RISK BASED ANALYSIS
FOR
THE AMERICAN RIVER WATERSHED PROJECT

by
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HISTORY

Sacramento was established in the 1840's at the confluence of the Sacramento and American Rivers. Flooding was common and regular as the community grew. Over the years, a complex system of levees, bypasses and dams and reservoirs were constructed to help reduce the damage caused by flooding. Still, the city is at great risk today. There are nearly 400,000 people and $37 billion of damageable property protected by 15 to 20 foot high levees. Many of these levees were built over 50 years ago. In February 1986, the "storm of record" occurred in the American River Basin and revealed the true nature of the flood control capabilities of the system. The system was taxed for nearly seven days with channel and levee capacity flows. As a result of this flood and subsequent hydrologic studies, the 1986 event showed that Sacramento has a modest level of protection at best - substantially below the 100-year threshold for the national flood insurance program.

SYSTEM

Sacramento is bound on the west by the Sacramento River and bisected by the American River which flows east from the Sierra Nevada Mountains. The American River watershed covers approximately 2,100 square miles and includes portions of Placer, El Dorado and Sacramento Counties. The basin is partially regulated on the North Fork of the American River by several small reservoirs and on the main stem by Folsom Dam. The Folsom Dam and Reservoir, located about 29 miles upstream of Sacramento**,** are key features in the flood control system (See Figure 1). The levees along the American River downstream of Folsom Dam are likely to fail at various locations when sustained flows reach between 130,000 cfs and 160,000 cfs. The risk or probability of failure, given a 100-year event, is approximately 0.60.

ALTERNATIVES

The goal of the American River Watershed Project is to significantly increase the level of flood protection for Sacramento. The local sponsor indicates that a flood control alternative implemented in the Sacramento area should provide nothing less than a 200-year level of protection. Seventeen individual measures were identified as possible configurations for project alternatives. These measures were arranged to compile an array of eight possible flood control alternatives(See Table 1). The nominated alternatives for evaluation fall into three basic categories.

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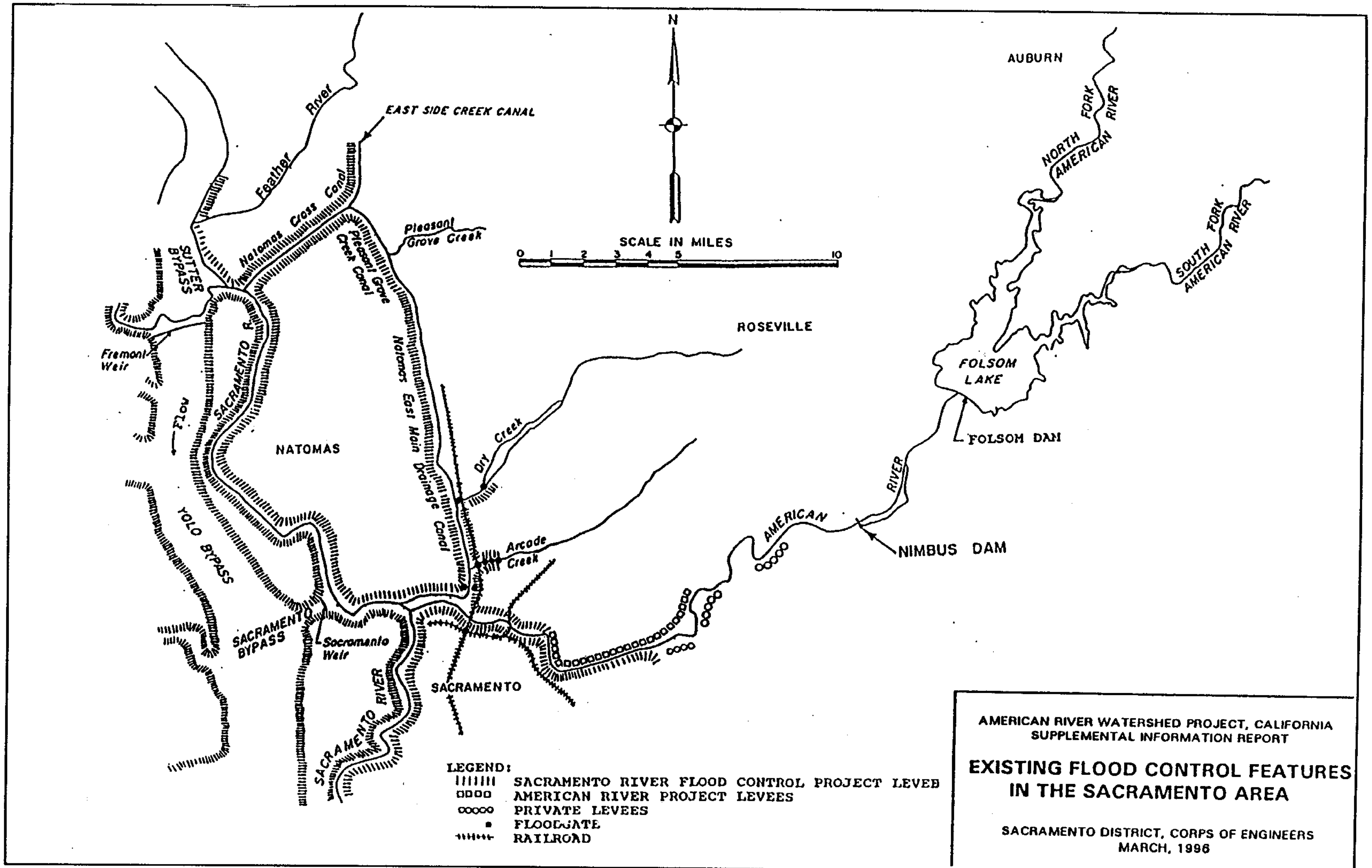


Figure 1 - Sacramento Area Flood Control Features

The categories include a flood detention dam on the North Fork of the American River just upstream of Folsom Dam, operational and structural modification to Folsom Dam, larger flood control releases from Folsom Dam requiring modification to the downstream flood control system, and the use of existing upstream storage. The final candidate plans that represent these three categories are the Folsom Modification Plan, the Folsom Stepped Release Plan and the Detention Dam Plan.

