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ENGINEERING REPORT

Folsom Dam Water Control Manual Update

U.S. Army Corps of Engineers
Sacramento District

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US Army Corps
of Engineers ®

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Appendices

1 Summary and Conclusion

1.1 Study Purpose

The purpose of the Folsom Dam Water Control Manual Update (Manual Update) is to reduce flood risk to the Sacramento area by fully utilizing the additional release capacity provided by the new Joint Federal Project (JFP) auxiliary spillway, while restricting the maximum required flood reservation (space) to 400 to 600 thousand acre-feet (KAF). This report documents the engineering analyses leading to identification of the selected operation as presented in the updated Water Control Manual (WCM) in Appendix H.

The Manual Update serves the purposes of Flood Risk Management (FRM) and dam safety. Maximizing FRM performance is the goal of the Corps and their non-Federal partners, the Central Valley Flood Protection Board (CVFPB) and the Sacramento Area Flood Control Agency (SAFCA). Ensuring Folsom dam safety is the role of Reclamation. However, all agencies and partners recognized that both purposes are best served by close coordination between agencies and partners. The following performance goals were identified by the study team. The first and second goals were considered performance requirements. It was not known at the beginning of the study whether goals 2 and 3 could be met.

1. Control a 1/100 annual chance exceedence (ACE) event to a maximum release of 115 thousand cubic feet per second (kcfs) as defined by criteria set by SAFCA to support Federal Emergency Management Agency (FEMA) levee accreditation along the American and Sacramento rivers.

2. Control a 1/200 ACE event to a maximum release of 160 kcfs, as defined by the State of California Department of Water Resources (DWR) locally preferred criteria.

3. Pass the probable maximum flood (PMF) event while maintaining 3 feet of freeboard below the top of dam to satisfy dam safety requirements of Reclamation.

Candidate operations (alternatives) were defined, which consisted of the flood operation rules that are specified in two key diagrams in the updated WCM: the Water Control Diagram (WCD) and the Emergency Spillway Release Diagram (ESRD). Two final alternative operations were developed. Both alternatives used the same ESRD, and both alternatives used the same seasonally varying guide curve (a key feature of the WCD). The alternatives differ in how variable TOC is computed during the winter. Alternative 1 uses current upstream storage credit and basin wetness. Alternative 2 uses forecasted inflow volume up to 120 hours (5 days) in the future. The two alternatives were configured in reservoir simulation models, and a suite of flood events, including PMF events, were simulated. FRM and dam safety metrics were evaluated, and based on these, one alternative was selected. The selected alternative was then further analyzed to consider effects on the downstream channel and levees, and effects on other (non-flood) project purposes.

1.2 Selected Flood Operation

Based on engineering analyses conducted and documented in this report, Alternative 2, the forecast-based flood operation, has been selected for inclusion in the updated WCM. The operation provides better FRM performance than Alternative 1, the (upstream storage and basin wetness) credit-based operation. Alternative 2 also explicitly promotes increased storage availability (within 400 to 600 KAF flood space) during winter months and allows refilling of the reservoir to the top of variable flood space when the event has passed. This characteristic improves the likelihood of spring refill operations starting at a higher storage level. Reclamation's dam safety requirement of passing PMF events with 3 feet of freeboard to top of dam was satisfied equally with both alternatives.

In addition to FRM and dam safety performance, analyses were conducted to compare effects of the selected alternative to existing condition operations. Areas of these analyses included:

1. Erosion to downstream banks, bridges, and levees
2. Effects to other (non-flood) project purposes
    - Water supply
    - Hydropower
    - Water quality
    - Fish and wildlife
    - Recreation
    - Navigation

1.3 Summary of Analyses and Findings

This section provides a brief description of each analysis, conclusions relevant to the selected operation, and references to pertinent sections in the report for further reading.

1.3.1 Hydrologic Analyses

All engineering and effects analyses were dependent on hydrologic datasets developed and described in Chapter 7. Winter and seasonal synthetic events, consisting of scaled versions of historical events, were developed for ACE ranging from 1/2 to 1/1000. These were used to assess FRM performance. Winter and seasonal PMF events were developed to assess the dam safety requirement. Eighty-one years of period of record (POR) data, spanning WY 1922-2002, were developed to support a range of environmental and effects analyses.

1.3.2 Development of Alternative Operations

Section 1.1 identified the three performance goals for the selected operation. Chapters 3, 4, and 5 describe existing operations and the development of alternative operations. The two final alternatives were:

1. Alternative 1 – Credit-based operation, and
2. Alternative 2 – Forecast-based operation

The credit-based operation leverages information on space available at upstream reservoirs and wetness of the watershed to compute how much flood space, between 400 and 600 KAF, in the reservoir must be reserved (kept empty) at Folsom Lake for managing winter flood events. The forecast-based operation leverages forecasts of reservoir inflow, provided by the California-Nevada River Forecast Center (CNRFC), to compute how much flood space, between 400 and 600 KAF, must be reserved for managing winter flood events. These forecasts also account for the effect of empty space in upstream reservoirs.

