 03P of Appendix C.

SCENARIO – KEYS_300K

This scenario uses the releases from Keystone Dam that occurred during the October 1986 flood as the upstream boundary condition. The maximum water surface profile for this scenario is shown on Panel 03P of Appendix C.

SCENARIO – KEYS_350K

This scenario uses a flood peak of 350,000 cfs (levee design flood for the Tulsa-West Tulsa and Jenks levees). The October 1986 flood hydrograph was used to form the shape of the 350,000 cfs hydrograph. Values in the October 1986 hydrograph were multiplied by a factor of 1.1667 to develop the 350,000 peak hydrograph. The maximum water surface profile for this scenario is shown on Panel 03P of Appendix C.

SCENARIO – KEYS_12K_HYDP1WR

This scenario was developed to illustrate the travel time and attenuation of a typical hydrograph for a single hydropower generation of 12,000 cfs for 4 hours. It should be noted that the hydrograph peak takes around 5 hours to travel from Keystone Dam to the Sand Springs proposed dam location, about 4 hours to travel from the Sand Springs dam location to the Zink Dam location, and another 4.5 hours to travel from the Zink Dam to the proposed Jenks Dam location. The peak flow attenuates from 12, 000 just below Keystone Dam to 10,450 cfs at the Sand Springs Dam, to 8,690 cfs at the Zink Dam, and then to 7,730 cfs at the Jenks Dam location. The travel times and attenuations do not consider the proposed Sand Springs and Jenks low water dams being in place with their proposed operational procedures. Figure B-7 in Appendix B illustrates the hydrograph travel and attenuation for the 12,000 cfs hydropower release.

ALTERNATIVE SCENARIOS –STEADY STATE HEC-RAS MODEL

Several different scenarios were evaluated using the modified Corps of Engineers' steady state backwater model for the Arkansas River. The primary focus of the evaluations was to determine the amount of rise in water surface elevations for various flows, including the 1% chance (100-year frequency) flood of 205,000 cfs and the levee design flood of 350,000 cfs. Each scenario was compared to the existing conditions backwater model at each cross section. Since no final design exists for any of the proposed or modified low water dams, the structures were evaluated using simplified designs. There were 4 basic ways of evaluating the low water dams: 1) Existing Conditions with the existing Zink Dam in place and no low water dams at the Sand Springs or Jenks locations. 2) Model each low water dam as a bridge using piers to simulate the structure as if all gages are open. This technique would give the least amount of rise in water surface elevations. 3) Model the low water dams with a combination of bridge piers and blocked obstructions. The blocked obstructions would represent the area block by the weir at the low water dams. 4) Model each structure using the Inline Structure Option in the HEC-RAS program. This option allows the input of actual weirs and gates and gives the option to have different gate opening scenarios. Table 5 gives the amounts of rise at key locations for the 4 different scenarios evaluated. Profiles of the 4 scenarios are shown in Appendix C on Panel 04P. It should be noted that the calculated water surface elevations for the final designs could be higher by 0.1 to 0.5 feet based on using a stepped overflow structure instead of an Ogee Weir and adding roughened channel sluices for fish passage and recreational venues. An examination of the changes in floodplain areas for the various plans indicates the 1% chance (100-year) floodplain will increase in area by about 200 acres (1%) over the entire length of the project area.

TABLE 5
COMPUTED WATER SURFACE ELEVATIONS
PLAN COMPARISONS[1]

LOCATION AND CROSS SECTION NUMBER	EXISTING CONDITIONS – EXISTING ZINK DAM IN, NO DAMS AT JENKS OR SAND SPRINGS ELEV	EXISTING CONDITIONS WITH JENKS RIVER DISTRICT IN AS PLANNED ELEV/RISE FT.	PLAN A – LOW WATER DAMS MODELED ONLY AS BRIDGE PIERS ELEV/ RISE FT.[2]	PLAN A1 - LOW WATER DAMS MODELED AS BRIDGE PIERS WITH BLOCKED AREAS AS WEIRS ELEV/ RISE FT.[2]	PLAN A1-2 - LOW WATER DAMS MODELED AS INLINE STRUCTURES ELEV/ RISE FT.[2]
1% Chance (100-year) flood (205,000 cfs)					
Above Jenks LWD – 298676.5	612.23	612.67 / 0.44	612.69 / 0.02	612.78 / 0.11	612.78 / 0.11
Above Zink LWD – 340498.0	631.11	631.12 / 0.01	630.78 / -0.34	630.79 / -0.33	630.74 / -0.38
Above Sand Springs LWD – 384445.6	649.29	649.29 / 0.00	649.31 / 0.02	649.65 / 0.36	649.44 / 0.15
Levee Design Flood (350,000 cfs)					
Above Jenks LWD – 298676.5	618.11	619.5 / 1.39	618.77 / -0.73	619.62 / 0.12	618.88 / -0.62
Above Zink LWD – 340498.0	638.00	638.11 / 0.11	637.95 / -0.16	638.03 / -0.08	637.89 / -0.22
Above Sand Springs LWD – 384445.6	657.44	657.44 / 0.00	657.52 / 0.08	657.76 / 0.32	657.49 / 0.05

[1] Plans A, A1, and A1-2 all assume the Jenks River District development is in as planned by Jenks.
[2] For Jenks Dam Area, Rise is the increase above the “Existing Conditions plus Jenks River District Development In Place as Proposed” condition.

River District Development

The proposed River District development is a 424-acre, multi-use project planned for the west bank of the Arkansas River south of the Creek Turnpike near Jenks, Oklahoma . This development consists of "moving" the existing right bank of the Arkansas River eastward and filling in the existing floodplain to elevation 617.0 feet, N.G.V.D. The proposed Jenks low water dam would tie into the River District development near cross section 295373.8. Figure 5 shows the approximate location of the proposed River District development, and Figures 6 and 7 show the extent of the proposed River District fill for 2 cross section locations.

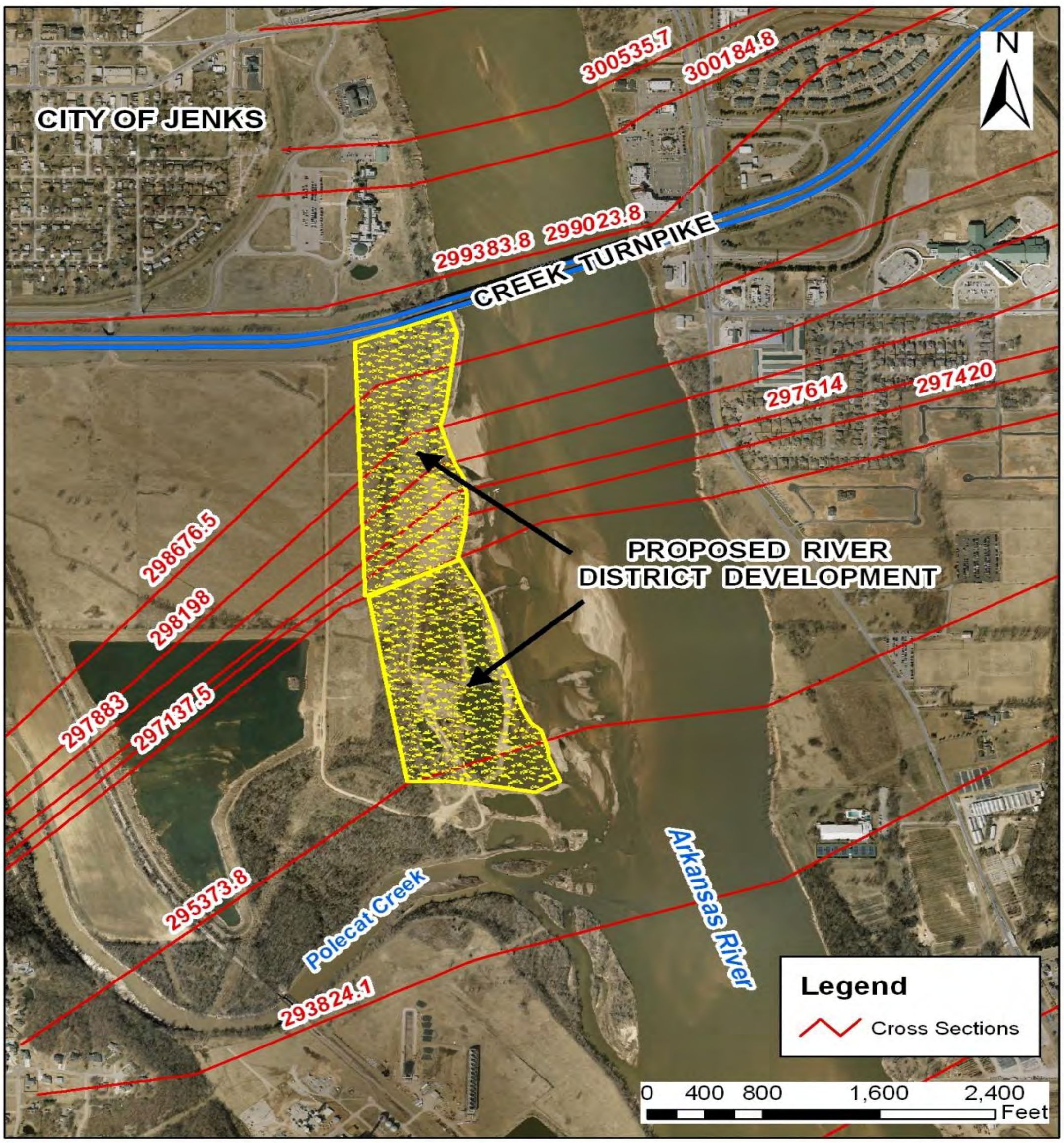


FIGURE 5. PROPOSED JENKS RIVER DISTRICT DEVELOPMENT

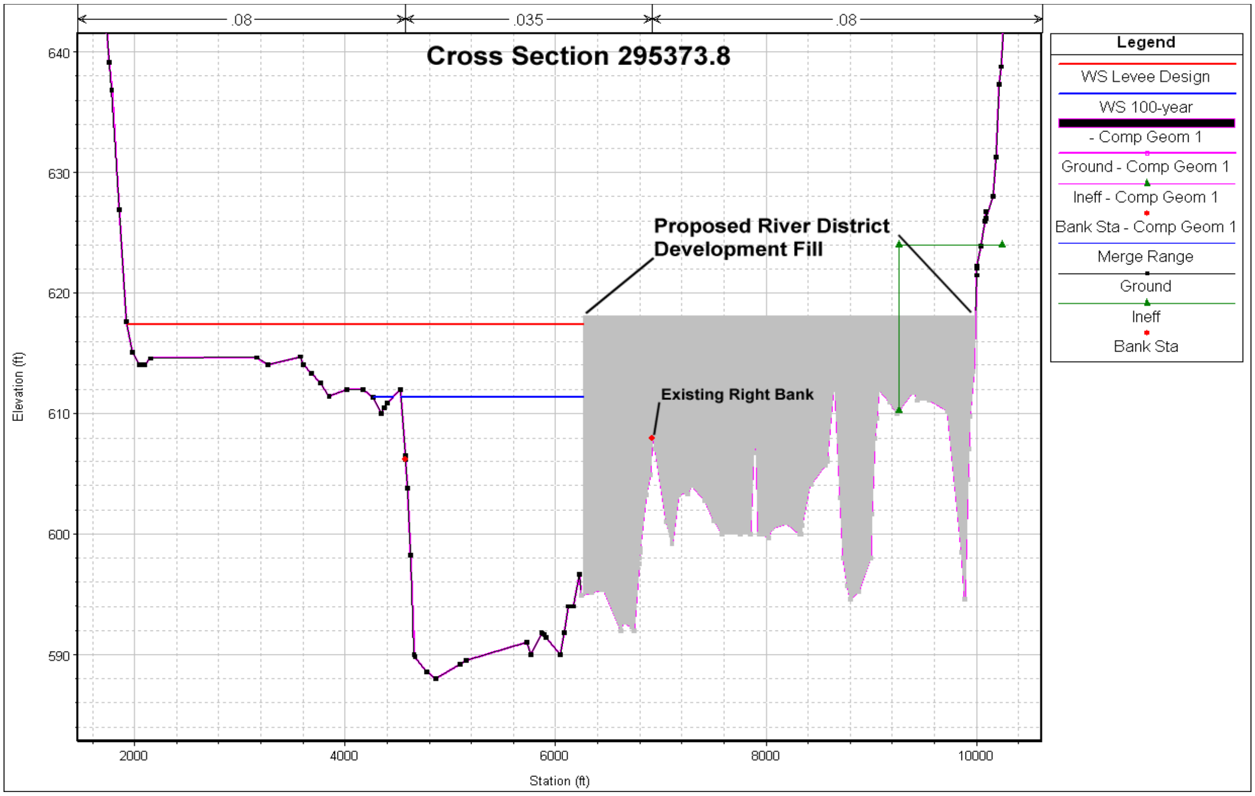


FIGURE 6. RIVER DISTRICT DEVELOPMENT AT CROSS SECTION 295373.8

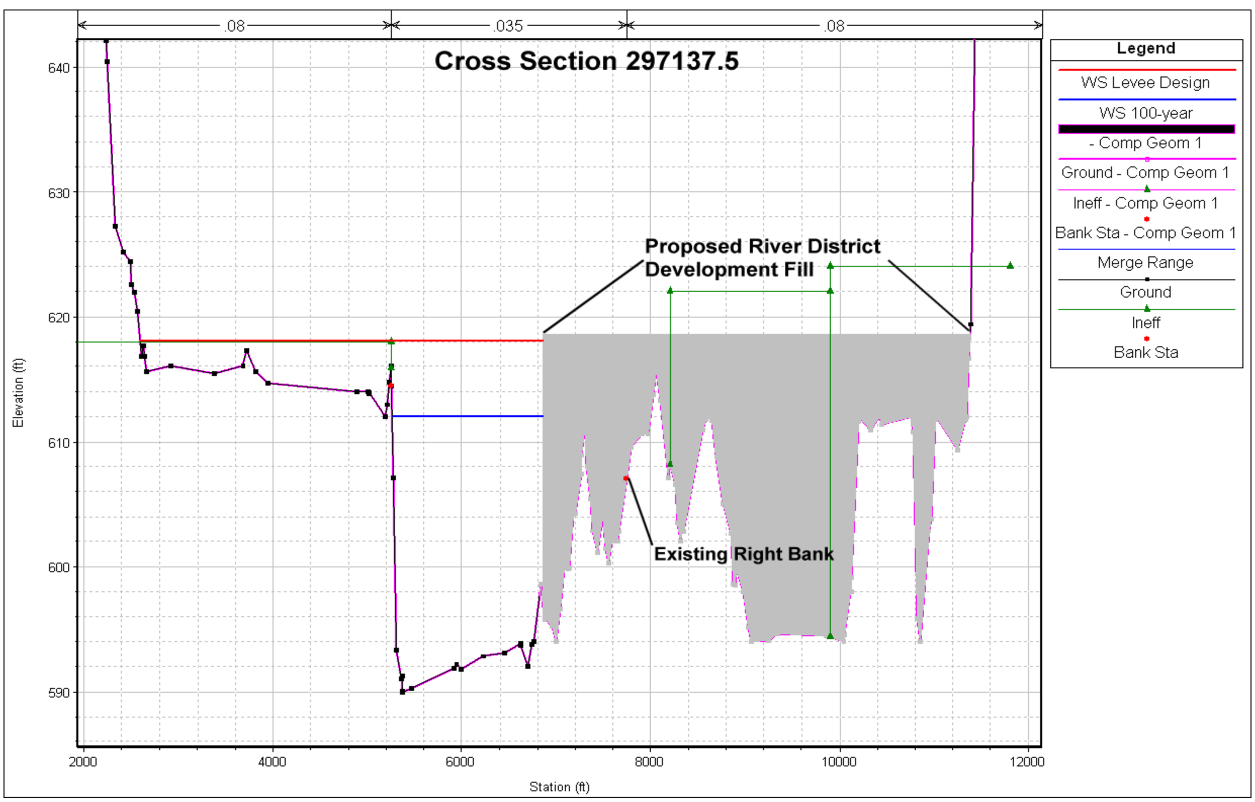


FIGURE 7. RIVER DISTRICT DEVELOPMENT AT CROSS SECTION 297137.5

"NO RISE" ALTERNATIVE AND RECOMMENDED PLAN

No Rise Alternative

Since all of the alternatives presented in Table 5 produce a rise above the 1% chance (100-year) flood at the Jenks and Sand Springs locations, an additional alternative was included for analysis that would minimize or eliminate the water surface rise. In this scenario, the proposed Jenks low water dam would moved about 2,000 feet north and placed along an alignment near cross section 297137.5 as depicted in Figure 8. This alignment could tie into either the existing high ground on the west end or to the proposed Jenks River District development fill, if constructed. The Jenks low water dam would consist of weirs located on either side of a gated structure. The weirs would be 500-600 feet in length with crests at elevation 598.0. With a weir crest elevation of 598 instead of the previous 596, the static pool would be 2 foot deeper and would extend closer to development located on the east side of the river north of the 96th Street bridge. The gated structure would consist of 7 - 100' wide gates able to drop to streambed level. This alignment and configuration would produce a net decrease in the 1% chance and Levee Design Floods upstream of the Jenks low water dam. Table 6 gives elevation information regarding this alternative.