RISK-BASED ANALYSIS

The Risk-Based Analysis (RBA) procedures developed by the U.S. Army Corps of Engineers' Hydrologic Engineering Center (HEC), Davis, Ca., were used to evaluate the economic benefits and the hydrologic performance of each alternative. In addition to the standard methods and tools, HEC customized software to accommodate the analysis of a regulated system, and index points

Table 1 - Summary of Initial Alternatives

Alternatives	Primary Alternative Features
Minimum Impact [1] Folsom Modification Plan	Increase flood control space, surcharge space, modify Folsom outlets, minor change to objective release and downstream channel capacity (115,000 cfs).
Minimum Objective Release	Increase flood control space, surcharge space, modify Folsom outlets, minor change to objective release and downstream channel capacity (130,000 cfs).
Moderate Objective Release	Increase flood control space, surcharge space, modify Folsom outlets, moderate change to objective release and downstream channel capacity (145,000 cfs).
Maximum Objective Release	Increase flood control space, surcharge space, modify Folsom outlets, major change to objective release and downstream channel capacity (180,000 cfs).
Stepped Release [1]	Increase surcharge control space, surcharge space, modify Folsom outlets,major change to objective release and downstream channel capacity (145-180,000 cfs).
200-Year Storage	Construct a 380,000 ac-ft flood detention dam upstream from Folsom Reservoir.
Equivalent Storage	Construct a 545,000 ac-ft flood detention dam upstream from Folsom Reservoir.
Detention Dam Plan [1]	Construct a 894,000 ac-ft flood detention dam upstream from Folsom

[1] Final candidate plans

where the use of stage-frequency data is more appropriate than discharge-frequency and stage-discharge data. Nine index points were used to evaluate existing and project conditions over the study area (See Figure 2). Two of the index points were used to determine expected annual damage (EAD) and all nine were used to evaluate the reliability of levee performance (Reliability). Figure 3 displays the performance matrix used to present the RBA results for the base condition and for each alternative. By index point, each matrix presents the Reliability afforded by a given levee for the base condition (R_x), as well as the Reliability for that alternative (R_b). Additionally, the matrix presents