1.3.3 FRM Performance Based on Reservoir Peak Release

This analysis consisted of configuring reservoir simulation models to reflect the candidate operations. A suite of winter and seasonal synthetic storm events were simulated. FRM performance was evaluated by considering the largest events could be successfully routed so as to hold the peak releases to 115 and 160 kcfs. Results of the suite of simulations are provided in Table 6-11 through Table 6-16. A summary of best-estimate peak releases for probabilities of interest are provided in Table 6-23. A summary of the largest events passing at the target releases is provided in Table 6-24. These results show that Alternative 1, the credit-based operation, can hold an ACE = 1/189 event to 160 kcfs (channel capacity). Alternative 2, the forecast-based operation, can hold an ACE = 1/237 event to 160 kcfs. Figure 6-14 indicates that both alternatives result in increased peak releases, as compared to existing condition operations, for events more frequent than ACE=1/15. This is attributed to increased release capacity provided by the JFP spillway and uncertainty in accurately modeling small floods. During small floods, operators frequently do not make full flood releases because they are not needed. The models do not reflect this reality.

1.3.4 Uncertainty in Forecast Information

Uncertainty in forecast information was considered in the robustness tests documented in Section 6.5.2. These tests showed that inflow forecast volumes having 75 percent Non-Exceedence Probability (NEP) should be used for operations.

1.3.5 Dam Safety Based on PMF Freeboard to Top of Dam

Alternatives 1 and 2 are equivalent with respect to capability to pass PMF events with 3 feet freeboard to the top of dam. This capability is governed by configuration of the ESRD, which is the same for both alternatives. ESRD development is documented, and PMF event routings provided, in Appendix F.

1.3.6 Downstream Flood Risk and Other Effects

Effects of the selected plan on downstream flood risk, other authorized Folsom Dam project purposes, and environmental resources have been analyzed, evaluated and considered in this study. Downstream flood risk is reduced by the selected Water Control Plan because it reduces the chances of system capacity exceedence and levee overtopping. It also reduces the frequency of flows most likely to cause a levee erosion failure, while also increasing the occurrence of flows that are unlikely to cause levee failure due to erosion. The National Environmental Policy Act (NEPA) and other legally required effects analyses and evaluations are documented in a companion draft *Supplemental Environmental Assessment/Environmental Impact Report* (SEA/EIR). Extensive stakeholder and public involvement was included in the development and evaluation of the selected Water Control Plan, and also documented in the SEA/EIR.

2 Background

This chapter provides information on the geographic location, history, and authorizations pertinent to the Manual Update. Requirements for revision of the WCM are also provided.

2.1 Local Area

The local area of analysis (local project area) for the Manual Update reflects both the area for which FRM is being provided and the Folsom Dam features designed to provide FRM (Figure 2-1)