For the Zink low water dam, no additional revisions are necessary since adding gates to the existing low water dam where none now exist will produce a net decrease in the 1% chance (100-year) and Levee Design Floods. The dam would consist of weirs located on either side of a gated structure at crest elevation 620.0. The gated structure would be comprised of 6 – 100' wide gates able to be dropped to streambed level. Refer to Table 6 for elevation information.

At the Sand Springs low water dam location, it was discovered that reshaping and maintaining the left (north) portion of the existing river channel downstream of the proposed dam alignment and for a distance of about 2,400 feet will result in lower tailwater conditions and thus an overall decrease in the 1% chance (100-year) and Levee Design Floods for all scenarios. The dam would consist of weirs located on either side of a gated structure at crest elevation 638.0. The gated structure would be comprised of 9 – 100' wide gates able to be dropped to streambed level. Figures 9 through 11 illustrate this configuration. Table 6 gives the elevation data for this scenario.

Since there is not yet a specific design proposed for the actual weir construction, a weir coefficient for the overflow sections of the 3 low water dams was not available. Therefore, a weir coefficient of 3.1 was used in the weir flow equations. Any decrease in the selected weir flow coefficients will ultimately cause an increase in computed water surface elevations without offsetting geometry or structural changes.

Recommended Plan

The plan recommended in this report consists of the No Rise scenarios for the Jenks and Zink Dams, but no channel shaping below the Sand Springs Dam due to the sensitive environmental nature of that option. In addition, the weir sections at each dam would consist of a stepped weir deck below each structure to prevent any roller effect typical of an Ogee type weir. All gates would be Obermeyer gates.

The Jenks low water dam would be located near section 297137.5, but could be shifted slightly further north or south to accommodate construction requirements and needs. The weir crest would be at elevation 596 and the gated structure would consist of 7 – 100' wide x 6' high gates with sills placed at elevation 590.0.

The existing Zink low water dam would modified at its current location. The weir crest would be raised from it current elevation of 617.0 to an elevation of 620.0 to increase depth. The gated structure would consist of 6 – 100' wide x 8.5' high gates with sills placed at elevation 611.5.

The Sand Springs low water dam would have a weir crest at elevation 638.0 and would consist of a gated structure with 8 – 100' wide x 10' high gates. Gate sills would be at elevation 628.0. No channel shaping would be required.

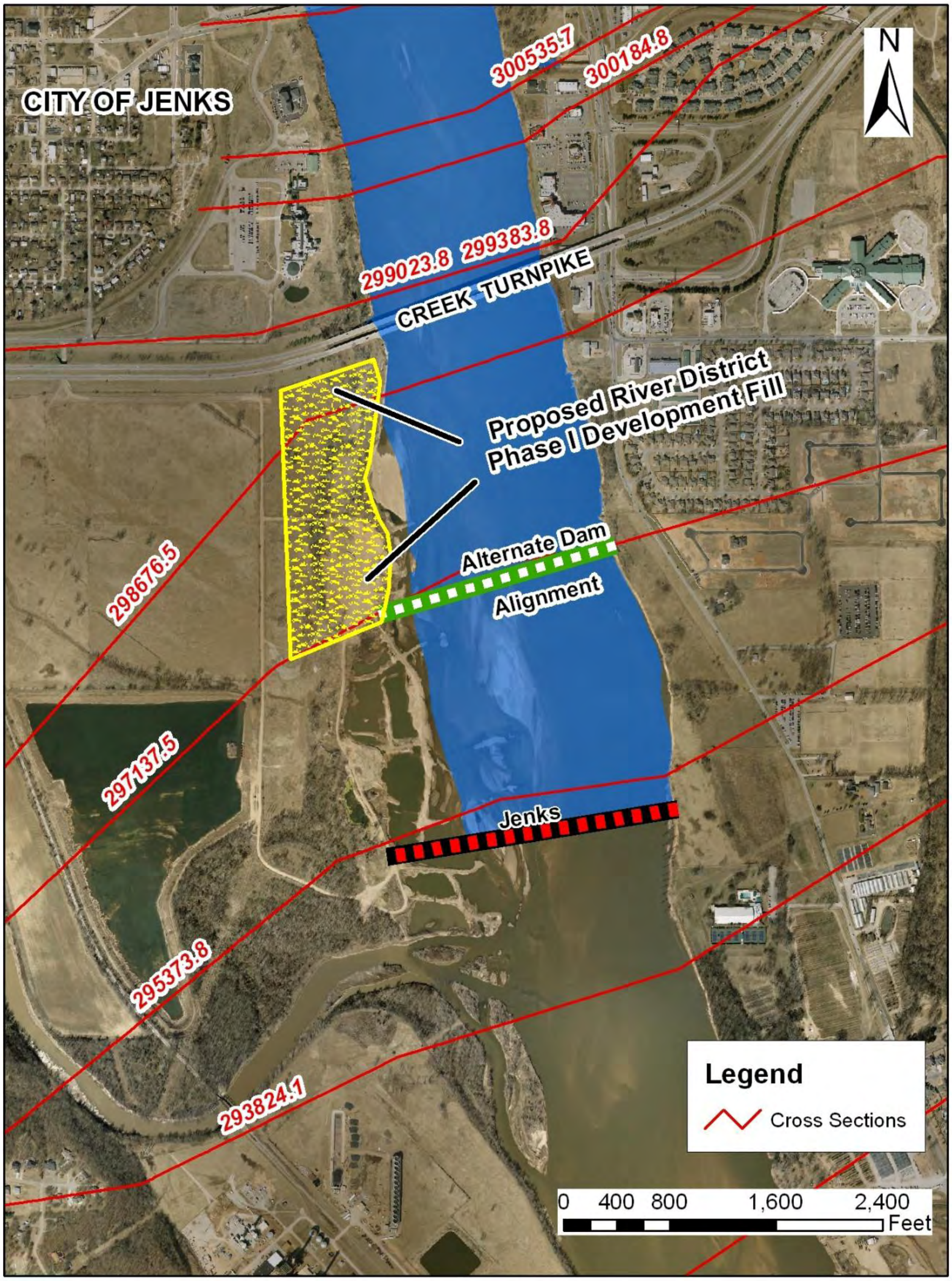


FIGURE 8. JENKS LOW WATER DAM "NO RISE" ALIGNMENT

TABLE 6
COMPUTED WATER SURFACE ELEVATIONS
"NO RISE" ALTERNATIVE & RECOMMENDED PLAN

LOCATION AND CROSS SECTION NUMBER	EXISTING CONDITIONS – EXISTING ZINK DAM IN, NO DAMS AT JENKS OR SAND SPRINGS ELEV	"No Rise" ALTERNATIVE - JENKS DAM RELOCATED, ZINK DAM MODIFIED, CHANNEL SHAPING DOWNSTREAM OFSAND SPRINGS DAM ELEV/ RISE FT.	RECOMMENDED PLAN – JENKS DAM RELOCATED, ZINK DAM MODIFIED, NO CHANNEL SHAPING DOWNSTREAM OF SAND SPRINGS DAM ELEV / RISE FT.
1% Chance (100-year) flood (205,000 cfs)			
Above Jenks LWD – 298676.5	612.23	611.77 /- 0.46	611.86 / -0.37
Above Zink LWD – 340498.0	631.11	630.84 / -0.27	631.00 / -0.11
Above Sand Springs LWD – 384445.6	649.29	649.27 / -0.02	649.48 / 0.19
Levee Design Flood (350,000 cfs)			
Above Jenks LWD – 298676.5	618.11	617.61 / -0.50	617.74 / -0.37
Above Zink LWD – 340498.0	638.00	637.92 / -0.08	638.24 / 0.24
Above Sand Springs LWD – 384445.6	657.44	657.47 / 0.03	657.64 / 0.20

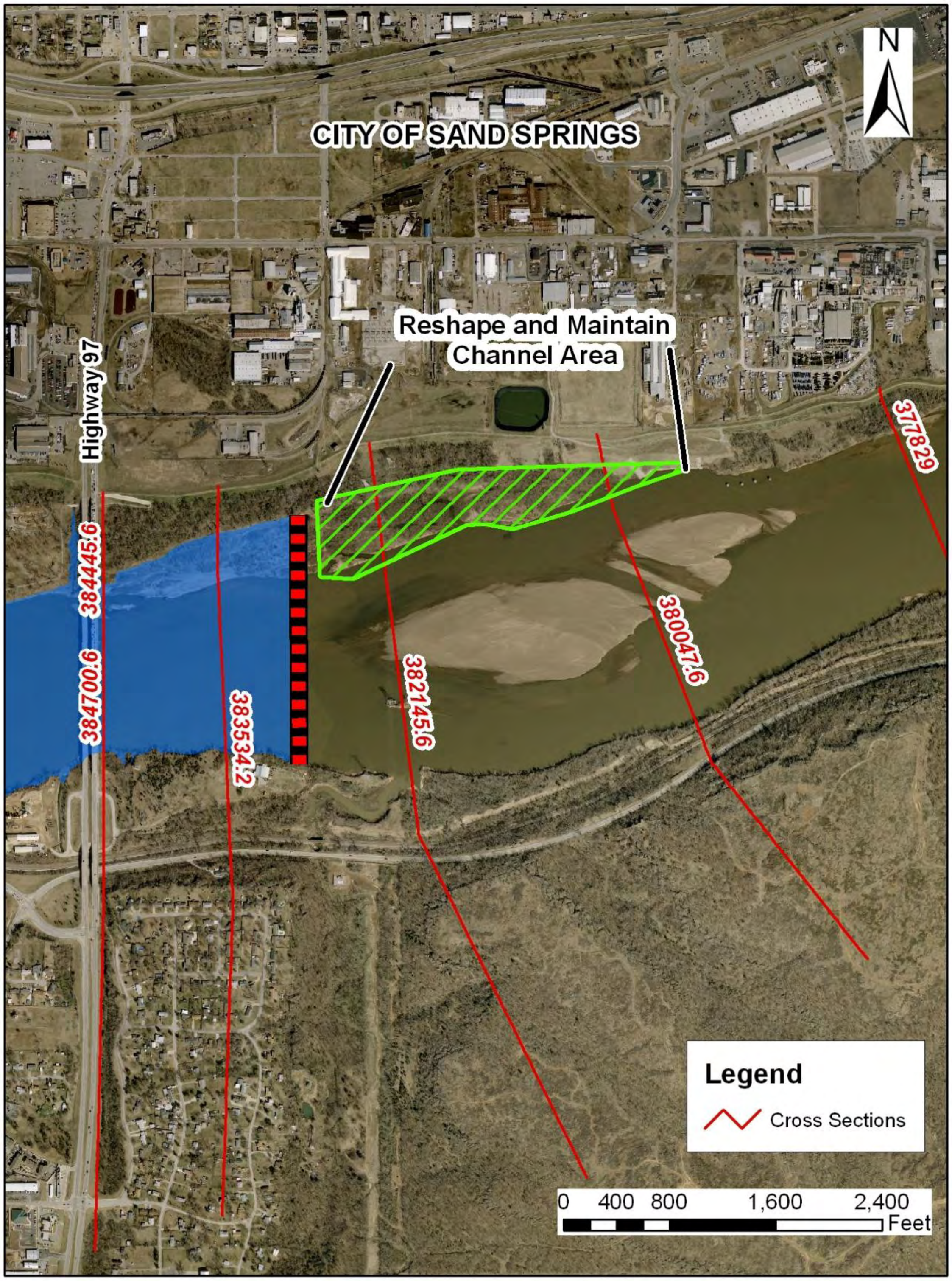


FIGURE 9. SAND SPRINGS LOW WATER DAM "NO RISE" ALTERNATIVE

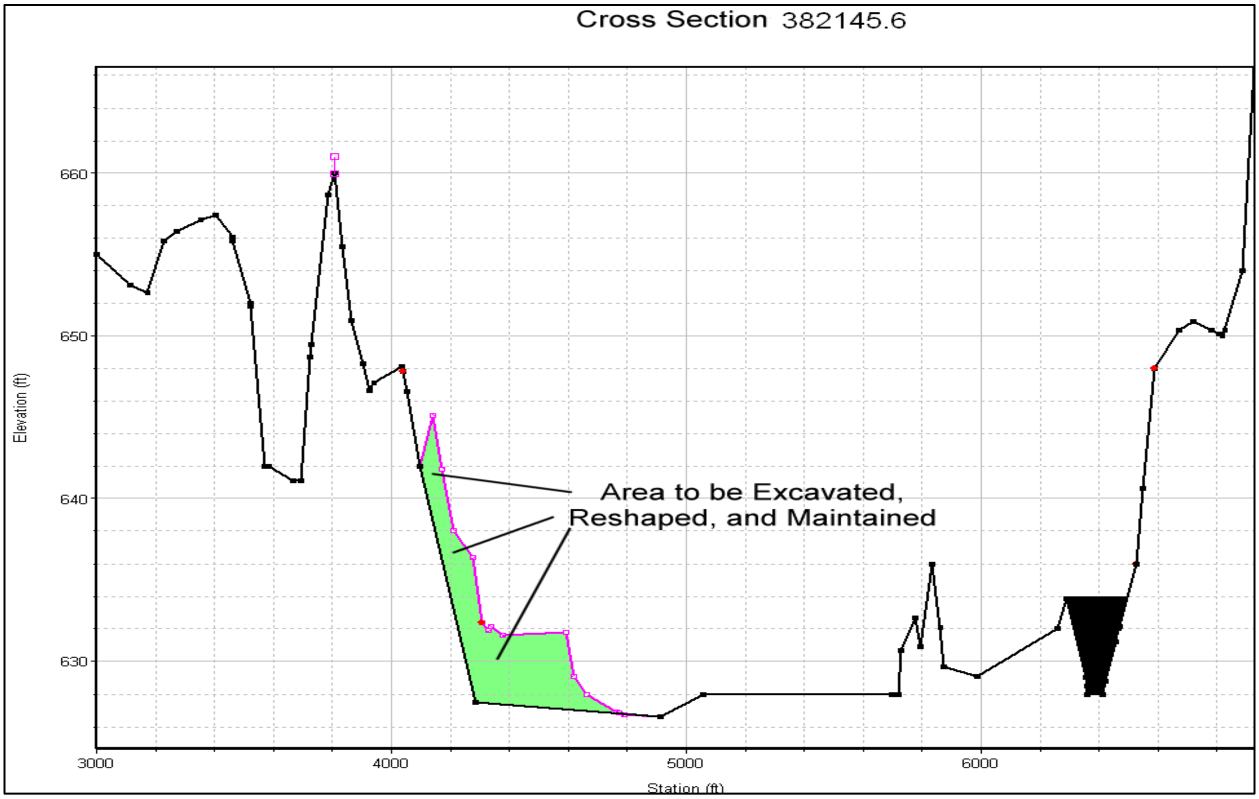


FIGURE 10. CHANNEL RESHAPING BELOW SAND SPRINGS DAM – CROSS SECTION 382145.6

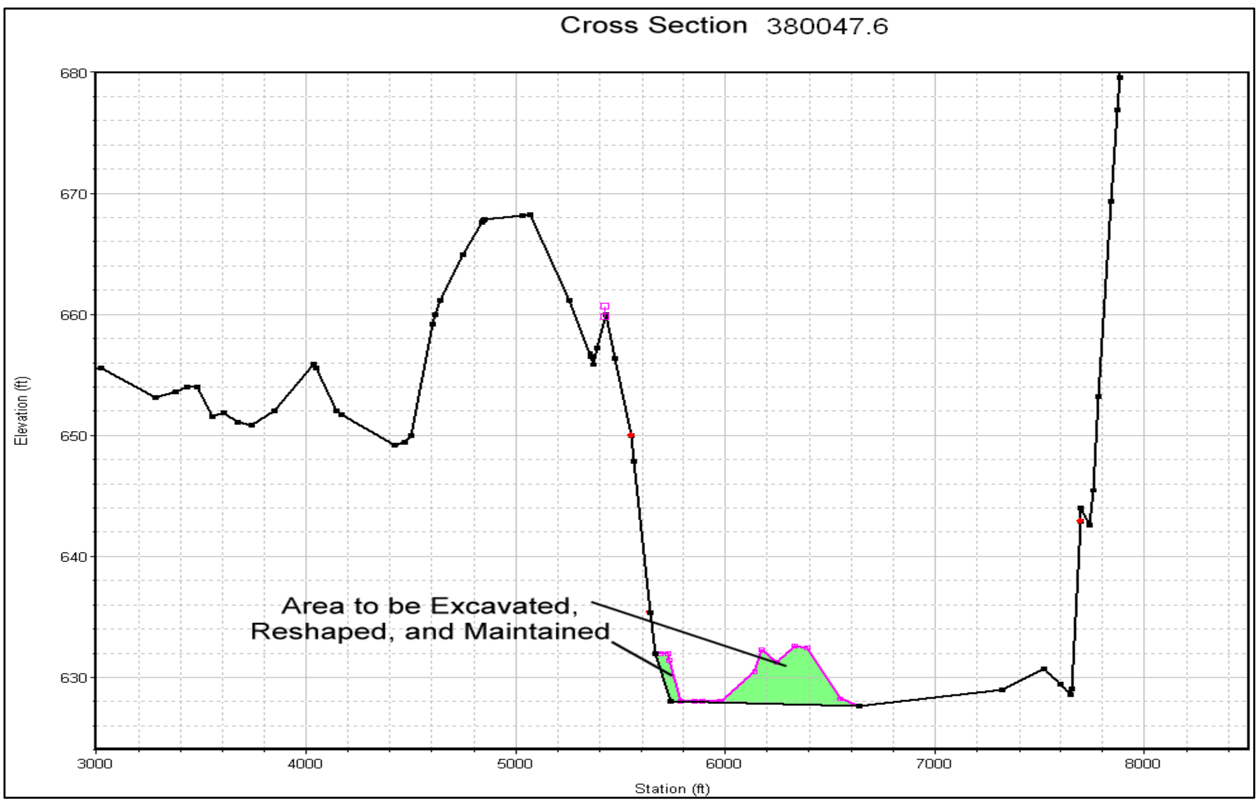


FIGURE 11. CHANNEL RESHAPING BELOW SAND SPRINGS DAM – CROSS SECTION 380047.6

FLOW FREQUENCY AND DURATION ANALYSES

One of the most important factors in the design and operational evaluation of the existing and proposed low water dams is the duration and frequency of Arkansas River flows for various discharge values and for varying operational scenarios. Critical factors such as seasonal fish migrations, recreational venues and scheduled times, seasonal fish spawning, and water quality requirements all play an important role in the design and operation of the dams. It should be noted that Keystone Dam is equipped with two hydropower generating units, each of which discharges approximately 6,000 cfs at full operation for a total discharge of 12,000 cfs from the dam. The electricity generated from these units is part of the Southwestern Power Administration's (SWPA) electrical grid system and are for peak loads only. During the summer a typical schedule for operation of the hydropower units involves peak generation for a period of 4-5 hours in the evening (1 p.m. through 6 p.m.) on a Monday through Friday basis. During the winter months, the operation involves peak generation for 2 periods during the day: 5 a.m. through 10 a.m. and 5 p.m. through 10 p.m., also on a Monday through Friday basis. These times and schedules are variable depending on SWPA's customers' needs and demands.