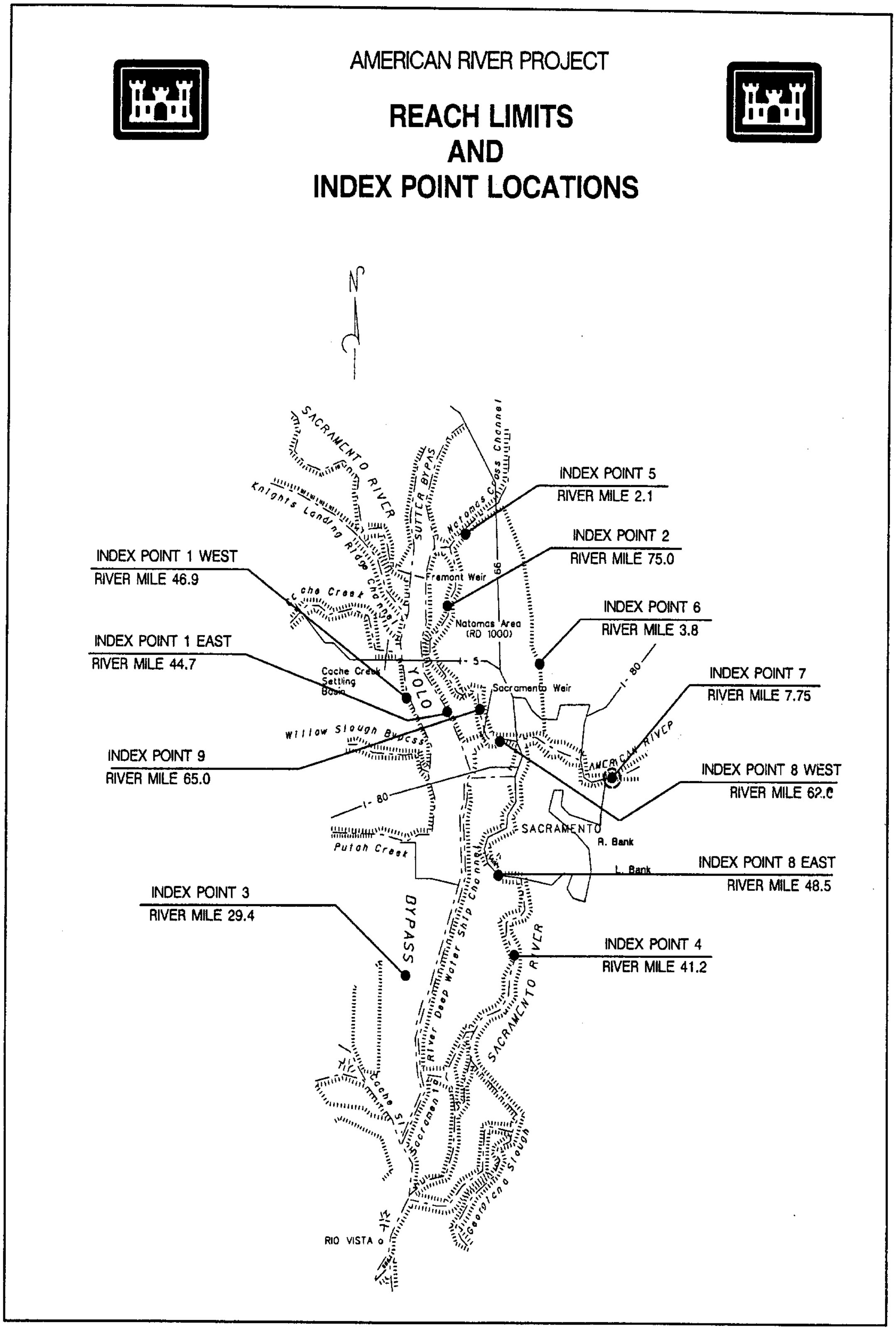


Figure 2 - Index Point Locations

PERFORM. INDEX POINT 1

Event	Percent Chance of Non-Exceedance			
	R_x Left	R_x Right	R_b Left	R_b Right
400 yr	19.8	16.7	12.4	12.3
200 yr	23.9	21.8	21.7	21.6
100 yr	45.0	41.1	35.8	35.8
50 yr	71.4	66.2	50.2	52.4

PNP Left = +2.0' PNP Right = 1.5'

PERFORM. INDEX POINT 2

Event	Percent Chance of Non-Exceedance			
	R_x Left	R_x Right	R_b Left	R_b Right
400 yr	42.9	11.0	31.7	11.0
200 yr	57.4	18.1	45.1	18.1
100 yr	84.3	39.3	73.2	39.3
50 yr	99.7	73.3	97.3	73.3
TE (Yr)	217		154	

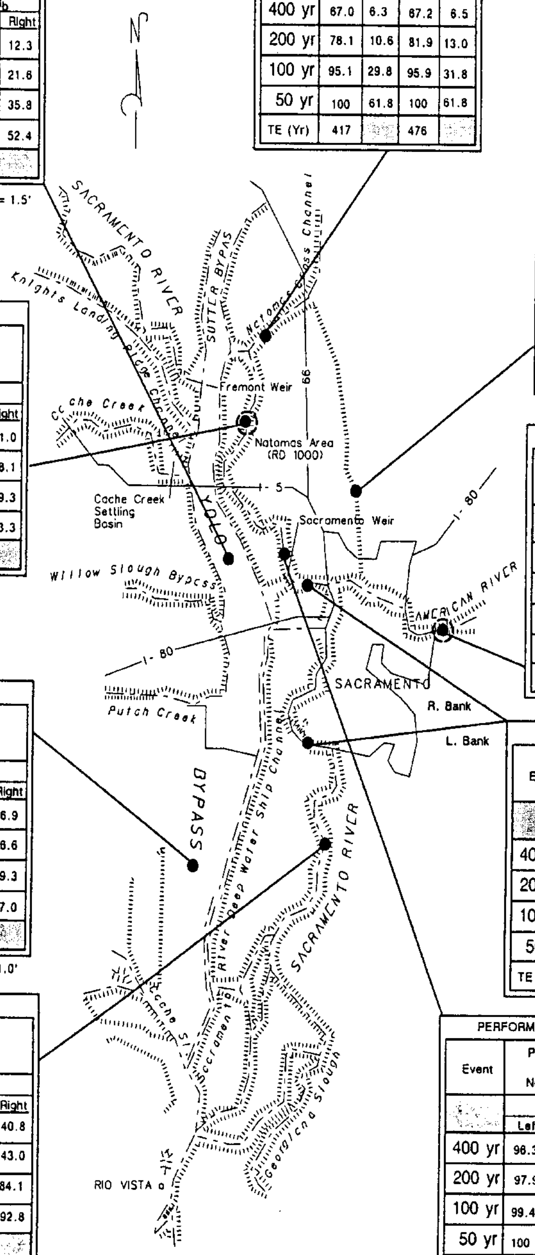
PERFORM. INDEX POINT 3

Event	Percent Chance of Non-Exceedance			
	R_x Left	R_x Right	R_b Left	R_b Right
400 yr	43.7	23.2	37.2	16.9
200 yr	48.1	28.8	47.8	28.6
100 yr	59.9	38.9	59.5	39.3
50 yr	86.2	70.4	75.7	57.0

PNP Left = +1.0' PNP Right = 1.0'

PERFORM. INDEX POINT 4

Event	Percent Chance of Non-Exceedance			
	R_x Left	R_x Right	R_b Left	R_b Right
400 yr	61.8	61.8	40.8	40.8
200 yr	73.2	73.2	43.0	43.0
100 yr	84.3	84.3	84.1	84.1
50 yr	94.1	94.1	92.8	92.8
TE (Yr)	238		154	



PERFORM. INDEX POINT 5

Event	Percent Chance of Non-Exceedance			
	R_x Left	R_x Right	R_b Left	R_b Right
400 yr	67.0	6.3	67.2	6.5
200 yr	78.1	10.6	81.9	13.0
100 yr	95.1	29.8	95.9	31.8
50 yr	100	61.8	100	61.8
TE (Yr)	417		476	

PERFORM. INDEX POINT 6

Event	Percent Chance of Non-Exceedance			
	R_x Left	R_x Right	R_b Left	R_b Right
400 yr	97.7	97.7	96.0	96.0
200 yr	100	100	98.4	98.4
100 yr	100	100	100	100
50 yr	100	100	100	100
TE (Yr)	>500	>500	>500	>500