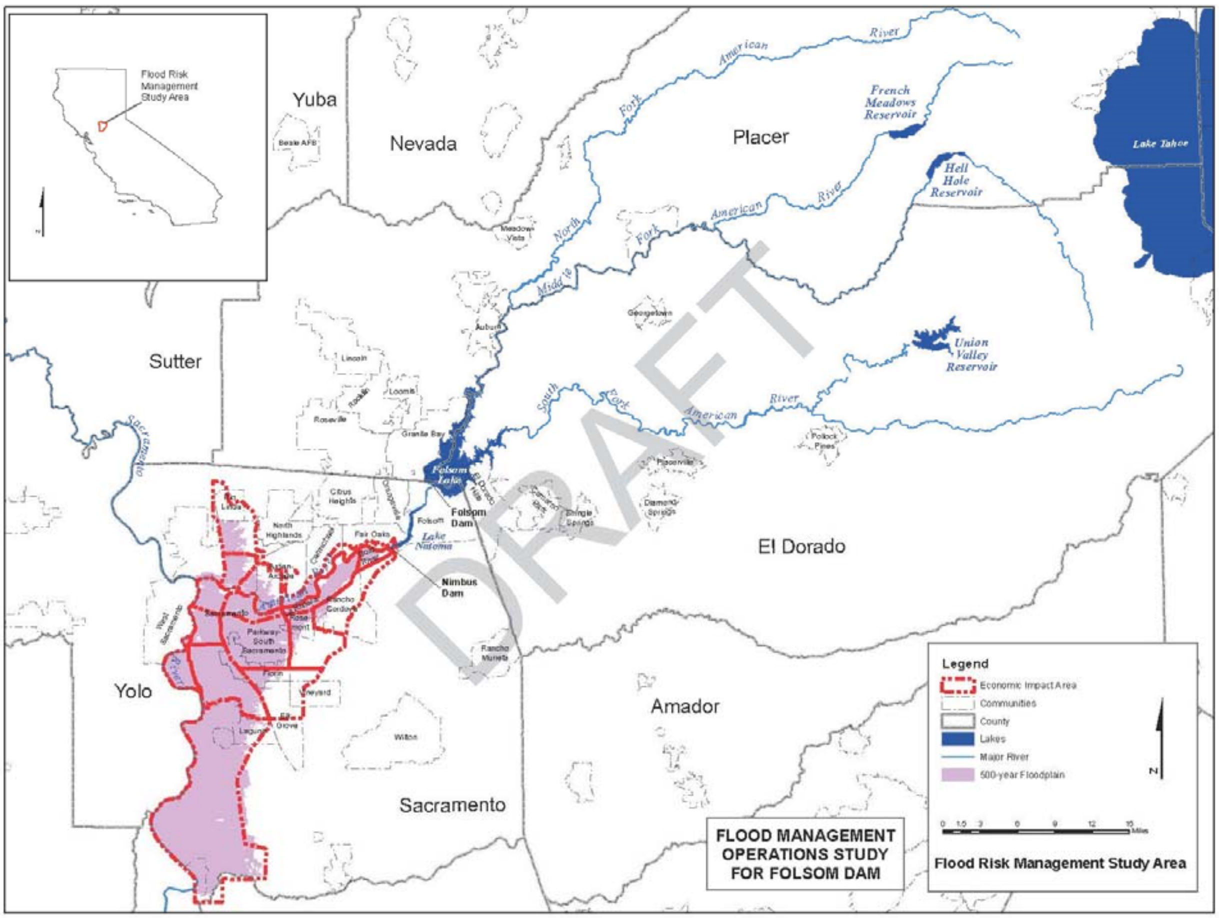

Figure 2-1: Local Project Area

The regional area of analysis (regional project area) was used for assessment of environmental effects. This area included the Central Valley Project (CVP) and State Water Project (SWP) facilities and service areas (Figure 2-2). Water released from Folsom Lake has many uses including generating hydroelectric power, meeting all water rights obligations, and maintaining environmental quality.

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Figure 2-2: Regional Area of Analysis

2.2 History

Folsom Dam is located along the American River, approximately 26 miles upstream from the confluence with the Sacramento River. Folsom Lake is the largest reservoir in the American River watershed with a gross pool capacity of 967 KAF, corresponding to a lake elevation of 468.34 feet NAVD88 (466.00 feet NGVD29). In conjunction with levees on the Lower American River (LAR) and Sacramento River and other system improvements, Folsom Dam and Lake provides FRM for the greater Sacramento area. Construction of the Folsom Dam and Lake project was completed by the Corps in 1956. The project was transferred to Reclamation for operation and maintenance as part of the Central Valley Project (CVP). Reclamation operates Folsom Dam for FRM with criteria established by the Corps along with other authorized purposes such as hydropower, recreation, and water supply.

A flood of record on the American River in 1986 seriously taxed both the control of Folsom Dam and the downstream flood control (FC) system and showed that there was a much greater flood risk to the Sacramento area than previously estimated based on observed inflows. As a consequence of that flood, the Corps conducted several studies under the authority of the Flood Control Act of 1962 (Pub. L. 87-874), which authorized study of the American River Basin for FC and allied purposes. A variety of alternatives to reduce flood and dam failure risk were investigated and received consideration. These included a detention dam upstream on the North Fork of the American River (Auburn Dam), raising Folsom Dam, increasing the capacity of existing Folsom Dam outlets, and improvements to downstream conveyance facilities.

Efforts were initiated to authorize and implement a FRM plan that would provide a higher level of protection. *The Folsom Dam and Lake Reoperation, California, Operations Plan and EIS,* March 1992, evaluated increasing the space allocated for FC on a temporary 10-year basis (1992-2002) until authorization and completion of a FC project to provide a high level of protection (1/200 ACE) to the Sacramento area. The report found that increasing the FC space from 400 to 590 KAF, on a temporary basis, would be economically feasible, but stated:

“New congressional authorization would be needed by the Reclamation and local FC beneficiaries. A cost-shared plan between the Department of the Interior and the non-Federal FC beneficiaries would be required for implementing the new reoperation criteria.”

The report was submitted to the Assistant Secretary of the Army for Civil Works (ASA [CW]) for approval on 22 December 1992.

Subsequent discussions were held between the Corps and Reclamation. By letter dated 6 October 1993, Reclamation informed the Corps that operating Folsom Lake to provide additional flood storage was within Reclamation’s operational flexibility. Reclamation then assumed the role of lead agency for the operation of the reservoir to provide increased flood protection. Reclamation and SAFCA then proposed a variable FC space regime that relied not on a fixed amount of FC space, but rather the provision of FC space varying between 400 and 670 KAF, depending on available storage in upstream non-Federal reservoirs. When the upstream reservoirs are full (no available storage space), then 670 KAF of storage would be required in Folsom Lake to provide the desired level of flood protection. SAFCA and Reclamation entered into an agreement for the reoperation of Folsom Dam and Lake. The Flood Control Operations Agreement, as modified, required SAFCA and Reclamation to compensate affected water service and hydroelectric power contractors for the costs of implementing the creditable storage regime as compared to the fixed storage regime.

Work on the existing Folsom Dam outlets was authorized for construction in the Water Resources Development Act of 1999 (WRDA 99). In a separate effort, downstream conveyance and levee improvement features common to all project alternatives were authorized for construction (American River Common Features project (ARCF)). Auburn Dam alternatives were never authorized as a result of these studies. Corps efforts to construct the authorized modifications to the existing Folsom Dam lower outlet gates were terminated in the procurement phase when it became evident that the technical, construction, and cost risks associated with the modification project were significantly greater than previously understood.