A peak discharge frequency curve for the Keystone Dam releases was previously developed by the Tulsa District Corps of Engineers and has been presented in this report as Figure 5. However, of equal importance to the design and operation of the project are the daily and hourly flow durations for flows in the range of 100 to 12,000 cfs. Mean daily flow records are available from the US Geological Survey for the Arkansas River at the 11th Street gage for the period June 24, 1964 through the current date. Hourly flow data is also available from the USGS for the period October 11th, 1987 through September 30th, 2008.

Table 7 presents mean monthly flow values determined from daily averages as recorded at the 11th Street stream gage on the Arkansas River for the period October 1964 through September 2008. The values highlighted in blue represent months where the monthly mean is above a flow of 15,000 cfs, which represents those values in the upper flow range. The values highlighted in pink represent months where the flows average below 1,000 cfs and represent flows occurring in the lower range. The months with no color highlighting are for those flows in the intermediate range. As illustrated in the table, the fall and winter months are more likely to experience periods of low flows while the spring and summer months are more likely to experience periods of flows exceeding 15,000 cfs.

TABLE 7
ARKANSAS RIVER AT 11TH ST. GAGE
MEAN MONTHLY FLOWS

YEAR	Monthly mean in cfs (Calculation Period: 1964-10-01 -> 2008-09-30)											
	Period-of-record for statistical calculation restricted by user											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1964										490.6	19,340	8,100
1965	4,043	674.2	1,790	7,130	5,824	22,450	10,710	2,443	18,680	3,498	2,484	2,985
1966	2,056	4,571	2,860	1,775	2,733	2,595	1,740	1,821	2,313	2,331	1,318	738.6
1967	483.3	494.1	1,400	692.9	880.9	5,350	16,470	5,712	4,847	5,795	1,614	1,148
1968	1,760	2,214	3,896	7,101	8,653	8,686	3,049	6,070	4,574	5,541	5,314	6,085
1969	3,336	3,696	8,355	10,960	25,950	20,770	7,345	3,404	8,011	4,599	3,002	2,740
1970	2,598	2,331	1,399	19,470	9,448	7,498	6,968	1,332	1,219	2,432	2,589	1,611
1971	1,466	1,597	4,045	2,034	2,441	6,714	3,463	2,979	4,326	3,154	4,816	5,955
1972	3,711	1,924	1,517	1,522	2,858	2,664	2,884	1,949	2,163	1,644	3,423	3,013
1973	12,260	13,640	37,350	44,460	26,890	7,660	3,361	4,569	4,793	48,920	11,640	12,260
1974	9,822	10,380	24,530	10,960	17,490	19,630	6,789	4,912	11,390	5,113	39,390	8,569
1975	11,010	18,550	17,520	11,450	21,090	38,930	8,808	4,364	3,546	1,745	2,052	1,661
1976	2,009	1,940	2,083	3,700	8,413	6,908	9,242	3,275	1,301	1,203	1,391	1,124
1977	1,155	518.2	490.2	700.6	8,881	15,550	8,789	6,778	12,920	4,393	4,990	1,894
1978	2,833	4,359	8,392	6,273	8,191	13,230	4,691	2,088	1,866	765.8	1,548	1,891
1979	1,409	2,301	13,870	15,180	11,900	8,542	6,822	6,856	7,401	1,388	18,940	7,211
1980	5,333	4,897	6,335	18,770	19,190	14,420	7,728	1,129	1,697	517.2	567.5	659.2
1981	780.2	712.5	668.4	556.6	2,583	5,712	3,867	2,534	3,082	1,755	11,490	3,935
1982	1,597	4,992	7,302	2,601	24,560	31,810	16,000	5,232	1,577	836.4	457.4	582
1983	2,254	3,853	6,566	29,190	18,610	13,410	12,270	1,409	1,293	5,571	3,977	2,108
1984	1,417	3,160	16,350	35,800	12,880	8,744	5,059	2,631	1,988	836	867.7	5,182
1985	8,464	6,252	21,000	10,630	14,500	13,180	7,063	4,282	6,695	26,600	10,460	11,690
1986	7,029	3,773	3,947	5,115	11,520	10,560	7,912	3,612	3,464	72,720	23,230	11,270
1987	9,578	19,450	42,890	18,970	14,000	28,880	19,450	6,210	6,684	7,070	2,067	5,335
1988	14,940	7,628	19,520	29,190	7,841	3,135	2,920	1,665	3,228	2,089	1,770	2,141
1989	2,747	2,308	3,736	5,791	4,485	16,750	11,080	10,750	23,280	7,313	3,510	1,818
1990	5,262	5,234	21,040	22,550	12,380	7,085	4,690	2,102	1,577	739.1	922	659.5
1991	1,279	1,460	662.3	2,179	4,302	4,572	1,314	1,150	2,585	2,165	3,112	6,805
1992	5,767	2,931	2,925	3,151	3,110	13,470	13,150	16,190	7,384	989.6	9,114	16,830
1993	19,630	22,500	18,110	16,640	81,400	32,010	24,800	16,690	7,753	2,771	2,731	1,985
1994	2,779	3,598	4,704	12,510	23,970	6,138	2,820	3,208	1,156	858	6,489	6,596
1995	3,646	2,132	13,540	5,994	25,960	69,820	27,790	32,970	5,573	3,024	2,206	2,730
1996	2,953	4,236	2,011	1,141	1,871	4,726	2,712	9,620	10,410	11,720	17,080	11,950
1997	5,191	9,221	11,390	25,360	12,580	11,250	24,650	17,630	12,590	11,730	4,093	6,554
1998	19,850	9,726	25,000	28,580	17,390	6,306	4,549	1,726	892.8	18,270	54,540	10,430
1999	5,245	15,920	20,520	22,650	34,310	42,850	37,630	8,196	4,955	3,649	1,353	13,380
2000	6,298	4,274	26,650	18,840	10,680	8,933	14,810	4,073	2,429	2,048	6,672	3,036
2001	4,225	10,400	24,620	7,247	9,956	17,730	5,194	783	1,078	1,765	1,340	545.1
2002	869.5	3,283	1,388	2,365	4,273	12,520	8,209	5,166	4,026	15,010	7,116	3,122
2003	3,482	3,807	15,970	10,090	11,700	10,990	4,527	2,677	5,513	11,360	2,429	1,914
2004	2,821	7,036	29,200	9,540	15,310	10,370	18,210	8,966	2,010	1,661	6,984	4,924
2005	12,220	10,730	8,418	5,793	3,554	27,240	8,908	11,900	7,013	3,065	2,196	955
2006	1,124	1,884	840.9	872.7	8,300	1,633	2,802	2,085	1,300	305	58.8	84.8
2007	163.4	1,314	5,749	25,440	30,470	44,040	52,540	20,280	7,636	6,372	2,329	3,023
2008	4,189	9,498	9,664	17,020	29,400	40,010	21,960	4,623	21,110			
Mean	**5,020**	**5,800**	**11,400**	**12,200**	**14,400**	**16,000**	**10,800**	**6,090**	**5,670**	**7,180**	**7,110**	**4,710**

** No Incomplete data have been used for statistical calculation

To gain a more detailed picture of the periods of high and low discharges, flow duration curves were developed. Figures D-1 and D-2 in Appendix D show plots of the mean monthly and mean daily flows on the Arkansas River at the 11th Street gage, respectively. Additionally, mean monthly and mean hourly flow duration curves have been developed and are shown in Figures D-3 and D-4 in Appendix D. The information show in the hourly flow duration curve is summarized in Table 8.

TABLE 8
ARKANSAS RIVER AT 11TH ST. GAGE
HOURLY FLOW EXCEEDENCE

MONTH OR TIMEFRAME	FLOW, C.F.S.	PERCENT OF TIME FLOW IS EQUALED OR EXCEDDED
Yearly	**1,000**	**78%**
March through May	**1,000**	**85%**
May through September	**1,000**	**81%**
May	**1,000**	**87%**
June	**1,000**	**88%**
July	**1,000**	**85%**
August	**1,000**	**76%**
September	**1,000**	**68%**

RECOMMENDATIONS FOR ADDITIONAL HYDROLOGIC AND HYDRAULIC ANALYSES

The information provided in this Technical Memorandum (excluding the existing conditions Arkansas River backwater analyses) should be considered exploratory and conceptual in nature. Additional detailed hydrologic and hydraulic analyses will be required to develop the final design, placement, and operation of the proposed low water dams. The following is a discussion of the required analyses.

FIELD SURVEYS

To ensure adequate levee freeboard is maintained, surveys along the crown of the levees should be performed at the beginning of the next phase of work. Additionally, detailed surveys of the channel and overbanks along the alignment of the proposed dam locations will be required for design of the structures.

HYDROLOGIC ANALYSES

The frequency peak flow information previously developed for the Arkansas River by the Corps of Engineers in 2002 will continue to be used for the backwater impact analyses of the low water dams. However, since that data is for instantaneous discharges only and the future hydraulic analyses will be unsteady state in nature, a hydrograph depicting the flow of the river will be required. One possible scenario is to use the October 1986 flood hydrograph occurring just below Keystone Dam as a general shape and configuration for the frequency releases, just altering the magnitude of the hydrograph.

In addition to the Arkansas River hydrology, the hydrology information developed in this Technical Memorandum for the intervening drainage basins should be reviewed and confirmed for use in the operational requirements of the low water dams.

STEADY STATE BACKWATER ANALYSES

A final "With Project Conditions" backwater profile and floodway model that includes the final plan for the proposed and modified low water dams will be required for submittal to FEMA. The final selected low water dam gate and weir structure configuration will need to be input into the final models.

UNSTEADY STATE BACKWATER ANALYSES WITH OPERATIONAL REQUIREMENTS

In order to properly design the low water dam structures and their operational requirements, an unsteady state backwater model, such as HEC-RAS version 4.0 or MIKE21 will be needed. Either of these models will allow the modeling of river flows over time. The modeling will allow the determination of the actual number of gates to be installed at each structure and the number and amount of gate openings at each structure for given operational scenarios.