NED and PERFORM. INDEX POINT 7

Event	Percent Chance of Non-Exceedance			
	R_x Left	R_x Right	R_b Left	R_b Right
400 yr	26.4	26.4	2.2	2.2
200 yr	64.7	64.7	17.3	17.3
100 yr	93.2	93.2	59.3	59.3
50 yr	99.6	99.6	93.3	93.3
TE (Yr)	217	217	100	100

PERFORM. INDEX POINT 8

Event	Percent Chance of Non-Exceedance			
	R_x Left	R_x Right	R_b Left	R_b Right
400 yr	93.4	73.9	75.2	43.4
200 yr	97.7	78.2	77.6	47.1
100 yr	99.4	90.8	99.2	90.8
50 yr	100	98.1	100	98.1
TE (Yr)	>500		>500	

PERFORM. INDEX POINT 9

Event	Percent Chance of Non-Exceedance			
	R_x Left	R_x Right	R_b Left	R_b Right
400 yr	98.3	75.3	84.7	50.1
200 yr	97.9	81.9	87.6	54.3
100 yr	99.4	90.4	99.5	90.2
50 yr	100	99.1	100	99.8
TE (Yr)	>500		>500	

Figure 3 - Risk Based Analysis Performance Matrices

the percent chance of non-exceedance expressed as True Exceedance (TE) for the base and alternative conditions. For alternatives that increase downstream flows, incremental decreases in the Reliability and True Exceedance were used to determine hydraulic mitigation requirements for that alternative. Geotechnical characteristics of the existing levees and proposed levee enhancements were evaluated using a combination of historic system performance, field testing and stability analysis. The economic evaluations were performed at two index points, one for the Natomas basin and one for the greater Sacramento area. The economic analysis included the generation of the stage-damage function by using Structural Inventory for Damage Analysis (SID) computer program.

HYDROLOGIC ANALYSES

General. The HEC Flood Hydrograph Package HEC-1 model and, Interagency Advisory Committee on Water Data, "Guidance for Determining Flood Flow Frequency, Bulletin 17B," USGS were used to develop discharge-frequency relationships. The American River system is regulated via several small reservoirs in the upstream watershed and by Folsom Dam and Reservoir on the mainstem. Reservoir routing to determine the inflow/outflow characteristics of the storage components were performed using volume balance spreadsheet incorporating spillway and outlet ratings and operational criteria. The HEC-2 model was used to develop stage-discharge data, and the UNET (One-Dimensional Unsteady Flow Through a Full Network of Open Channels) model was used to develop stage-frequency data and to route outflow hydrographs throughout the system. Figure 4 presents a schematic of the hydrologic analysis, including the error distributions for each function.

Discharge-Frequency Functions. In 1961, a statistical analysis was performed to establish the frequency of occurrence for various flows in the American River at the Fair Oaks gage downstream from Folsom Dam. However, because the 1986 flood and five of the ten largest flows in the basin for the 82 years of record have occurred since 1961, and seven of the 10 largest events have occurred since 1951, a new flow-frequency analysis was conducted. A subsequent analysis was performed to include the last eight years of record. The re-analysis included establishing the adjusted unregulated flow removing the routing effects of the upstream storage including Folsom Reservoir. The guidelines set out in Bulletin 17B were used to cast the data into a log Pearson Type III discharge-frequency distribution.

Discharge-Frequency Uncertainty. Appendix 9 of Bulletin 17B, was used to quantify uncertainty in the unregulated Folsom Reservoir inflow discharge-frequency function. The period of record used to develop the confidence limits and subsequent uncertainty about the discharge-frequency function was 90 years.

Reservoir Routing. The lower American River is a highly regulated system with Folsom and Nimbus Dams located immediately upstream. Flows and stages in the river are controlled by releases from Folsom Dam. Therefore, in analyzing flood control alternatives for the lower American River, reservoir routing is required. Consequently, EAD computation via simulation must sample Folsom outflow, use stage-discharge functions that represent system performance with the levees in place, and employ error functions representative of these conditions. An inflow-outflow function was developed by repeatedly computing, with a spreadsheet, outflow peaks for given