The Energy and Water Development Appropriations Act of 2006 (EWDAA 2006) then directed further joint study by the Secretary of the Army (through the Corps) and the Secretary of the Interior (through Reclamation) to maximize flood damage reduction (FDR) improvements and address dam safety needs. These successor studies formulated an auxiliary spillway alternative that addressed both overtopping risk reduction (passing the PMF) to Folsom Dam and reduced downstream flood risk. Study results were refined and formalized in the *Post Authorization Change Report* (PACR) *for the American River Watershed Project* dated March 2007. This report included recommendations for the JFP auxiliary spillway and a 3.5-foot raise of the dam and reservoir dikes.

By memorandum dated 25 April 2007, the Director of Civil Works submitted the PACR to the ASA (CW) for approval and to request congressional authorization for an increase in the total cost of the Folsom Modification project. In describing the project, the memorandum states:

"It [the comprehensive plan for improved FDR] also includes modification of the FC storage space in Folsom Lake from a variable space ranging from 400,000 to 670,000 acre-feet, to 400,000 to 600,000 acre-feet."

The PACR was transmitted by the ASA (CW) to Congress by letter dated 27 August 2007.

The Water Resources Development Act of 2007 (WRDA 07) authorized the changes to the Folsom Dam Modifications project in accordance with the PACR, resulting in approval of design and construction of the Folsom Dam JFP Auxiliary Spillway and the Folsom Dam Raise, both of which share an objective of improving flood risk management on the LAR primarily through structural changes to the existing Folsom Dam. As documented in the PACR, the without-project condition used for the evaluation of alternatives is the continued interim operation by Reclamation and SAFCA of a creditable flood space between 400 to 670 KAF. The with-project condition includes a permanent reoperation of 400 to 600 KAF as directed by WRDA 99.

The Flood Control Act of 1944 (FCA 1944) and implementing regulations hold the Corps responsible for prescribing operations for FRM at Folsom Dam including revision of the existing WCM.

2.3 Authorizations

2.3.1 Corps Authorities

Federal authorizations of the Folsom Dam Modification Project include the following:

1. The Flood Control Act of 1944, Pub. L. 78-58, § 7, 58 Stat. 890; 33 U.S.C. 709 and implementing regulations contained in 33 C.F.R. § 208.11. The Corps is responsible for prescribing operations for FRM at Folsom Dam. Through this authority, the Corps will be revising the existing WCM to account for the increased release capability of the new auxiliary spillway and an increase in authorized variable flood storage to 400,000 to 600,000 acre-feet.

2. The Water Resources Development Act of 1999, Pub. L. 106-53, § 101(a)(6), 113 Stat. 269, 274-75 (1999) (WRDA 99), authorizing:

"…The Folsom Dam Modification portion of the Folsom Modification Plan described in the United States Army Corps of Engineers Supplemental Information Report for the American River Watershed Project, California, dated March 1996, as modified by the report entitled, 'Folsom Dam Modification Report, New Outlets Plan,' dated March 1998, prepared by the Sacramento Area Flood Control Agency, at an estimated cost of $150,000,000, with an estimated Federal cost of $97,500,000 and an estimated non-Federal cost of $52,500,000."

3. WRDA 99 further provided interim direction regarding the operation of the Folsom Dam and Lake as follows:

"Upon completion of the [Folsom Modifications Project], the variable space allocated to flood control within the Reservoir shall be reduced from the current operating range of 400,000-670,000 acre-feet to 400,000-600,000 acre-feet."

"The Secretary…shall update the flood management plan for Folsom Dam…to reflect the operational capabilities created by the modification authorized in subparagraph (A) and improved weather forecasts based on the Advanced Hydrologic Prediction System of the National Weather Service."

4. The Energy and Water Development Appropriations Act of 2006, Pub. L. 109-103, § 128, 119 Stat. 2247, 2259-60 (2005) (EWDAA 2006) directed further study of the recommended modifications:

"The Secretary of the Army and the Secretary of the Interior are directed to collaborate on authorized activities to maximize flood damage reduction improvements and address dam safety needs at Folsom Dam and Lake, California. The Secretaries shall expedite technical reviews for flood damage reduction and dam safety improvements. In developing improvements under this section, the Secretaries shall consider reasonable modifications to existing authorized activities, including a potential auxiliary spillway. In conducting such activities, the Secretaries are authorized to expend funds for coordinated technical reviews and joint planning, and preliminary design activities."

5. The resulting PACR, prepared by the Corps, recommended changes to the Folsom Dam Modifications project (as well as the reduction of the dam raise authorized by The Energy and Water Development Appropriations Act of 2004, Pub. L. 108-137, § 128, 117 Stat. 1827, 1838-39 (2004) (EWDAA 2004) from 7 feet to 3.5 feet). The Water Resources Development Act of 2007, Pub. L. 110-114, § 3029, 121 Stat. 1041, 1112-13 (2007) (WRDA 07) authorized the changes to the Modifications project:

"...authorize the Secretary to construct the auxiliary spillway generally in accordance with the Post Authorization Change Report, American River Watershed Project (Folsom Dam Modification and Folsom Dam Raise Projects), dated March 2007…"

2.3.2 State Authorization

State authorizations of the Folsom Dam Modification Project include the following:

1. California Water Code Sections 8617, 12657, and 12670.14 authorized the State of California to cooperate on the Folsom Dam Modifications project, and the CVFPB to give satisfactory assurances to the Corps that the required cooperation will be furnished by the State in connection with the Project.