APPENDIX A

ARKANSAS RIVER CORRIDOR STUDY MASTER PLAN H&H ANALYSES VERIFICATION PLOTS

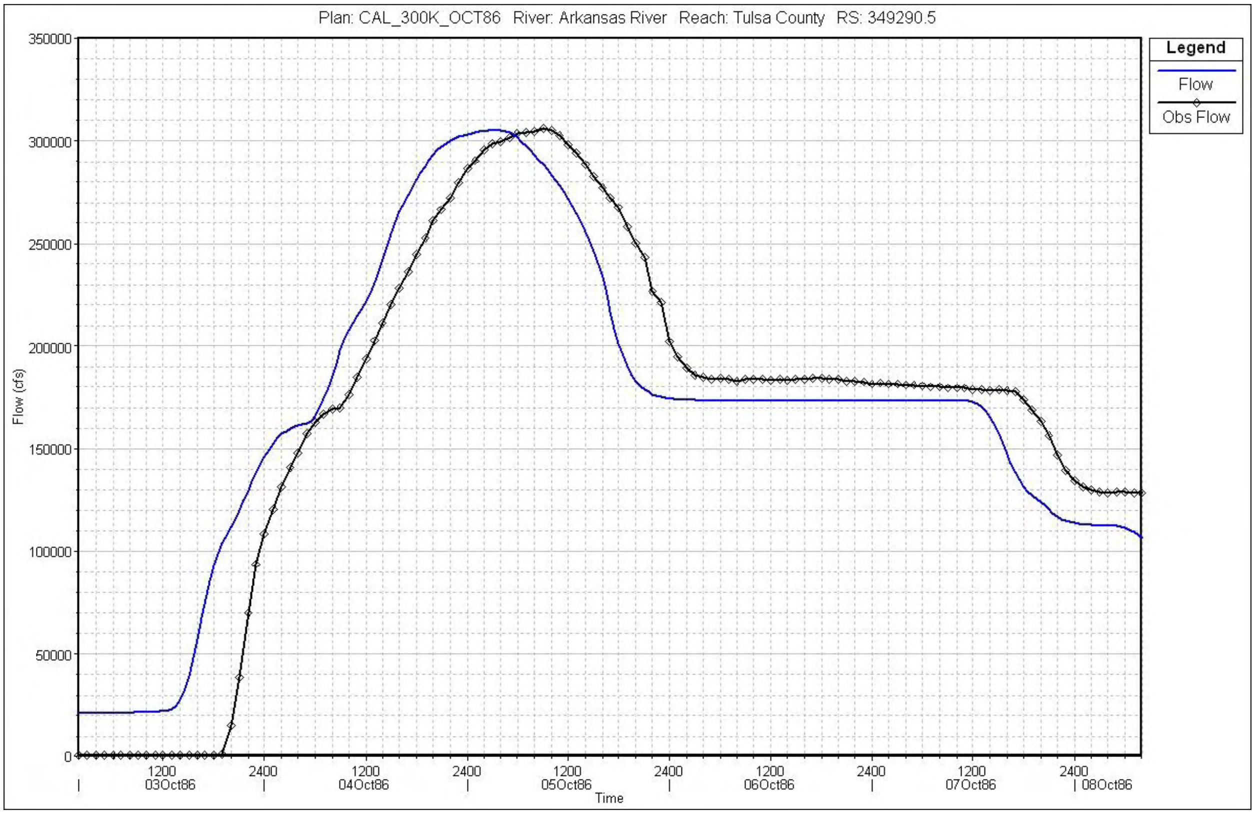


FIGURE A-1 OCT 1986 FLOW HYDROGRAPH VERIFICATION

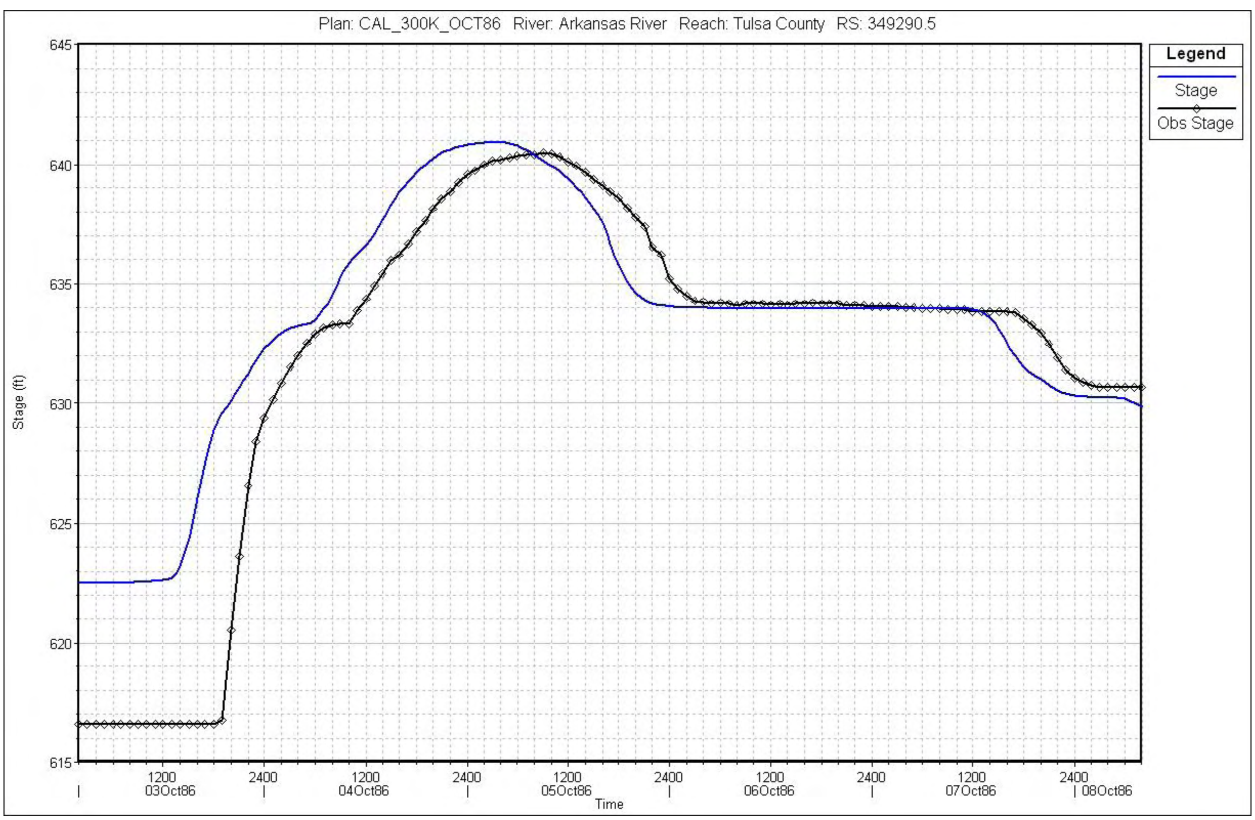


FIGURE A-2 OCT 1986 STAGE HYDROGRAPH VERIFICATION

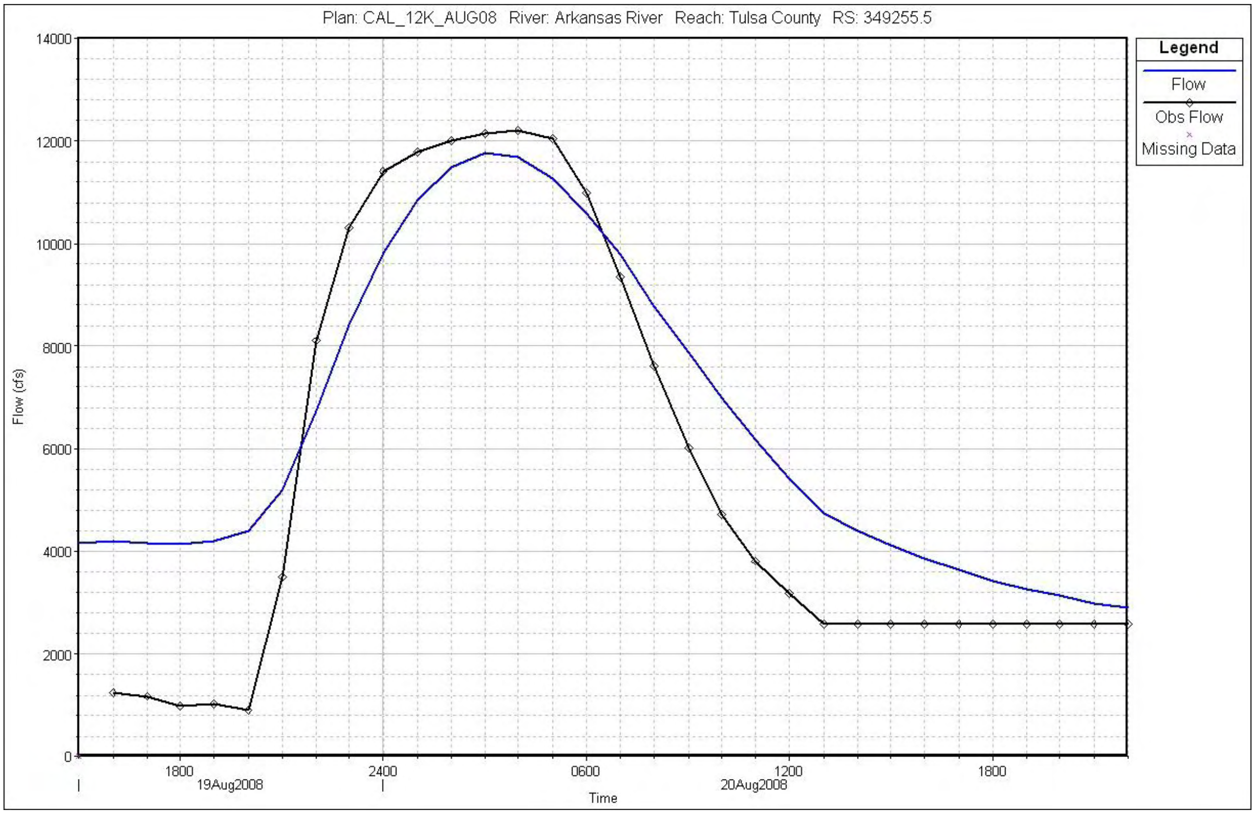


FIGURE A-3 AUG 2008 FLOW HYDROGRAPH VERIFICATION

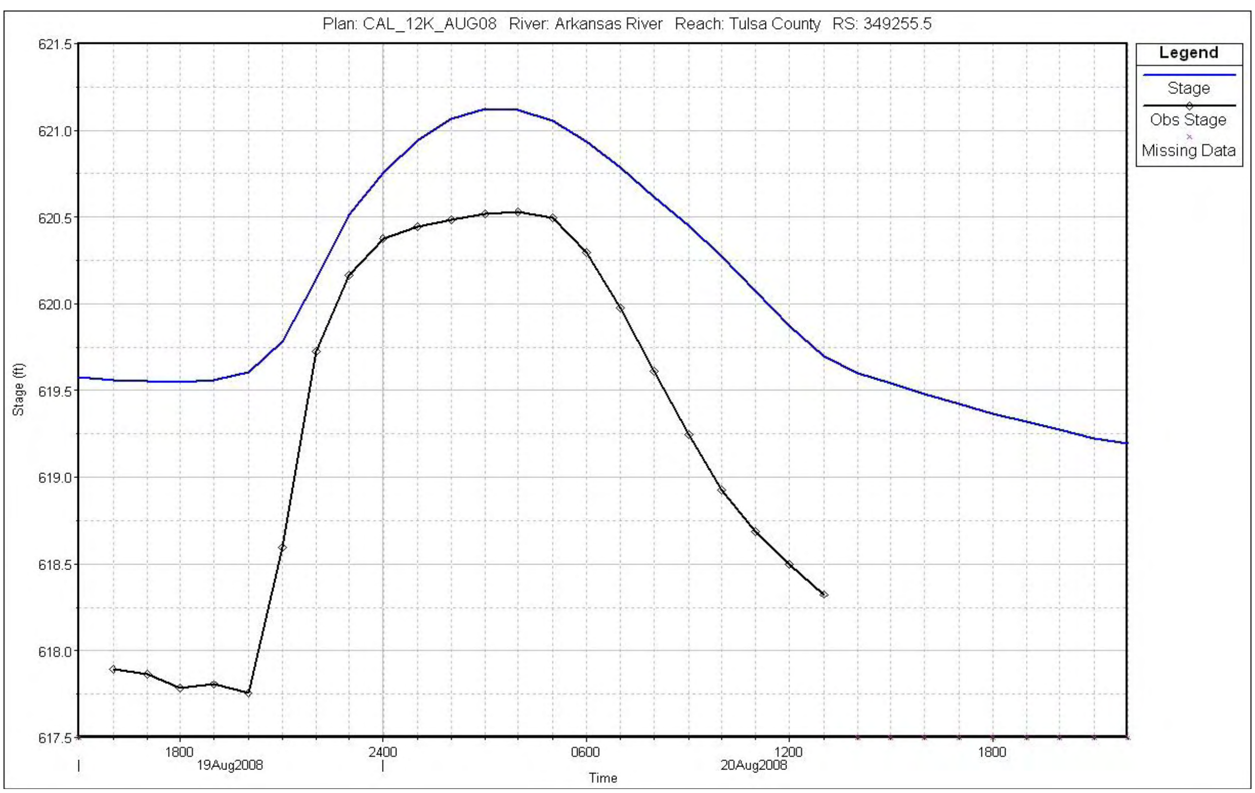


FIGURE A-4 AUG 2008 STAGE HYDROGRAPH VERIFICATION

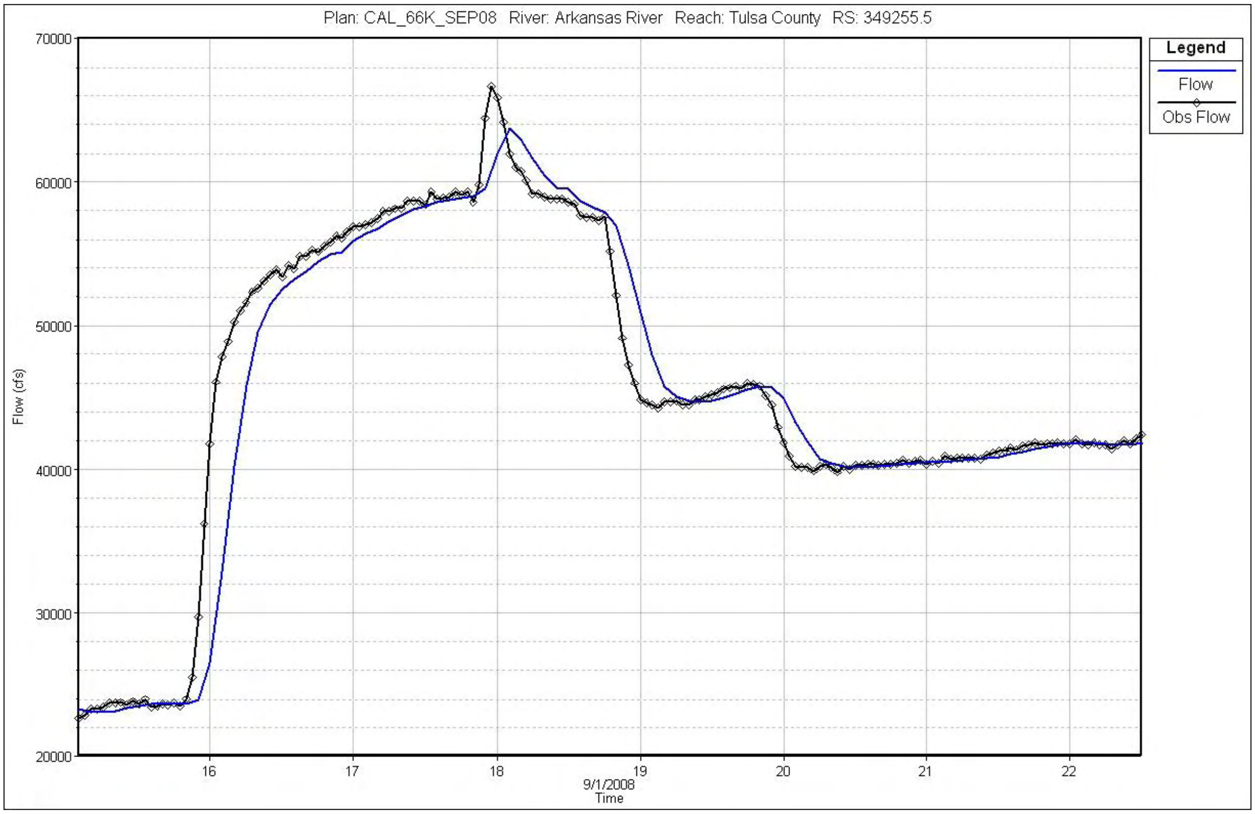


FIGURE A-5 SEP 2008 FLOW HYDROGRAPH VERIFICATION

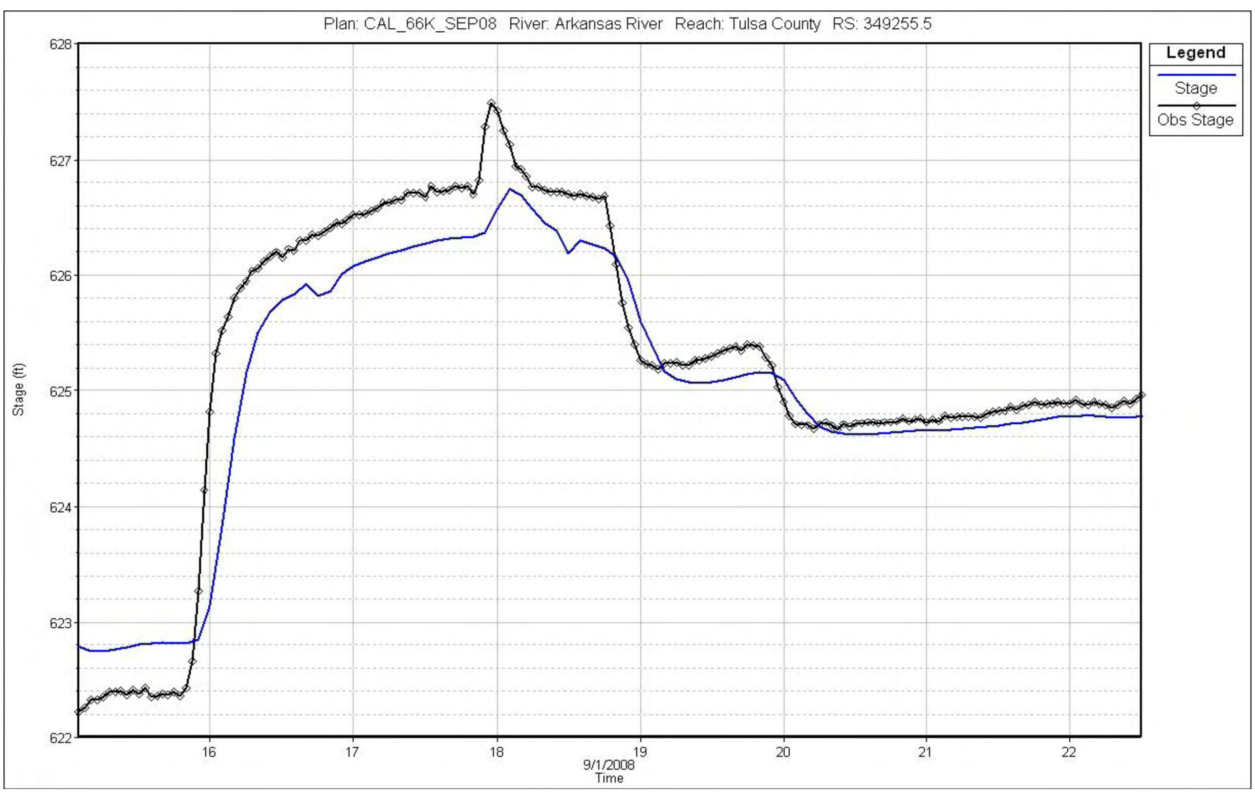


FIGURE A-6 SEP 2008 STAGE HYDROGRAPH VERIFICATION

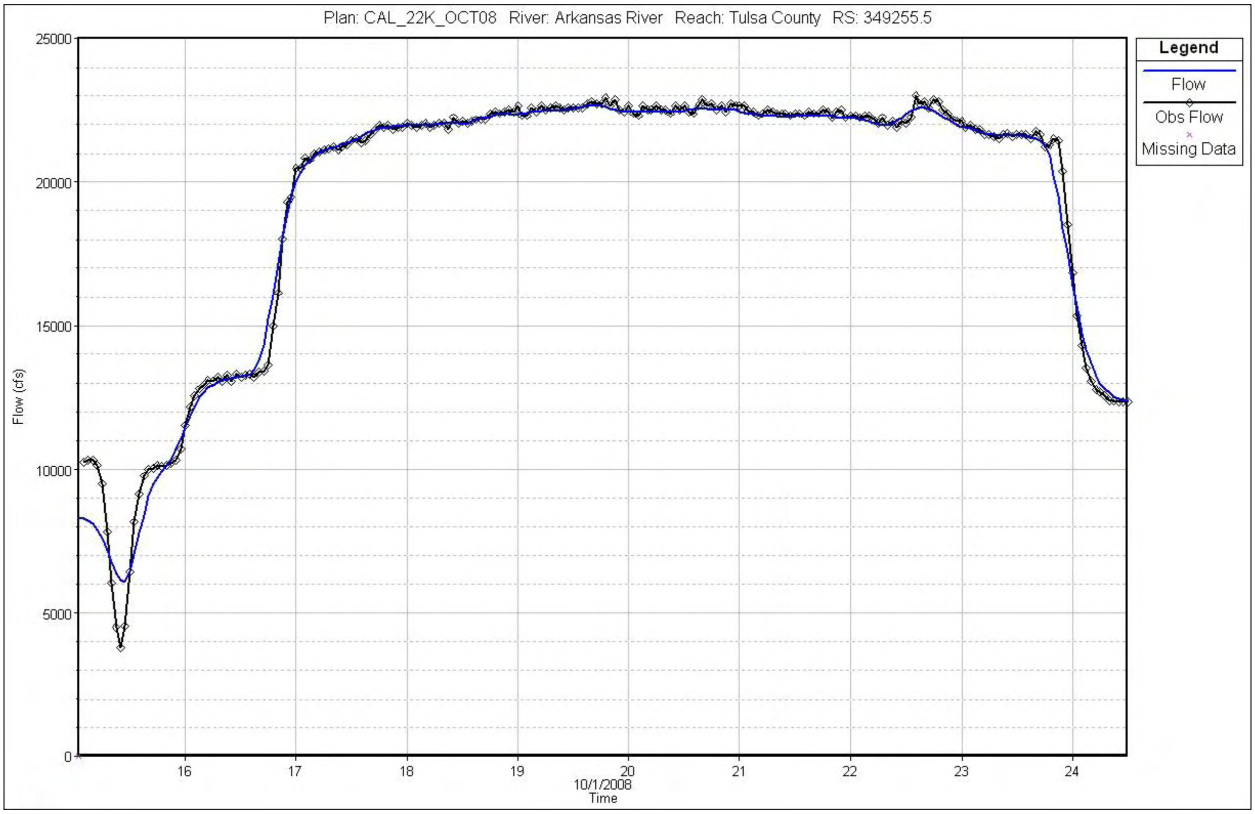