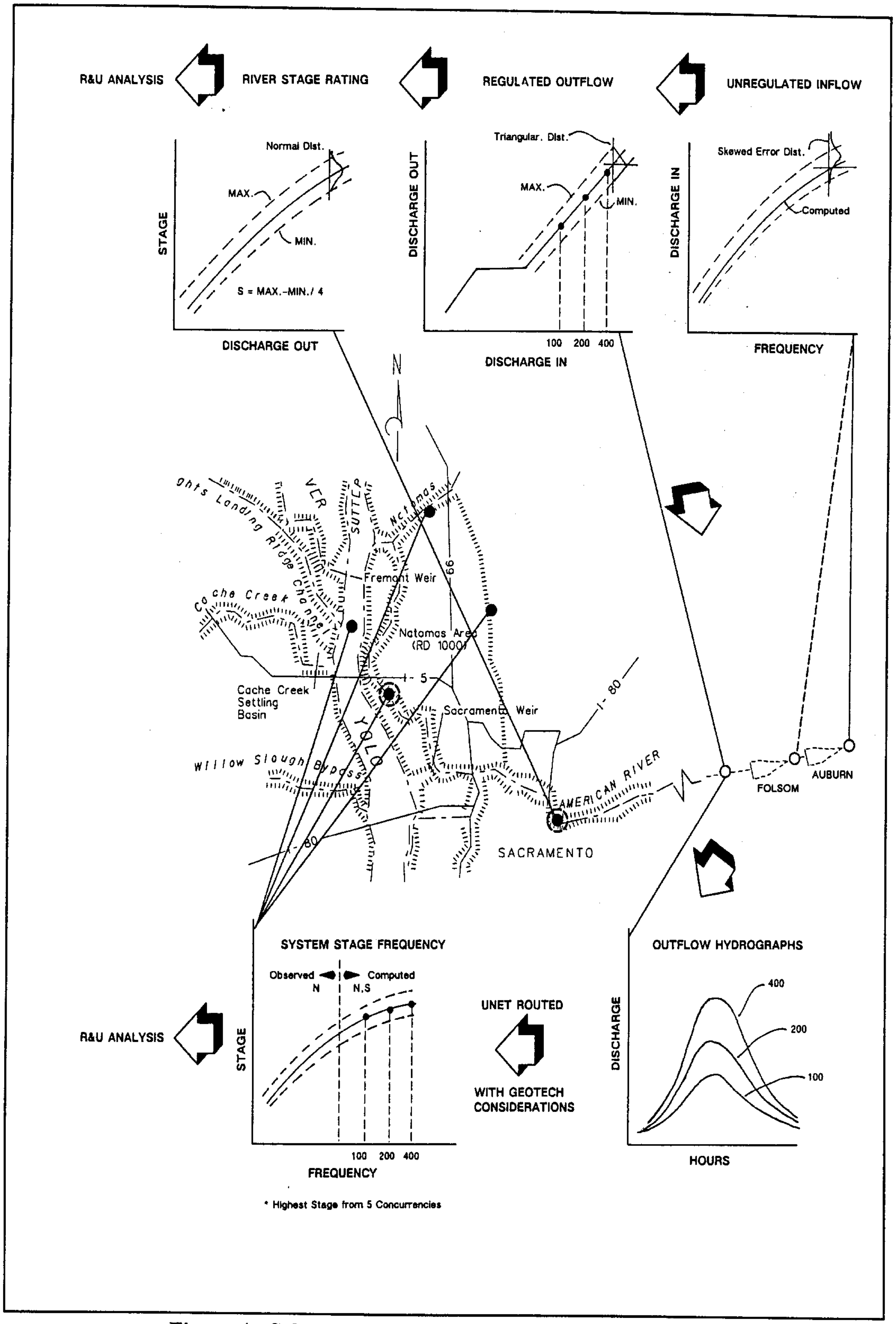


Figure 4 - Schematic of Hydrologic Analysis Components

inflow peaks. Reservoir routing to determine the inflow-outflow characteristics of the storage components was performed using a volume balance spreadsheet incorporating spillway and outlet ratings and operational criteria. Table 2 summarizes the operational criteria used to perform the base routing and to develop the base and alternative inflow-outflow functions. Figure 5 presents an example of the structure of such a function and is presented in the figure as the "most likely case."

Reservoir Routing Uncertainty. There are uncertainties in the operation of Folsom and the performance of the new detention facility. Itemizing all possible parameters that may impact the outflow was the first step in establishing an uncertainty distribution about the inflow-outflow function. The list included items such as variances in the spillway gates operations, hydropower penstocks operations, river outlet cavitation requiring modification to releases, insufficient personnel to make matching changes at Nimbus and Folsom Dams, political pressure to change operations, inaccurate inflow data, flood event waves varying from the expected, the amount of the available space in Folsom Reservoir, etc. For most of these items it was difficult to quantify the uncertainty they would have on the operation. However, it was determined that sets of conditions could be identified to describe the most likely "best" and most likely "worst" set of conditions that could

Table 2
Current Operation Criteria Used for Base Reservoir Routing

CURRENT OPERATION ASSUMPTIONS
Multiple-Waves (Two 4-day Waves)
No Initial Encroachment
Initial Release of 8,000 cfs (Maximum Power)
Initial Flood Release Delay of 10 Hours (Applied to Second Flood Wave if Flood Reservation was Evacuated)
4 Hour Response Time Matching Outflow to Inflow
Rate of Change of Release Increase - 5,000 cfs/hr to 25,000 cfs - 15,000 cfs/hr above 25,000 cfs Decrease - 5,000 cfs/hr
Folsom Dam Release - Existing Full Capacity of Main Spillway in Combination with River Outlets (60% Gate Opening) Power Release of 8,000 cfs (Full Capacity)
Surcharge as Prescribed by Emergency Spillway Release Diagram
Routing with Credit for Upstream Space - Reduce Folsom Unregulated Inflow by 12%

occur to set the bounds on the operation. These combinations and subsequent sensitivity routings were cast into a triangular distribution of error about the inflow-outflow function. These triangular distributions were typically asymmetric. Sensitivity runs were made to determine how many factors should be combined to capture approximately 95 percent of the uncertainty. Table 3 shows the operational criteria used to set the operational bounds on the inflow-outflow curve for an example alternative that include a detention dam at Auburn and reoperation of Folsom Reservoir. For each alternative analyzed, inflow-outflow curves with uncertainty bounds were developed to serve as input to the risk-based analysis. This outflow error distribution is incorporated in the EAD computations. The sampled discharge with error is treated as Folsom inflow. The outflow with error is found by sampling the triangular distribution defined about the most-likely outflow. It should be noted that for actual alternative functions, the uncertainty limits about the flat portion of the curve was zero when the outflow is the objective release. It was determined that there was no uncertainty associated with the actual objective release itself.

System Routing. The UNET program was used to perform the system routings for this study. This model was developed by Dr. Robert Barkau and is currently supported by the HEC. UNET simulates one-dimensional unsteady flow through a full network of open channels. To facilitate model application, cross section data are input in a modified HEC-2 forewater format.