2. On 30 March 2004, the Corps entered the Project Cooperation Agreement with the CVFPB and SAFCA for construction of the Folsom Dam Modifications project, wherein the State shall be responsible for cost sharing of the Project during construction, which includes funding of the Manual Update.

2.3.3 Local Sponsor Authorities

Local sponsor authorizations of the Folsom Dam Modification Project include the following:

1. The SAFCA Board of Directors authorized SAFCA's Executive Director in January of 2004 to enter in a cost sharing agreement to fund improvements at Folsom Dam. SAFCA's Board further authorized the Executive Director in August of 2007 to enter an agreement to fund improvements associated with the JFP, including the Manual Update.

2. On 10 March 2004, CVFPB entered the Local Project Cooperation Agreement with SAFCA. Both boards agreed to jointly serve as the non-Federal sponsor of the project. SAFCA agreed to contribute the local cost share of the construction and fund the additional cost for operation, maintenance, repair, replacement, and rehabilitation of the JFP upon its completion.

2.4 Requirements for Revision of the Water Control Manual

The Manual Update is being prepared in accordance with instructions contained in the Corps publications: Engineer Manual (EM) 1110-2-3600, and Engineer Regulations (ER) 1110-2-240, and ER 1110-2-8156.

This Engineering Report serves as the basis for the Manual Update. It includes revisions to the WCD and ESRD, along with documentation and results of supporting technical analyses. Supporting technical analyses include development of hydrologic datasets, development of alternative operation plans, evaluation of flood performance, and evaluation of downstream effects. The Engineering Report is accompanied by a PACR and all required National Environmental Policy Act and California Environmental Quality Act (NEPA/CEQA) compliance documentation. These documents and supplemental technical documentation are also the basis of multiple levels of technical review, and serve as the basis for policy and legal compliance review.

The review of the Engineering Report and WCM is in accordance with Engineer Circular 1165-2-214 (EC 214), dated 15 January 2012. This circular establishes an accountable, comprehensive, life-cycle review strategy for Civil Works products by providing a seamless process for review of all Civil Works projects from initial planning through design, construction, operation and maintenance, repair, replacement and rehabilitation (OMRR&R). EC 214 does not explicitly address WCM updates, but is applicable given the cost, complexity and potential controversy associated with the Manual Update.

EC 214 outlines four general levels of review: District Quality Control/Quality Assurance (DQC), Agency Technical Review (ATR), Independent External Peer Review (IEPR), and Policy and Legal Compliance Review. The requirements and proposed scope of each of these levels of review for the Manual Update are described in Appendix B.

2.5 Vertical Datum

The elevations referenced in this report are in the North American Vertical Datum of 1988 (NAVD88) unless otherwise noted. Previous WCMs for Folsom Dam and Lake were in National Geodetic Vertical Datum of 1929 (NGVD29). The Manual Update will reference NAVD88. For Folsom Dam and Lake, an elevation can be converted from NAVD88 to NGVD29 by subtracting 2.34 feet, and from NGVD29 to NAVD88 by adding 2.34 feet.

3 Existing and Alternative Flood Operations

This Chapter describes development of alternative operations considered in the WCM update. Descriptions of the existing operations are also included, as these provided a starting point for development of the alternatives and also serve as baselines against which the alternatives are compared. There are two existing operations at Folsom Dam: 1) the Existing Corps operation as documented in the current 1987 WCM, and 2) the Existing Interim operation developed by SAFCA and implemented in 2004 by Reclamation. The WCM Update developed two alternative operations, both reflecting the additional release capacity provided by the new JFP spillway. These are referred to in this report as Alternative 1 and Alternative 2.

3.1 Folsom Flood Operation

This subsection describes the operational framework of the flood operations at Folsom Dam, and applies to both existing and proposed operations. The flood pool is the portion of reservoir space to be reserved (kept empty) for the purpose of maintaining a target level of downstream flood protection. It is bounded on the bottom by the guide curve, or top of the conservation (TOC) pool, which can vary by date or as a function of watershed state. When water is stored above TOC, the reservoir is said to be encroached. When encroached, water is released as rapidly as possible subject to operational and physical constraints. Under “normal” flood operations, releases are made for the purpose of providing downstream flood protection by safely conveying releases in the downstream leveed channel. The maximum release that can be made under routine flood operations is the normal objective release of 115 kcfs, and the maximum allowable pool elevation for normal flood operations is the top of flood pool (bottom of surcharge pool) at 468.34 feet NAVD88 (466.0 feet NGVD29). Once the objective release is being made, if the combination of current inflow and pool elevation are sufficiently great, the ESRD can require releases greater than 115 kcfs. When releases are governed by the ESRD, “emergency” flood operations are in effect and releases are made to prevent the dam from overtopping. The greatest release that can be made without overtopping downstream levees is the emergency objective release of 160 kcfs. ESRD releases can greatly exceed the emergency objective release.