FIGURE A-7 OCT 2008 FLOW HYDROGRAPH VERIFICATION

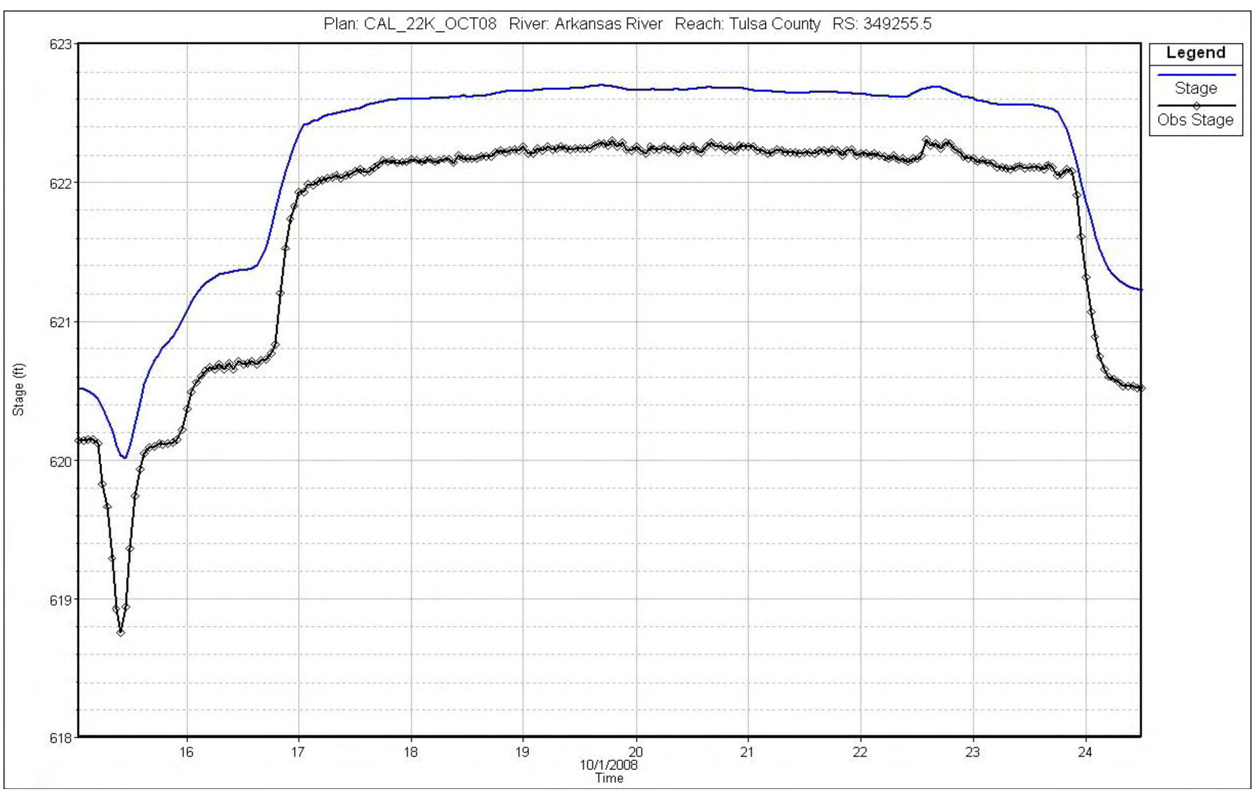


FIGURE A-8 OCT 2008 STAGE HYDROGRAPH VERIFICATION

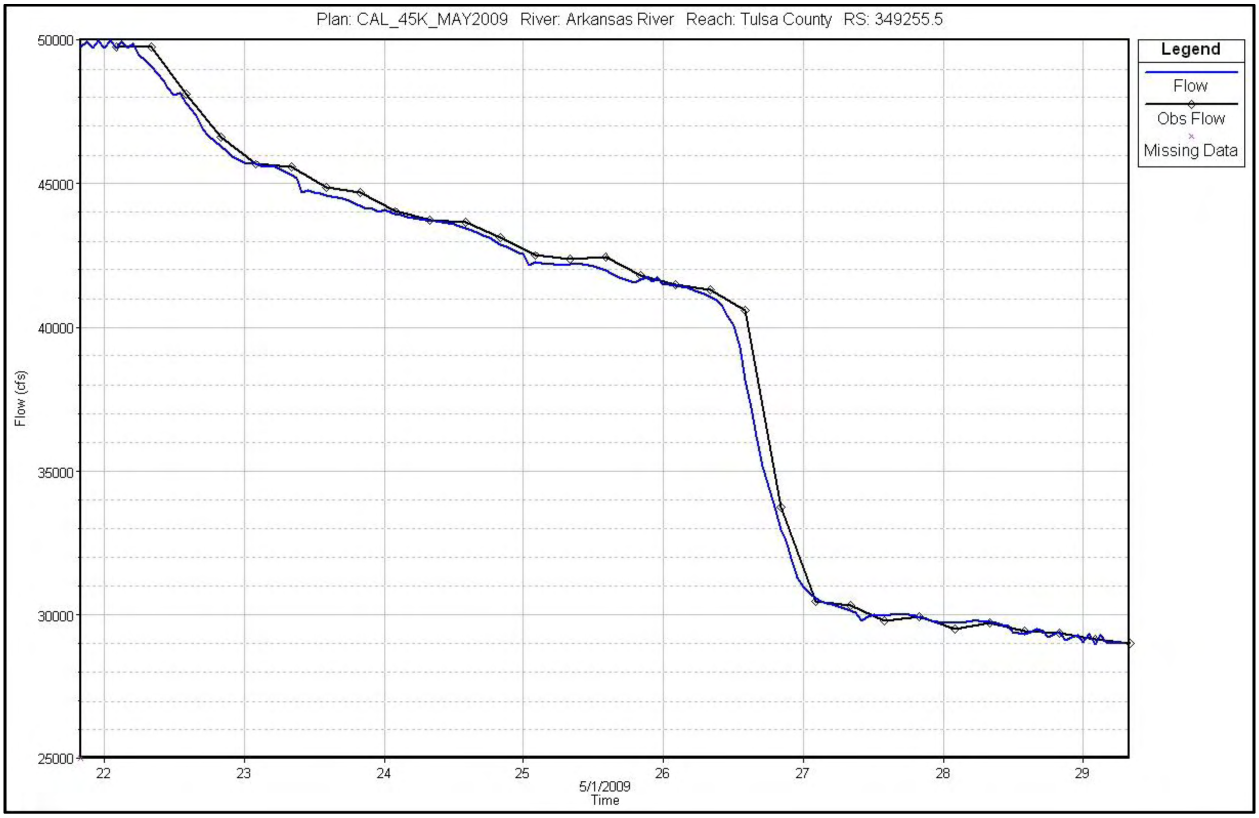


FIGURE A-9 MAY 2009 FLOW HYDROGRAPH VERIFICATION

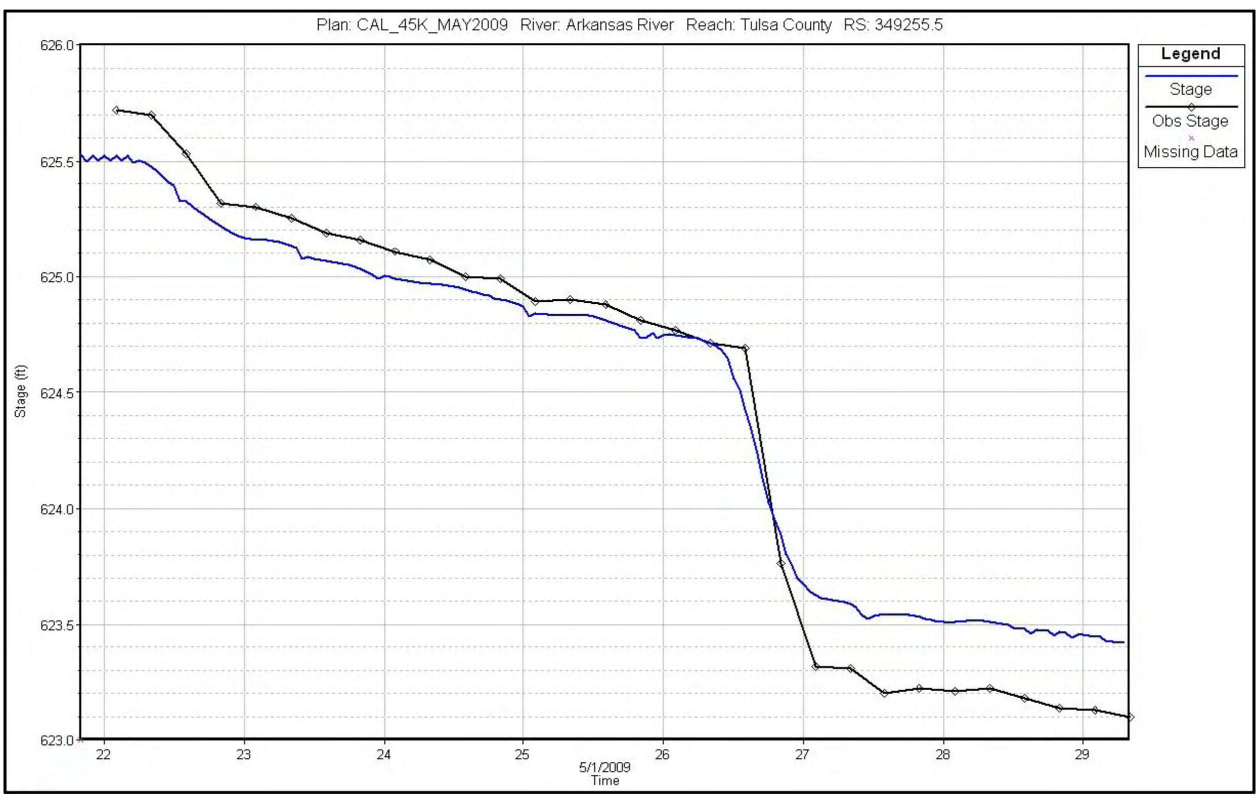


FIGURE A-10 MAY 2009 FLOW HYDROGRAPH VERIFICATION

APPENDIX B

ARKANSAS RIVER CORRIDOR STUDY MASTER PLAN H&H ANALYSES
FLOW HYDROGRAPHS FOR STUDY SCENARIOS

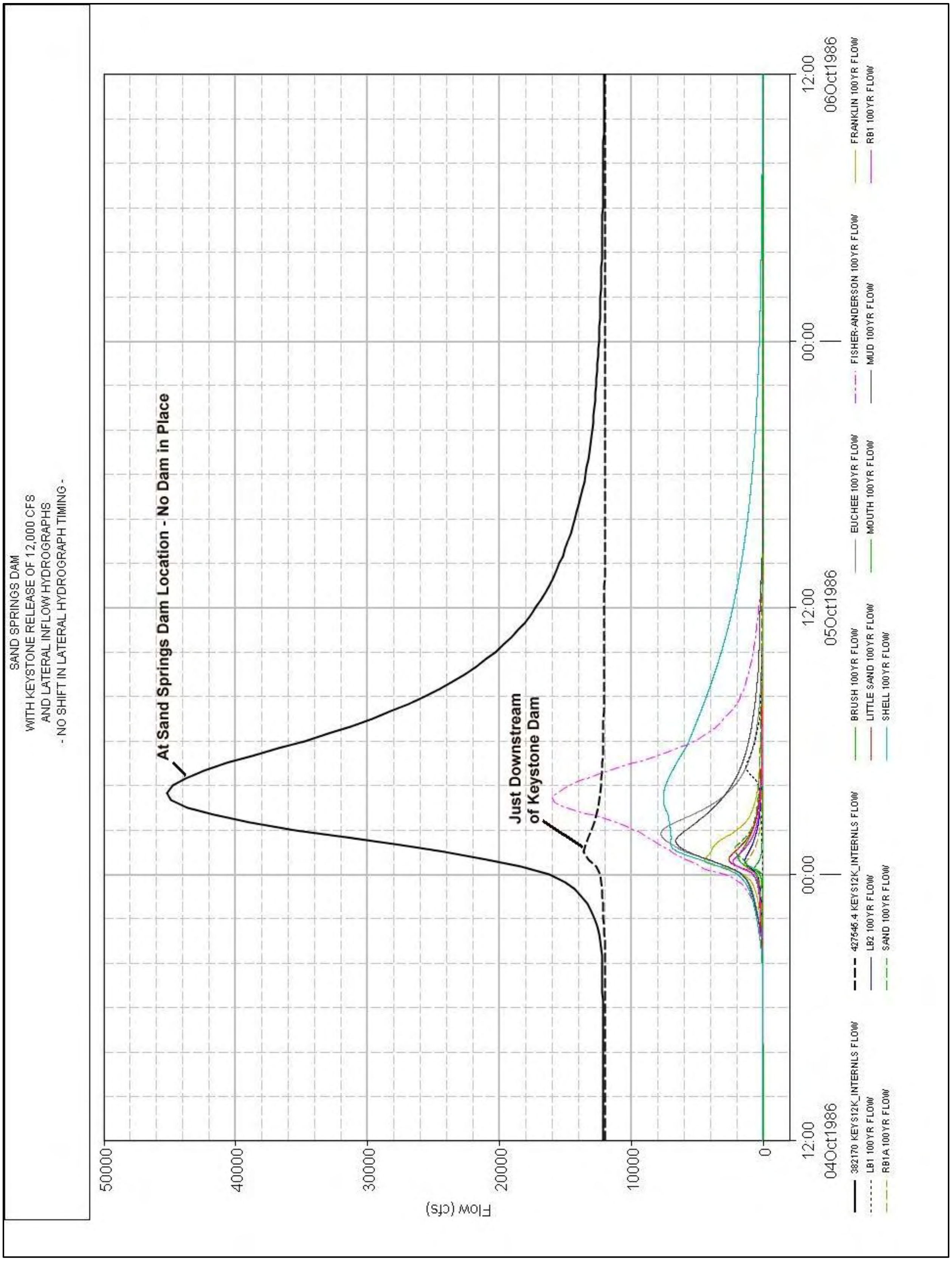


FIGURE B-1. Proposed Sand Springs Dam Location with Lateral Tributary Hydrographs Added

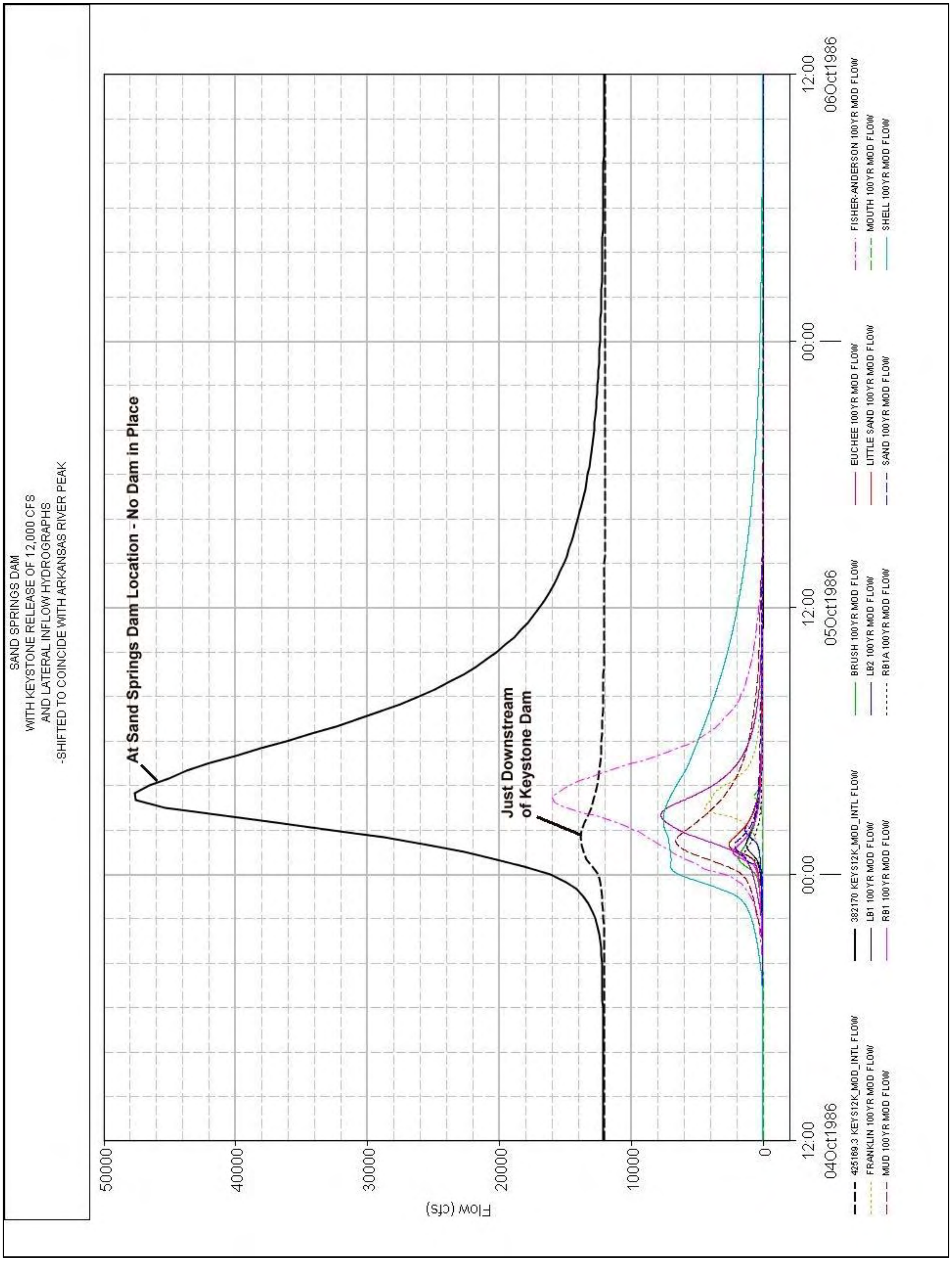


FIGURE B-2. Proposed Sand Springs Dam Location with Lateral Tributary Hydrographs Shifted