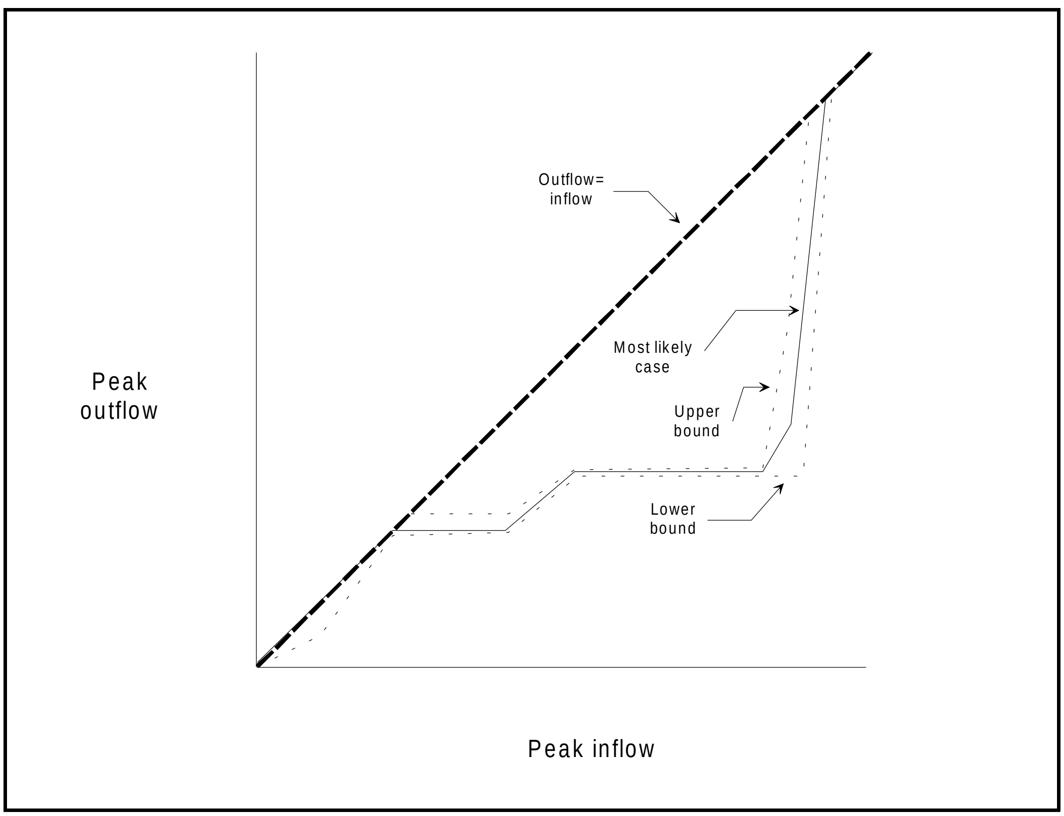


Figure 5 - Example Inflow-Outflow Function with "Confidence Bands"

Boundary conditions for UNET can consisted of stage and flow hydrographs which were read in from any existing HECDSS (Hydrologic Engineering Center Data Storage System) data base. The Sacramento River, American River and Yolo Bypass models were combined into one large model. The ability to use one model affords the advantage of making a change at one location and see how that change affects the rest of the system. The UNET model contains 29 river reaches. The 1986 event was used to calibrate the model since it was the largest flood of record at many locations, a large amount of field observations exist, and a large network of stream gaging stations were in place during the flood to measure flows and stages at many locations.

Table 3
Folsom Dam Operation Uncertainty Evaluation
Alternatives Including an Existing Reoperated Folsom Dam and Detention Dam

	Operations Conditions		
Factor	**Base**	**Maximum**	**Minimum**
Extra Space in Folsom Lake	0	0	100,000 Ac-Ft
Flood Waves	2	2	1
Upstream Reservoir Space	0	0	150,000 Ac-Ft
Outlet Works Operation	60%	0%	60%
Spillway Efficiency	Free Flow	3 Ft Surcharge	Free Flow
Initial Delay in Releases	10 hours	24 hours	0

Stage-Discharge Functions. The HEC-2 computer program, "Water Surface Profiles" (Version 4.6 - May 1991) was used to compute water surface profiles for the existing Lower American River system and for potential levee upgrades along the Lower American River. An HEC-2 model of the American River from Nimbus Dam to its confluence with the Sacramento River was developed for and utilized in a Flood Insurance Study (FIS) of Sacramento City and County in 1989. This model utilized cross sections surveyed by California Department of Water Resources (DWR) downstream of River Mile 14 and by CESPK upstream thereof. Cross sections of bridges were either taken from construction drawings or field surveyed. The model was calibrated using high water marks recorded during the February 1986 flood event. The starting water surface elevations for the study were developed via the UNET model of the American and Sacramento Rivers confluence area assuming no upstream levee failures on either the American or the Sacramento River systems. Several types of energy loss coefficients are used in the HEC-2 model for computation of head losses between cross-sections. These losses include boundary roughness associated with type and amount of vegetation, channel configuration, and channel meander, changes in the shape of the river channel, such as contractions and expansions of the floodway, and bridge losses associated with pier number and type and flow type (pressure, etc) and entrance and exit conditions. The calibrated model has a SWSEL of 31.3 feet and discharge at the Sacramento River of 144,500 cfs and upstream of the NEMDC the discharge is 130,000 cfs. This represents the peak flow of the 1986 flood event.