Constraints on releases can be operational or physical. Operational constraints can limit the rate of change of reservoir releases. Other operational constraints include delays in downstream coordination efforts and delays in implementing gate changes to achieve the required release. The ESRD reflects physical constraints on how long tainter gates on the top of the main dam can be kept closed when the pool is in surcharge and rising. This is due to requirements to maintain freeboard to the top of these gates. The maximum release that can be made from a tainter gate is limited by how far the gate can be opened while maintaining a controlled release. Unanticipated constraints can also occur due to hardware failures.

3.2 Release Capacity of Existing and Auxiliary Spillways

This section describes spillway rating curves and summarizes operational constraints and general findings identified from physical and computer modeling of the main dam and auxiliary spillways. The existing condition spillways at Folsom Dam include five service and three emergency tainter gates on the top of the main dam. These gates all share the same invert elevation of 420.3 feet NAVD88 (418.0 feet NGVD29). Functionally, these eight gates are identical with the exception that vertically the three emergency gates are 3 feet taller than the five service gates when all gates are closed. Alternative operations described in this report reflect the additional release capacity provided by the new JFP auxiliary spillway. The auxiliary spillway includes six submerged tainter gates sharing an invert 50 feet lower in elevation than the main and emergency spillways.

Restrictions identified in this section pertain to maximum gate openings recommended to maintain controlled flow such that undesirable hydrodynamic loading on the gates is avoided. In the Manual Update, these restrictions are reflected in the ESRD, which is reflected in reservoir routing simulations.

3.2.1 Discharge Rating Curves for the Main Dam Tainter Gates

There have been several studies on discharge ratings for the main dam gates. Ratings for the main dam gates were officially established in *Folsom Dam Service and Emergency Spillways Discharge Curves*, January 2010 (referred to hereafter as the 2010 Technical Memorandum). The 2010 Technical Memorandum (TM) combines the results of several different model studies to develop the family of discharge curves: 1:36 scale Sectional Model (2009), the 1:48 JFP Confluence Model (2009), a FLOW-3D® numerical model of main dam spillway, and the 1:50 spillway model results. The discharge curves are shown in Figure 3-1.[1]

A separate project known as the Folsom Dam Raise calls for a 3.5-foot raise of the dam and modifications to the existing spillway gates. Additional modeling of the main dam gates (1:36 scale sectional model) was completed in 2014 in support of that effort, mainly to consider potential hydraulic effects of seismic bracing recently added to the dam (Reclamation, 2014a). No previous physical modeling had included the seismic bracing. While it was not the intent of the Dam Raise Study to generate new discharge curves for the main dam gates, there was some additional gate rating data developed. Comparison of the old and new rating data is documented in the *Folsom Dam Raise 95% Design Documentation Report for Gate Modifications*. The seismic beams have the effect of increasing discharge for a given reservoir elevation and, as a result, shift the discharge curve to the right. This may be because the seismic beams influence the flow path by directing it toward the gate opening. However, the differences in the discharge curves compared to the 2010 TM were not considered large enough to warrant change to the established curves.

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[1] Gate openings are based on the amount of travel of the hoist chain to the desired opening and are referred to as hoist chain travel (HCT). Gate openings based on HCT are used by field personnel to operate the gates.