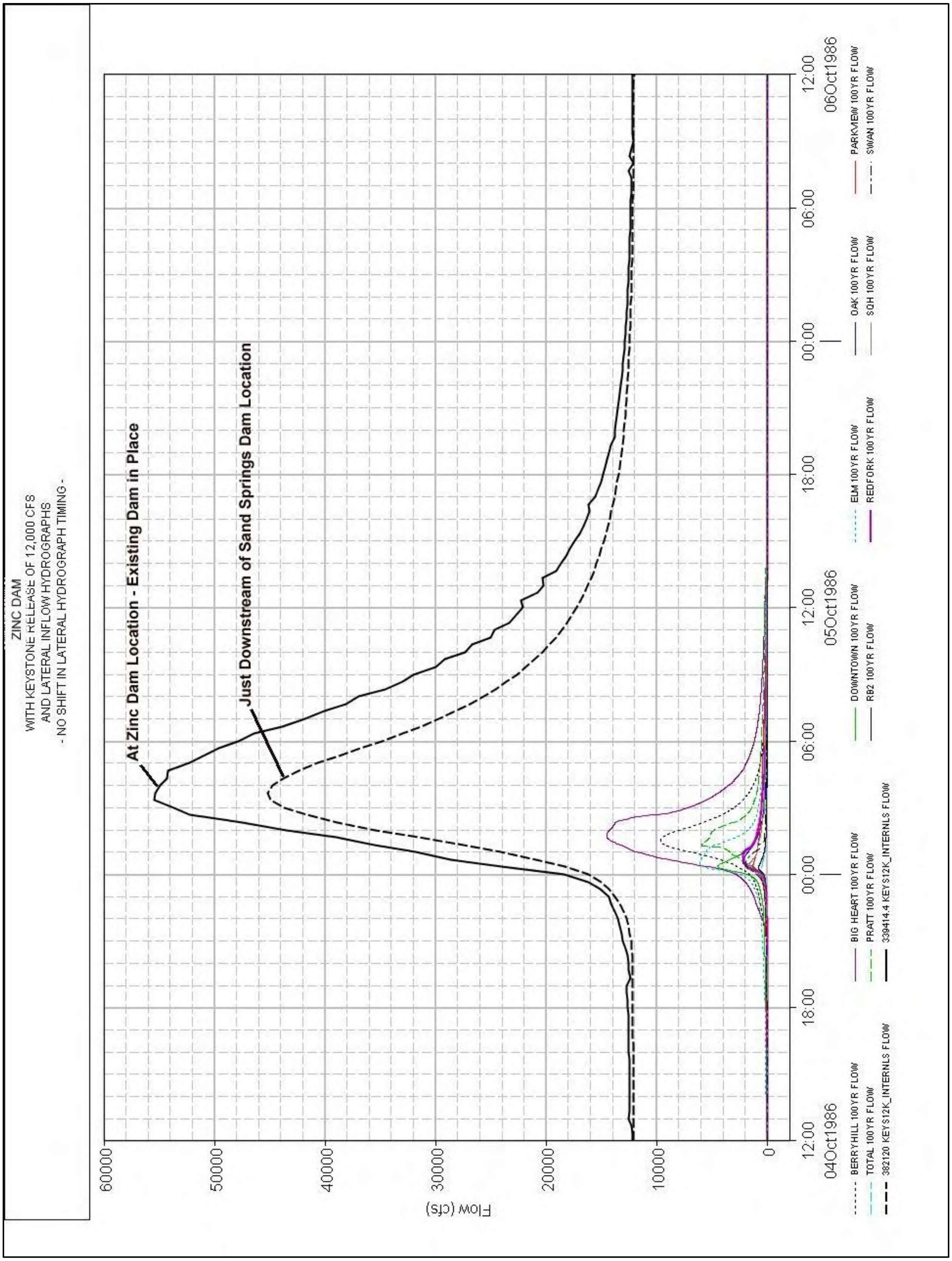


FIGURE B-3. Existing Zink Dam Location with Lateral Tributary Hydrographs Added

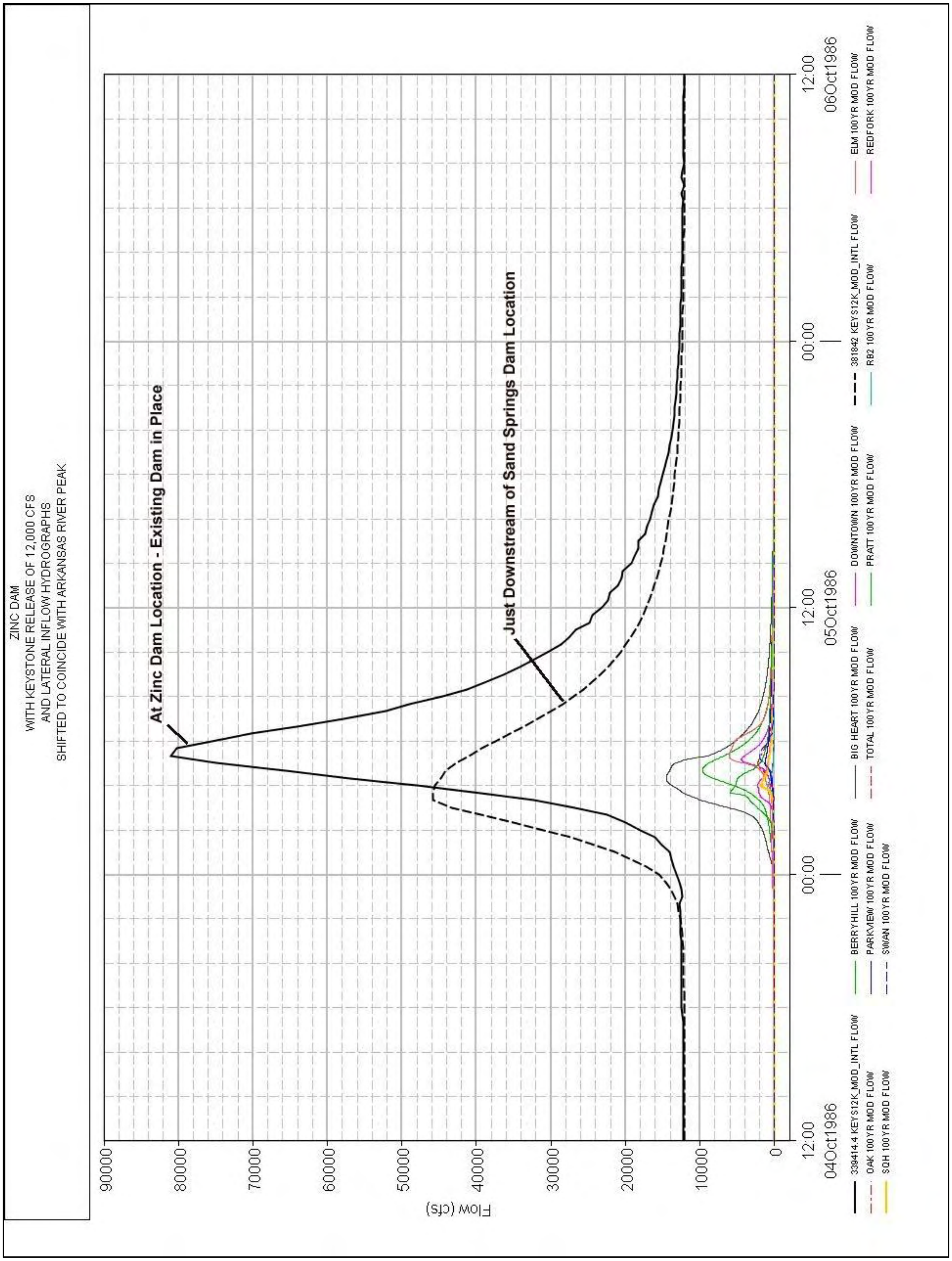


FIGURE B-4. Existing Zink Dam Location with Lateral Tributary Hydrographs Shifted

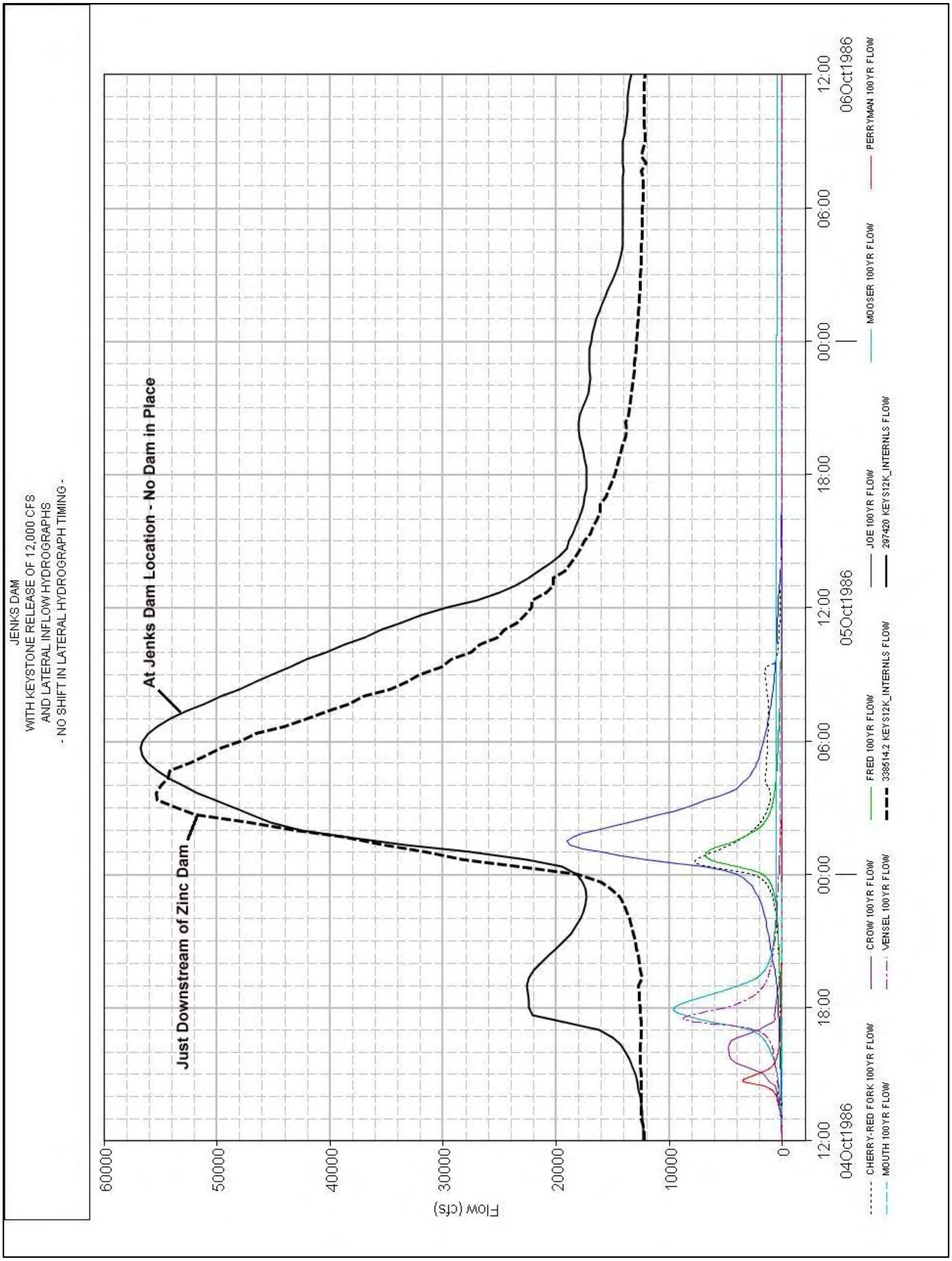


FIGURE B-5. Proposed Jenks Dam Location with Lateral Tributary Hydrographs Added

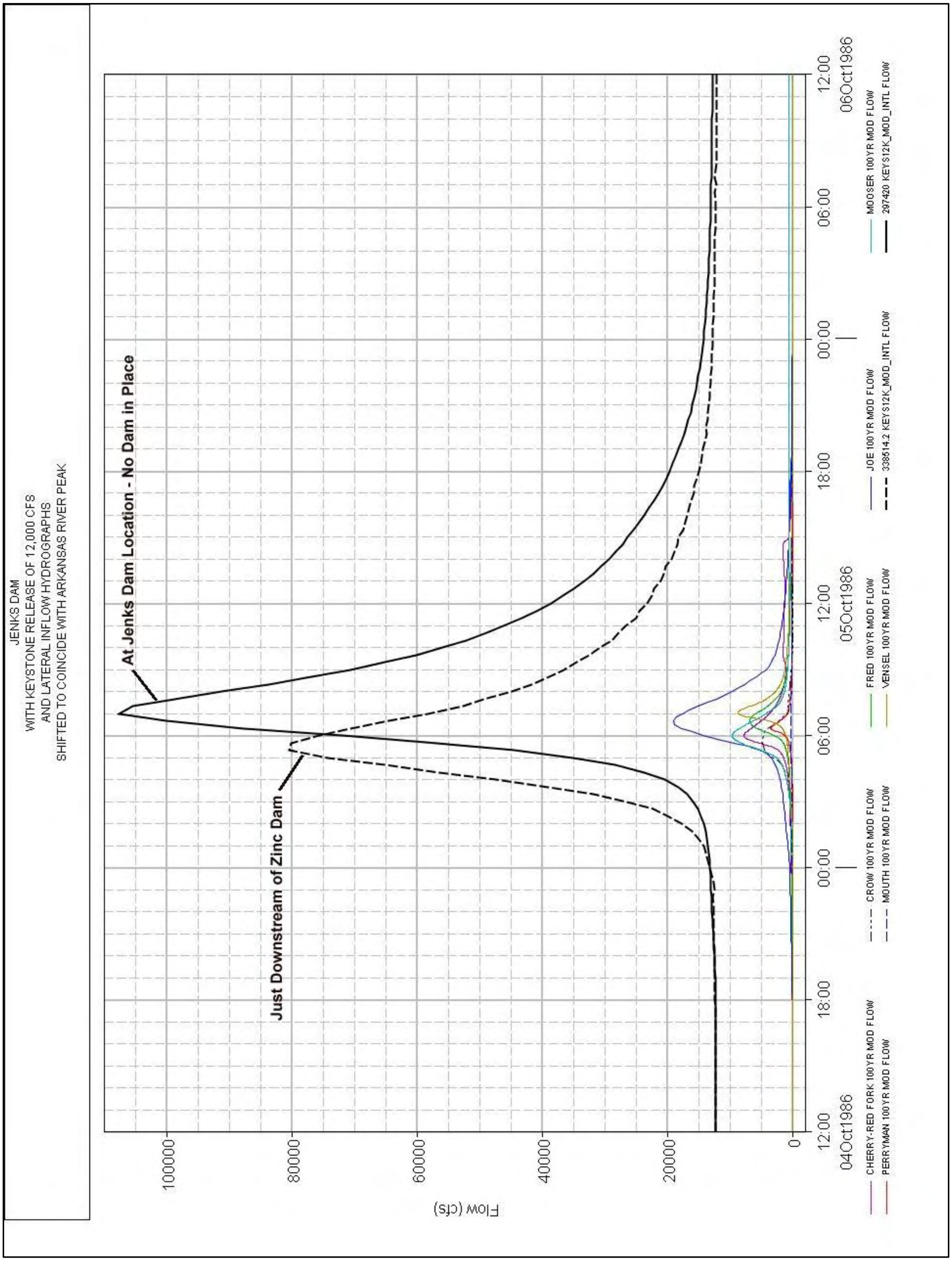


FIGURE B-6. Proposed Jenks Dam Location with Lateral Tributary Hydrographs Shifted