Stage-Discharge Uncertainty. Uncertainty in stage was determined through a sensitivity analysis for defining the upper and lower limits for the one-percent chance event. The sensitivity analysis included adjusting the Manning's roughness coefficients, sediment accumulation, bridge blockage, and error in cross-section definition, starting water surface elevation, cross section geometry, debris on bridges, and scour. This hydraulic sensitivity analysis includes computation of the standard deviation of the stage differential, for the maximum and minimum conditions assuming higher objective releases from Folsom Dam. The base condition water surface profile, for each higher objective release from Folsom Dam, was compared to the corresponding water surface profiles resulting from the combination of hydraulic parameters to be tested which produce either the maximum or minimum conditions. This comparison identifies the sensitivity of the stage to the variance of the combination of hydraulic parameters. The cumulative maximum condition for all hydraulic parameters included modeling for debris on bridges, increased roughness ("n" values increased 15%), sediment deposition, 10% variance in geometry horizontally (except at bridges), and increased starting water surface elevation. This cumulative maximum condition resulted in a profile that was 1-foot higher at the downstream end of the American River and approximately 2-feet higher than the base condition at the upstream end of the study limits. The cumulative minimum condition for hydraulic parameters tested roughness (n values reduced 15%), 10% variance of each cross section horizontally, and starting water surface elevation for sensitivity to stage. Given the range of the computed water surface stages from 2 feet higher to 1.5 feet lower than the base, a standard deviation for the uncertainty in stage was selected at 1.0 feet.

Stage-Frequency Functions. Given the complexity of the Sacramento and American Rivers system, it is not possible to define a unique set of discharge-frequency and stage-discharge relationships at all index points. It was therefore necessary to develop stage-frequency functions at each index point where this condition exists. The stage-frequency curves were developed by combining observed data (at "base" locations where stage data exists) with 100-, 200- and 400-year stages computed by UNET at the actual index point. Base stage-frequency curves were developed for the Sacramento River at Verona, I-Street and Snodgrass Slough and for the Yolo Bypass at Woodland and Lisbon. Therefore, the stage-frequency function is made up of observed data in the high frequency range and computed stages in the low frequency range (See Figure 4).

1) Observed Data. The observed data for each base curve were plotted using Weibull plotting positions. The stage-frequency curve was drawn graphically through the points. The observed data at the base curve locations were translated to the index point locations based on the difference in the water surface profile of the 1986 flood between base curve location and other index point locations.

2) Computed Data. The development of the 100-, 200-and 400-year flood contribution from the Sacramento River required an understanding of what causes high stages at the Sacramento-Feather River Confluence (SFRC). A review of several large floods revealed that a large number of flow combinations from the Sutter Bypass, Sacramento River and Feather River can occur. Therefore,

a volume-frequency relationship was developed at the SFRC, which reflects the many concurrent flows that have occurred historically. The 100-, 200- and 400-year floods were calculated using this relationship. The eleven largest floods from 1929-1988 (59 years) were chosen to determine the volume-frequency curves. Historic hydrographs were developed to reflect the routing effects of the upstream flood control reservoirs. These hydrographs were routed using HEC-1, to the Sacramento-Feather River confluence to obtain peak and volume flows at this point. The 100-, 200- and 400-year points on the stage-frequency curves were computed using the UNET model. The stage resulting from the 100-year American and 100-year Sacramento was used as the 100-year stage for all index points for all alternatives. For the 200- and 400-year events, the combination that resulted in the highest stage for the given event was used.

Uncertainty in the Stage-Frequency Function. The observed and computed data required to construct the stage-frequency functions are used as input to the LIMIT program. LIMIT combines and calculates the uncertainty about the non-analytically derived frequency curve. LIMIT can be used when a frequency curve is developed based on systematic observations, hypothetical events, or both. Input to the LIMIT program consists of a stage-frequency curve, equivalent years of record for the systematic record and hypothetical events, and the estimated model error for the computed portion of the function.

1) Systematic Record. The equivalent years of record for each index point is based on the actual period of record of the base curve for that point. The equivalent years of record were computed using Interim Guidelines presented in Engineering Circular (Planning Guidance A-11) at HEC workshop on Risk and Uncertainty in February 1993. The guidance presents several ways of estimating the equivalent years of record. The most appropriate for base gage locations was using the adjustment corresponding to a long period gage within the watershed with the model calibrated to a gage based location. This adjustment calls for the equivalent years of record to be 50 to 90 percent of the actual record. The equivalent years of record ranged from 35-44 years depending on location.

2) Hypothetical Events. The stages for the 100-, 200-, and 400-year hypothetical events were computed, by UNET, from hydrographs derived from the volume-frequency analysis at the Sacramento-Feather River Confluence (SFRC) described earlier. The volume-frequency analysis at the SFRC is based on 59 years of record. The adjustment corresponding to a long period gage within the watershed with the model calibrated to a gage based location was used to determine the equivalent years of record. Since the volume-frequency analysis involved routings, which may further decrease the reliability of the values, 70 percent of the record, or 41 years, was used for the equivalent years of record.

3) Estimated Model Error. In order to determine the estimated model error in the hypothetical portion of the stage-frequency curve, a sensitivity UNET analysis was performed. Invert elevations were increased and decreased by 2.5 feet which is half of the contour interval. N-values were increased and decreased by 25 percent. These adjustments were run separately and in combination with each other. Analysis of the results determined that the most reasonable error range is + or - 1.0 foot, or 2 foot total error range. In other words, the error band about the hypothetical elevations is one foot on either side. The model error is input to the LIMIT program as 25 percent of the total error. Using a 2-foot total error, the model error was set at 0.5 feet.

GEOTECHNICAL ANALYSIS

General. One of the elements considered in the risk and uncertainty analysis is the uncertainty in levee failure. According to the Engineering Technical Letter 1110-2-328, "Reliability Assessment of Existing Levees for Benefit Determination", the PNP is defined as the stage elevation below which it is highly likely that the levee would not fail. The PFP is defined as that stage above which it is highly likely that the levee would fail. The target stage for reliability computation, as described, typically represents a stage at which significant damage initially is incurred. For an existing levee, that stage is greater than the PNP stage and is less than the PFP stage, but is not known with certainty. The design parameters used in evaluating the stability of the existing levee system were determined by geotechnical evaluation and investigation.