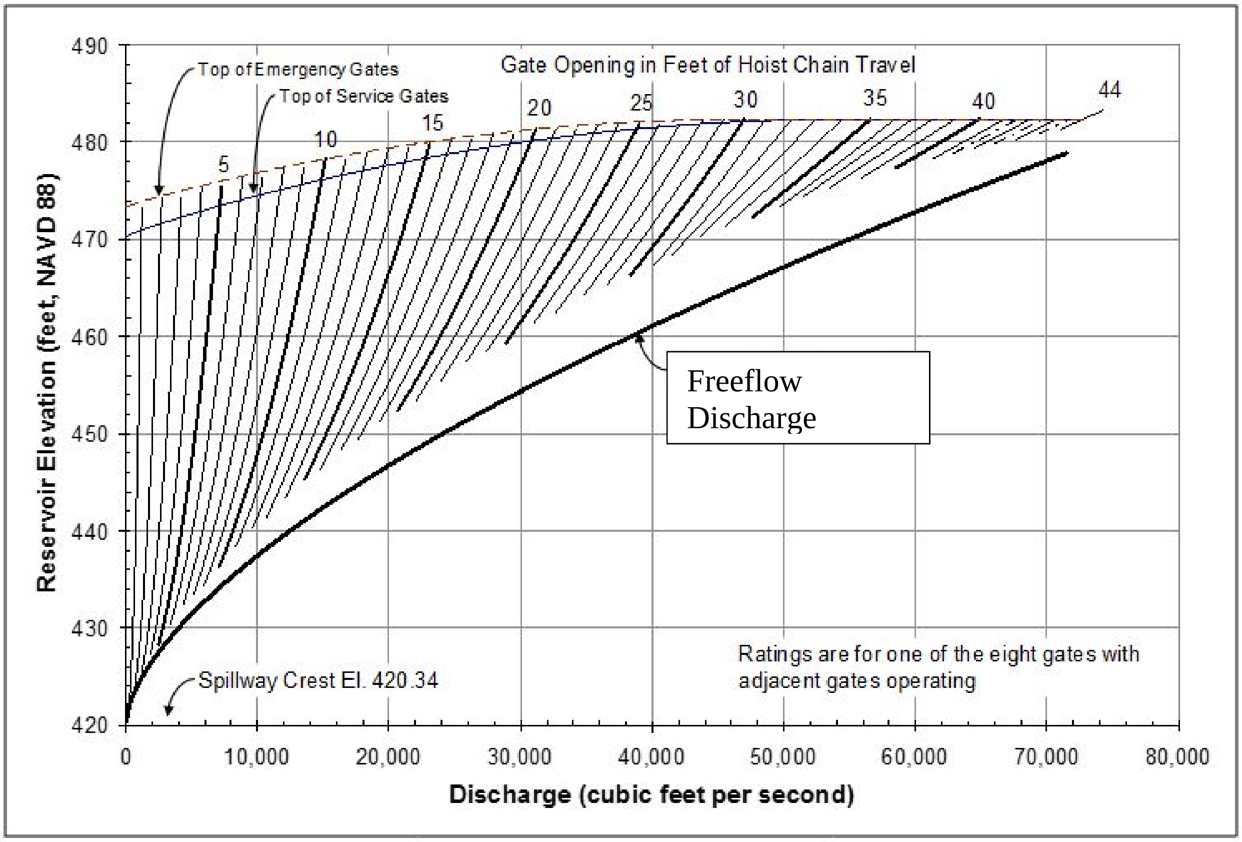


Figure 3-1: Main Dam Final Rating Curve

3.2.2 Main Dam Tainter Gates Full Open Limitation

It was discovered during the course of the Dam Raise Study, that significant turbulent conditions exist for large gate openings of the main dam tainter gates. Such conditions could lead to the undesirable hydraulic conditions and forces that could lead to damage. Potential problems include dynamic uplift forces that make it difficult to close gates; hydrodynamic forces that damage and/or fail seismic struts that lead to pier damage; as well as gate vibrations from dynamic loadings and trunnions impacts that lead to gate failure and/or inoperability. Conclusions from the model study were largely based on visual observation because the model did not have dynamic similitude for quantitatively capturing the hydrodynamic forces.

For a gate opening of 39.5 feet hoist chain travel (HCT) or greater, there were highly turbulent conditions under the bridge and around the seismic struts, including water impacting the trunnions. For a gate opening of 36 feet HCT, flow conditions were much more tranquil. For gate openings between 36 feet HCT and 39.5 feet HCT, the results were mixed, with a general tendency for flow conditions to worsen with larger openings.

Based on the model results, gate openings for all the main dam gates should be limited to 36 feet HCT during most operation scenarios to achieve acceptable flow behavior. Under extreme frequency events or operation scenarios, it may be necessary to open gates beyond 36 feet HCT to prevent reservoir encroachment into the dam freeboard and should only be undertaken considering the risks associated with operating gates in such a manner.

3.2.3 Discharge Rating Curves for the Auxiliary Spillway

Discharge curves for the auxiliary spillway were established based on the data from the 1:30 scale physical model (UWRL, 2009) and further documented in *Folsom JFP, Phase 4 Design Documentation Report*. The full open and part gate open discharge curves are shown on Figure 3-2[2].