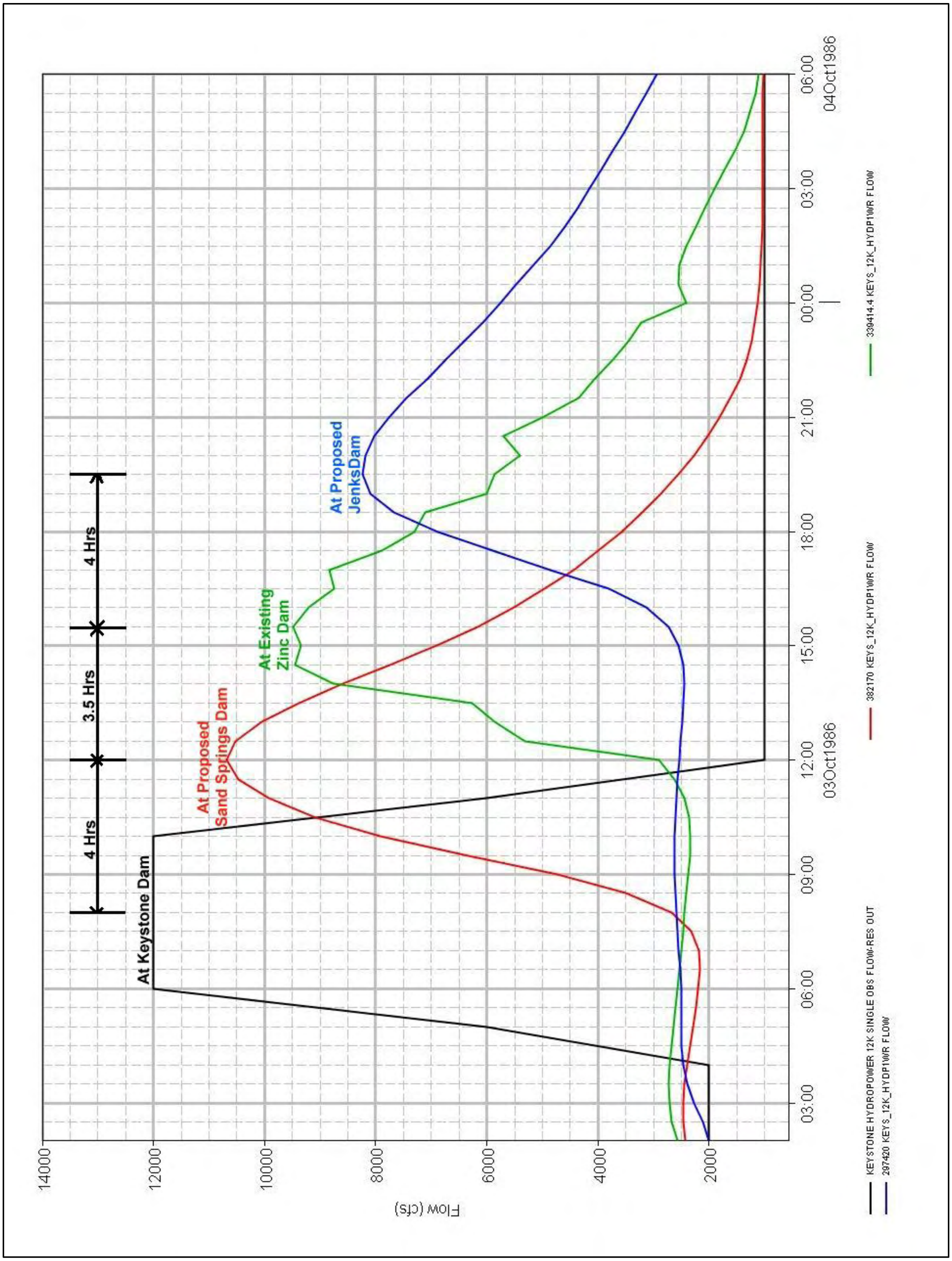
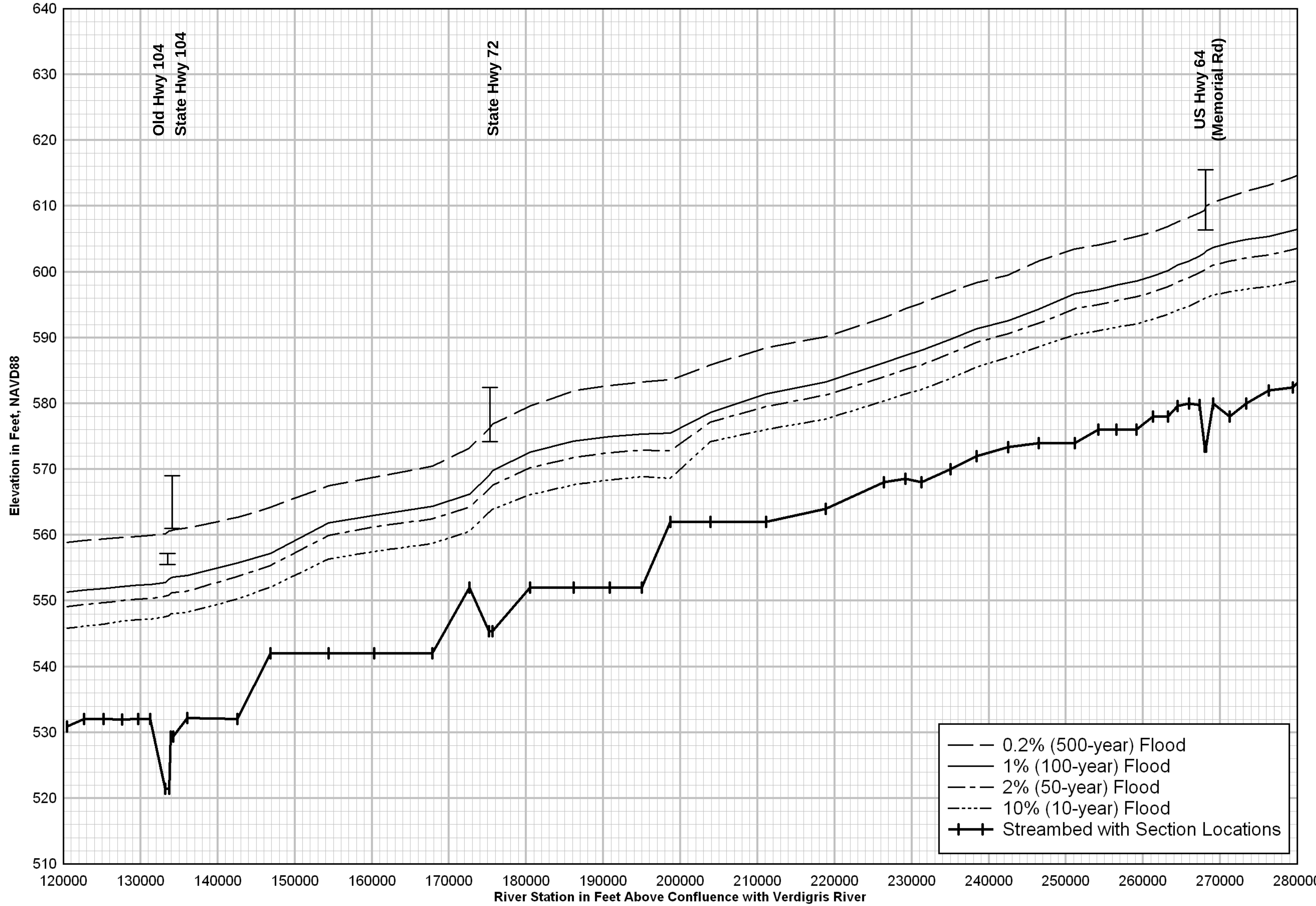


FIGURE B-7. Hydrograph Travel Time and Attenuation

APPENDIX C

ARKANSAS RIVER CORRIDOR STUDY MASTER PLAN H&H ANALYSES STEADY STATE AND UNSTEADY STATE BACKWATER PROFILES

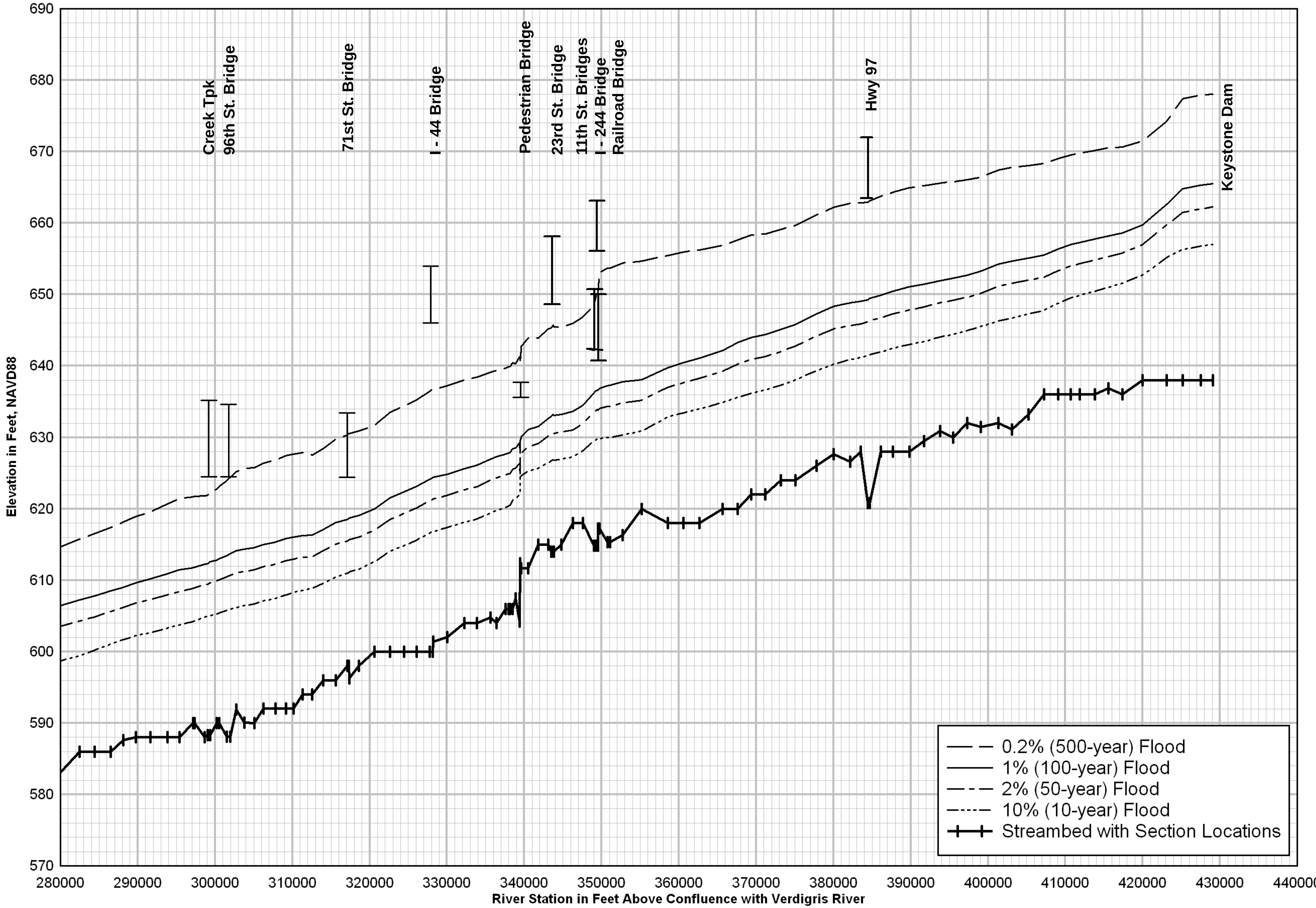


FLOOD PROFILES - DUPLICATE EFFECTIVE MODEL

ARKANSAS RIVER

ARKANSAS RIVER CORRIDOR
MASTER PLAN
(TULSA CO.)

01P



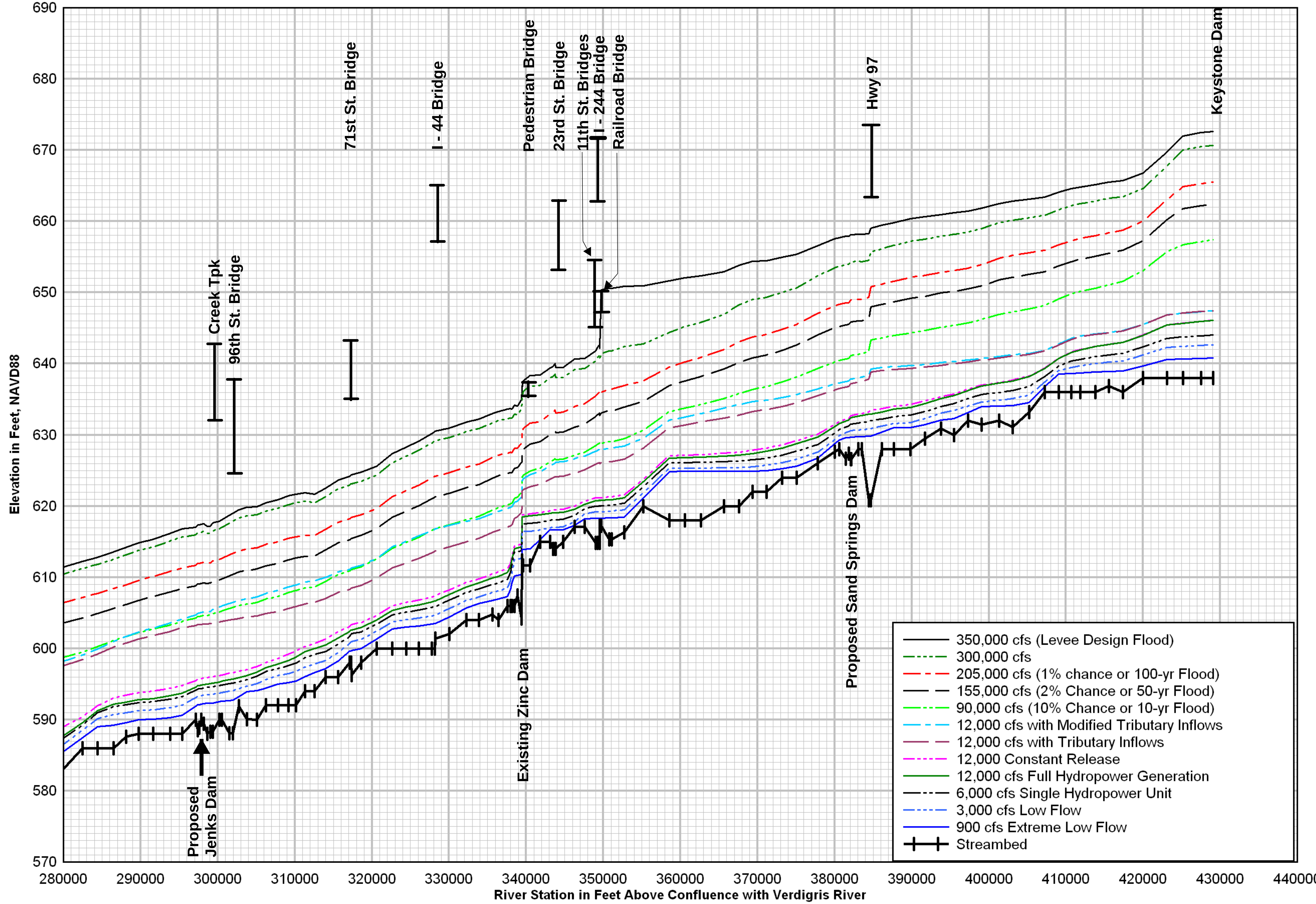
FLOOD PROFILES - DUPLICATE EFFECTIVE MODEL

ARKANSAS RIVER

ARKANSAS RIVER CORRIDOR MASTER PLAN

(TULSA CO.)

02P



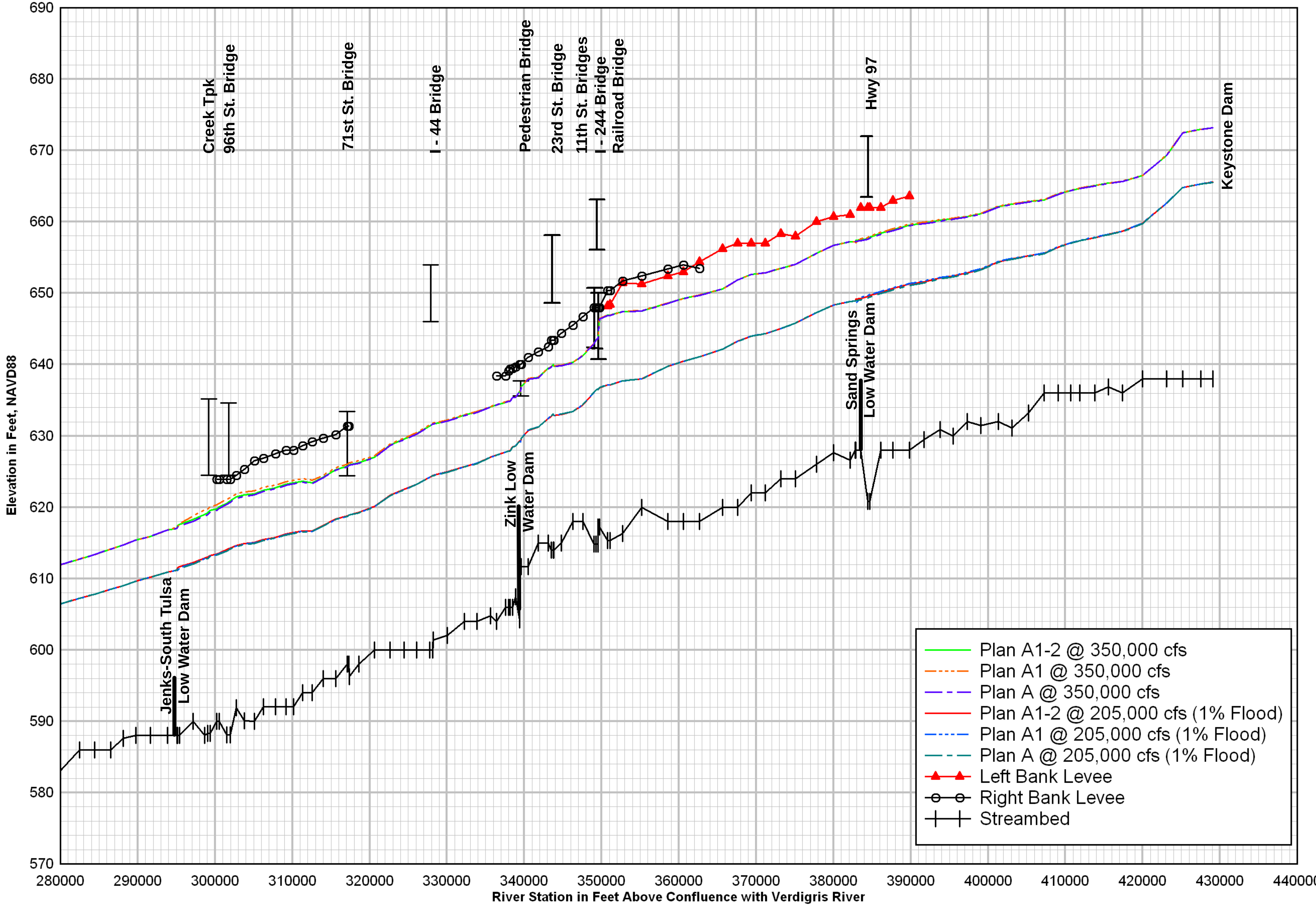
FLOOD PROFILES - UNSTEADY FLOW MODEL

ARKANSAS RIVER

ARKANSAS RIVER CORRIDOR MASTER PLAN

(TULSA CO.)