American River Levees. There are approximately 22 miles of levee protecting the land north and south of the American River. For the Lower American River project reach, the PNP/PFP values were determined primarily by using the slope stability criteria developed in a 1988 report and levee performance during the 1986 flood. An evaluation of landside slope instability were conducted at each cross section. It was determined that for a levee section to be considered stable, three criteria should be met. These criteria include: 1) a minimum of 3 feet of freeboard; 2) an estimated steady seepage water exit height above the landside levee toe of no more that 0.6 foot; and 3) a hydraulic head difference between flood stage and the adjacent landside levee toe of no more than 6.0 feet. With consideration of 1986 levee performance, the PNP values along the lower American River are determined at least equal to the 1986 flood level, which is equivalent to a flow of approximately 130,000 cfs.

Sacramento River Levees. The Sacramento River project areas include the left and right bank levees starting from Rio Vista to Verona. Including both banks, this reach includes a total of 134 miles of project levees. The selection of the PNP and PFP values are based on the evaluation of existing levee profiles, the design water surface profile, and the estimated 1986 flood profile. For much of the levee reach, the selected PNP/PFP values were based upon past performances, i.e. the system passing previous floods near the design flood level. Levee modifications were performed in several miles of left bank levee above Freeport. As a result of the repairs made, the PNP and PFP values in these areas are higher than the rest of the study area. The higher PNP and PFP values were selected based on analytical approximations and judgement of the effects of a higher water surface on the modified levee cross sections. In project locations where landside berms were constructed, the height of the inclined drain is typically one-third to one-half of the height of the levee. The estimated increase in the seepage line through the levee resulting from a 2-foot higher river stage is minimal and has little impact on slope stability. Also, because the foundation soils of levees are predominantly fine grained and clay, the overall threat of foundation piping, in areas of berm construction, is minimal. Along the levee reaches where slurry cutoff walls were constructed, PNP profiles vary from 1 to 5 feet from the top of cutoff wall because of the varying levee height.

Yolo Bypass Levees. The determination of PNP and PFP values along the Yolo Bypass was handled differently from the reaches of American River and the Sacramento River. The Yolo Bypass is not considered as part of the flood control system that will receive increased flood protection from this project. The area was only analyzed from a hydraulic mitigation standpoint. The intent of the hydraulic mitigation work is to remedy the impacts due to the increased objective release from American River so that the additional flow will not worsen reliability of the existing flood control system. The important calculation is the relative change in the system reliability in the hydraulic mitigation areas to determine if there is an impact. Therefore, along the Yolo Bypass, the PNP level was set at stages equivalent to the existing design water elevation. This is based on the assumption that even though the existing levee system has some known geotechnical problems, measures such as floodfighting would be executed to pass flows up to the design water surface.

ECONOMIC ANALYSIS

The economic analysis is based on October 1995 price levels, 7 5/8 percent interest rate, 100 year project life and future growth conditions from 1995 to 2008. Damage categories included residential, commercial, industrial, public, agricultural, emergency costs, and auto. HEC computed the damage-frequency relationships using the Structural Inventory for Damage Analysis (SID) computer program. The structure inventory was entered into the SID program to develop elevation-damage functions by category. The overall study had two damage assessment index points, one for the Natomas area, Index Point 2 and one for the greater Sacramento area, Index Point 7. The Sacramento Index Point was composed of five subreaches - Northern Sacramento, Rancho Cordova, South Sacramento, and Richards Boulevard. There were no uncertainty estimates used for this analysis for the economic parameters.

EVALUATION PROCESS

After configuring the initial seventeen potential measures into the eight initial alternatives, an initial risk-based was performed for two basic reasons. First, for alternatives that included increased releases from Folsom Reservoir, hydraulic impacts to the downstream system needed to be evaluated. As compared to the base, or existing system performance matrix, if there was a decrease in the levee reliability or true exceedance at any given index point, hydraulic mitigation features were included into that alternative and the cost adjusted. Additionally, once a full alternative was structured and the associated costs, the EAD and net benefits were determined to narrow the array of alternatives. The final array of alternatives reduced to four as shown in Table 1. A final incremental analysis was performed on these final plans cycling through the measures that made up that alternative to ascertain which features were the most cost effective or incrementally justified.

RESULTS

The analysis revealed a clear and substantial difference in the flood reduction benefits afforded by increased upstream storage as opposed to increased Folsom outflow releases and modification to the downstream system. The annual net benefits for the detention dam and the increased downstream system is $109 million and $77 million, respectively. The probability of failure in any one year is less than 1 in 500 for the detention dam and 1 in 235 for the increased downstream system plan. Table 4 presents a summary of the evaluation for the No-Action and the three finalist alternatives.

EPILOG

Political fallout, total project costs and environmental issues suspended the decision on selection of an alternative. Features common to all alternatives have proceeded to the design phase and those features will be constructed in FY98.

Table 4 - Summary of Risk Based Analysis

	Alternative			
	No-Action Plan	Folsom Modification Plan	Folsom Stepped Release Plan	Detention Dam Plan
Probability of Flooding in any one year	1 in 100	1 in 180	1 in 235	< 1 in 500
Probability of Passing a 200-yr Flood Event (%)	16	54	68	97
Benefit Summary				
First Cost ($ Million)	-	470	627	949
Annual Cost	-	49	72	95
Annual Benefit	-	126	130	204
Annual Net Benefit	-	77	58	109