03P



FLOOD PROFILES - COMPARISON OF PLANS

ARKANSAS RIVER

ARKANSAS RIVER CORRIDOR MASTER PLAN

(TULSA CO.)

04P

APPENDIX D

ARKANSAS RIVER CORRIDOR STUDY MASTER PLAN H&H ANALYSES
FLOW FREQUENCY AND DURATION

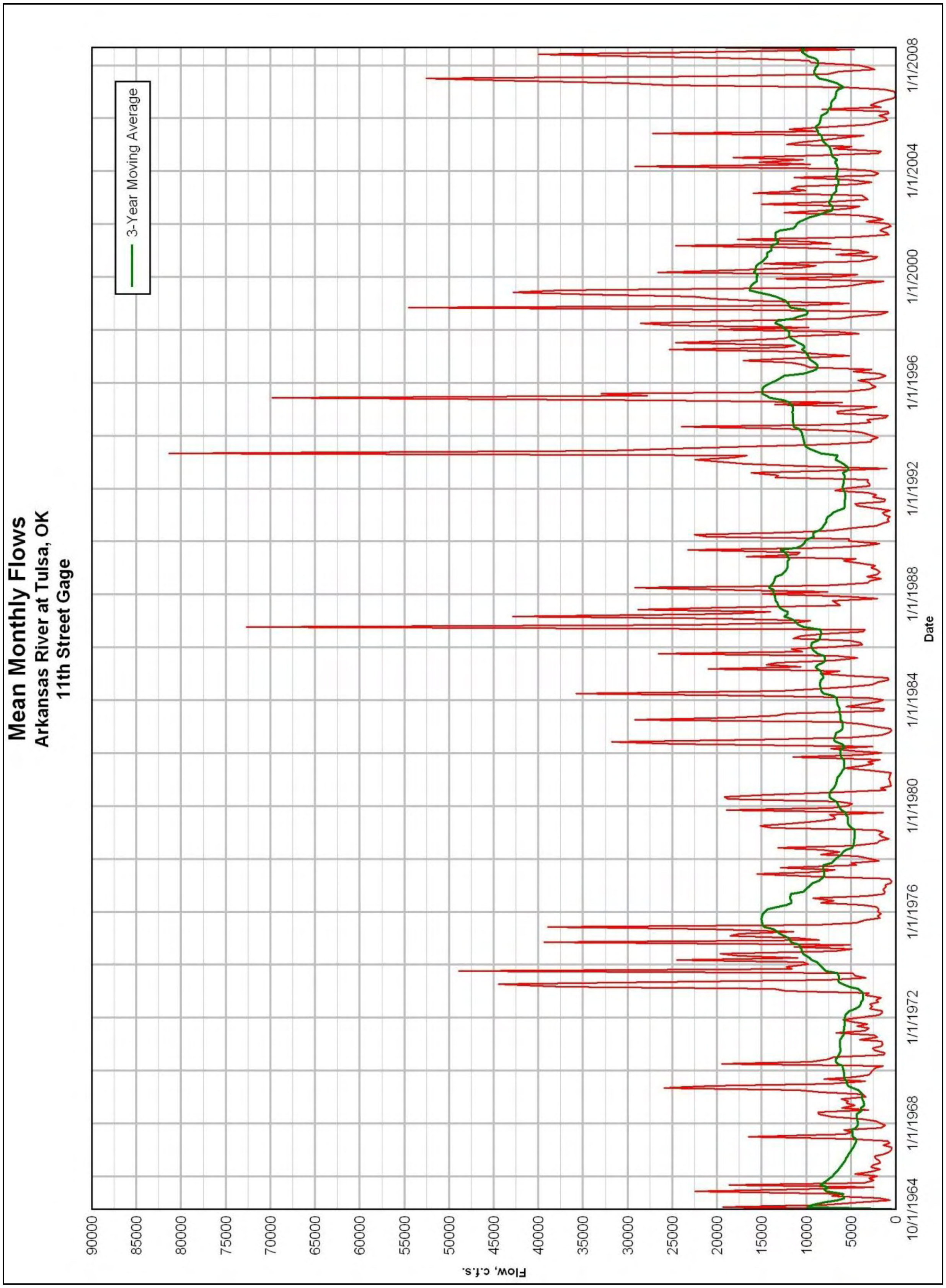


FIGURE D-1. Arkansas River, 11th Street Gage, Mean Monthly Flows

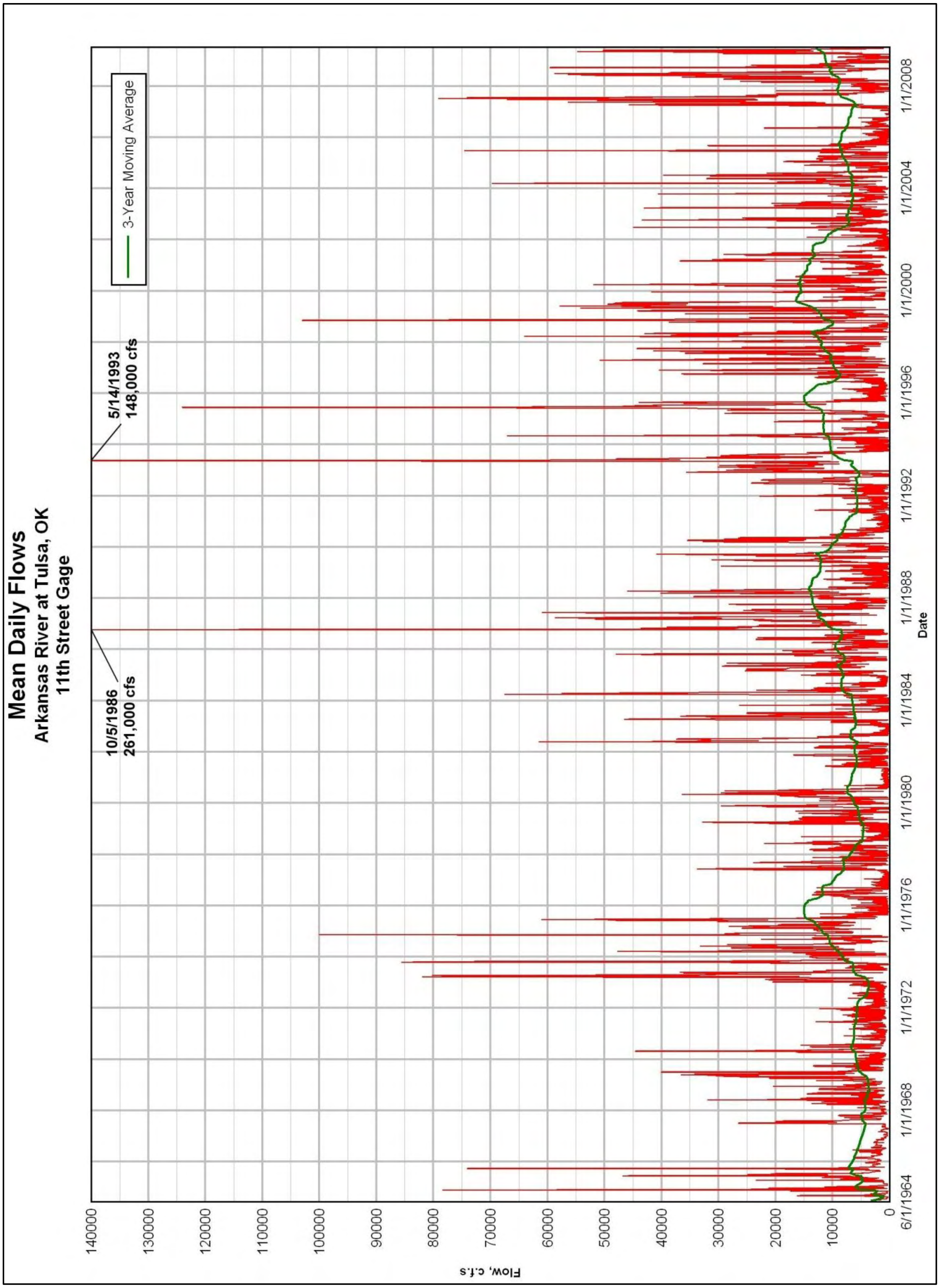


FIGURE D-2. Arkansas River, 11th Street Gage, Mean Daily flows

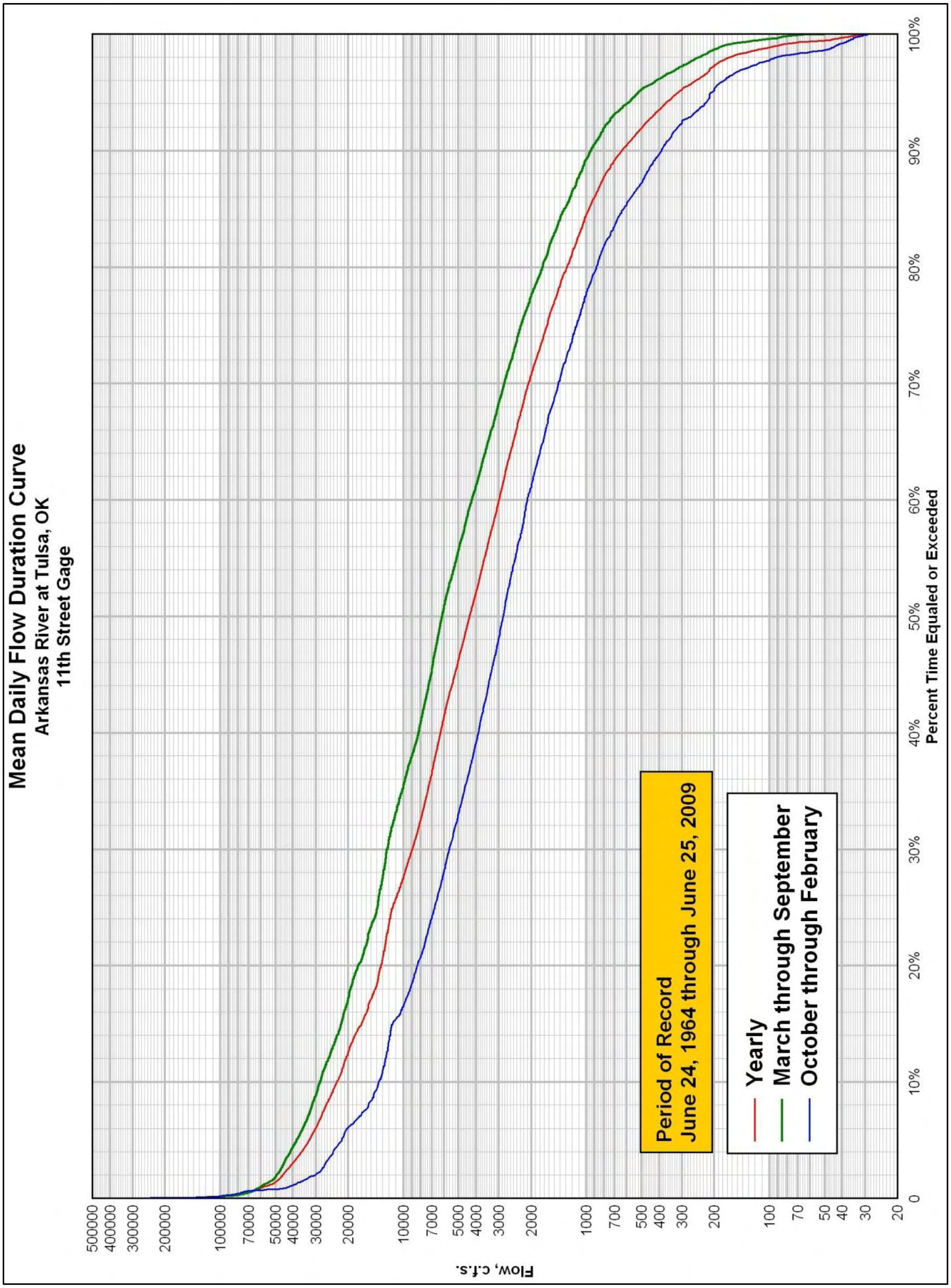


FIGURE D-3. Arkansas River, 11th Street Gage, Mean Monthly Flow Duration Curve

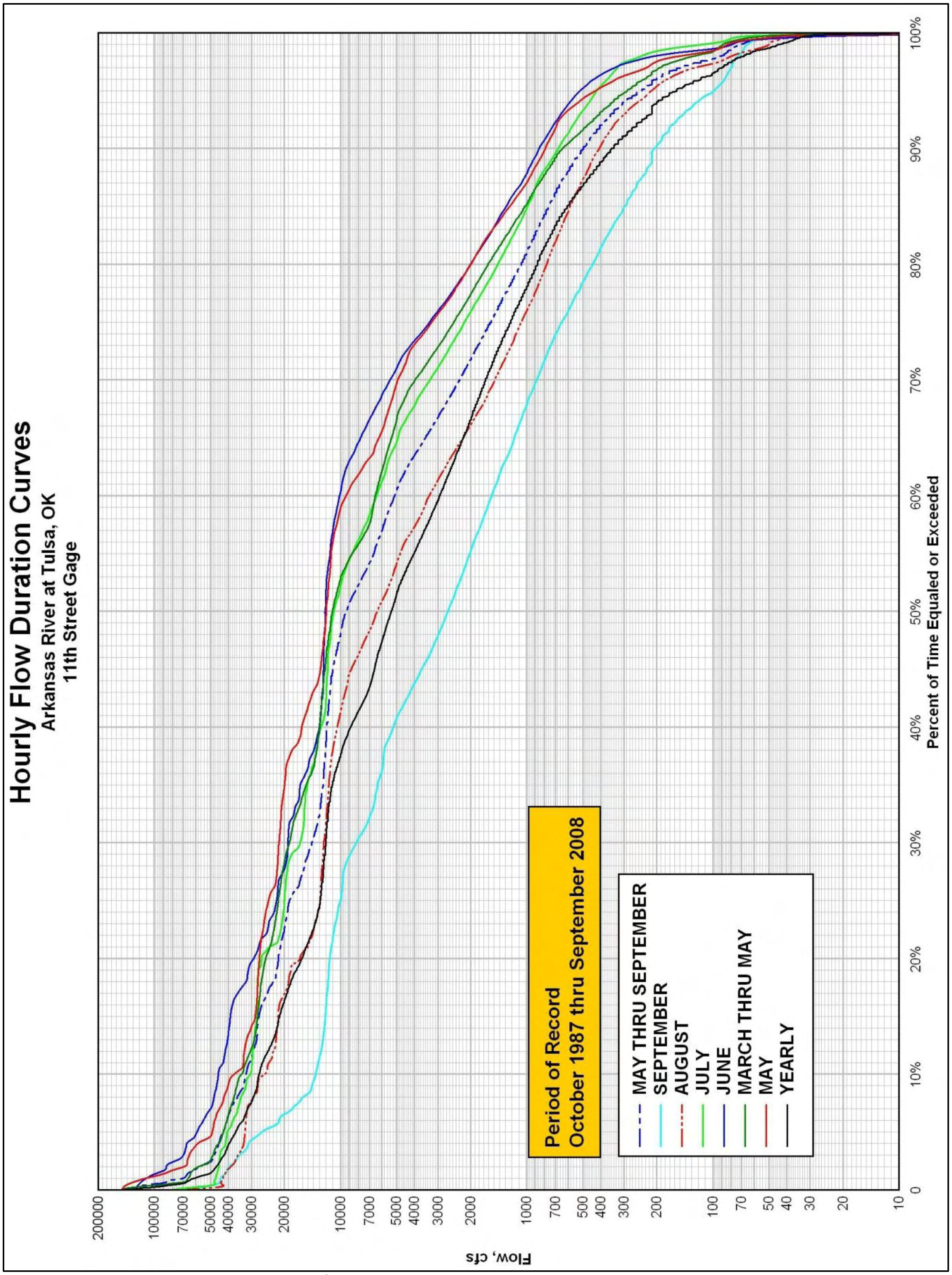


FIGURE D-4. Arkansas River, 11th Street Gage, Mean Hourly Flow Duration Curve