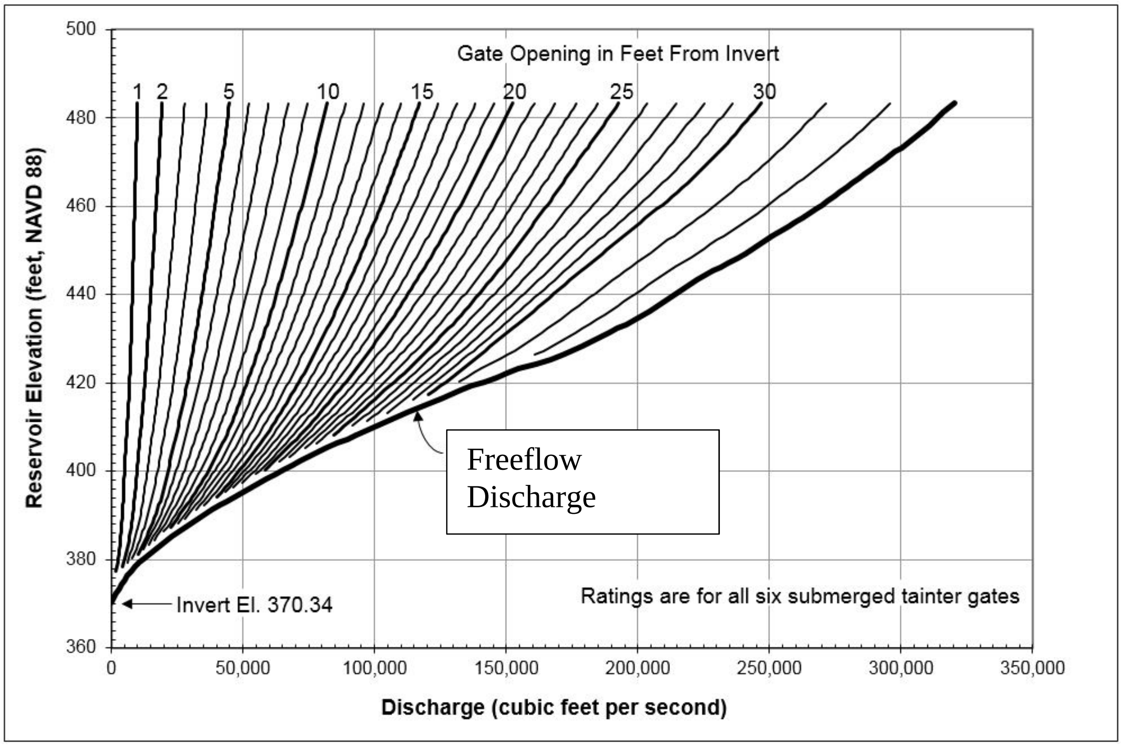


Figure 3-2: Auxiliary Spillway Control Structure Discharge Rating Curve

3.2.4 Auxiliary Spillway Tainter Gate Full Open Limitation

Subsequent to the 2009 report that established the discharge curves, another model study was conducted because of design changes to the approach channel (UWRL, 2013). Discharge curves were not the focus of the second study and new discharge curves were therefore not generated. However, some interesting observations were made at the condition of full open gates, necessitating further consideration as to acceptability of operating at the full open condition.

Further testing during the second model study resulted in the conclusion that the auxiliary spillway gates should be limited to 95 percent open (31.4 feet) except under extreme conditions, as opposed to 100 percent or fully open. At 100 percent open gates, the flow was observed to be

[2] The gate opening for the discharge curves was based on measuring the vertical distance between the bottom of the gate and the invert of the control structure upstream of the 1 percent grade break (at full open, 403.34 feet NAVD88 - 370.34 feet NAVD88 = 33.00 feet).

unsteady with an oscillating water surface with some splash and impact to gate trunnions. One hundred percent open gates also resulted in negative pressures along the roof curve downstream of the bulkhead gate slot. These issues were eliminated when gates were 95 percent open.

3.3 Spillway Operational Restrictions

This section describes operational restrictions for the main dam and auxiliary spillways. Restrictions related to maintaining required freeboard to the top of the service and emergency gates on the main dam are explicitly reflected in the ESRD and therefore also in simulations of reservoir operations in this study. Restrictions pertaining to the allocation of releases are described below to inform the development of a schedule for allocating releases among gates and spillways. Developing these operational restrictions and the corresponding release allocation strategy are the responsibility of the operating agency, Reclamation.

3.3.1 Current Limitations

Per Reclamation's 2002 Standard Operating Procedures, the eight lower river outlets are limited to 60 percent open while releases are made through service gates. Based on physical hydraulic model testing completed in 2014, the Corps concluded that "Based on the model results, gate openings for all the main dam gates should be limited during operation. To achieve acceptable flow behavior, gates should be limited to a 35 foot VGO under most operation scenarios." (Corps, 2014d). For greater gate openings, flow through gates becomes flow contacts gate seismic struts and trunnions.

3.3.2 Limitations on Opening Main Dam River Outlets to Prevent Cavitation

The current limitations on operating the main dam river outlets concurrent with main dam service spillway gate releases can be maintained. This is to limit the potential for cavitation at the exit portals of the main dam river outlets.

3.3.3 Opening Main Dam Service Tainter Gates to Prevent Overtopping

The top of service gates in the closed position is at elevation 470.34 feet NAVD88 (468.00 feet NGVD29). The service gates must commence opening when the pool reaches elevation 468.34 feet NAVD88 (466.00 feet NGVD29) to prevent overtopping of the service gates. For pool elevations above elevation 468.34 feet NAVD88 (466.00 feet NGVD29), the gate openings must be at least as large as the pool elevation rise.

3.3.4 Opening Main Dam Emergency Tainter Gates to Prevent Overtopping

The top of emergency gates in the closed position is at elevation 473.34 feet NAVD88 (471.00 feet NGVD29). The emergency gates must commence opening when the pool reaches elevation 472.34 feet NAVD88 (470.00 feet NGVD29) to prevent overtopping of the gates. For pool elevations above elevation 472.34 feet NAVD88 (470.00 feet NGVD29), the gate openings must be at least as large as the pool elevation rise.

3.3.5 Operations to Provide Tailwater Cushion for Emergency Spillway Releases

The 1:48 scale physical hydraulic model testing conducted by Reclamation's Hydraulic Investigations and Laboratory Services Group at the Technical Services Center in Denver, Colorado (Reclamation, 2010 and 2011) indicated that for conditions when the emergency spillway would be utilized, tailwater below the flip bucket will be pushed downstream by flows coming from the stilling basin. The result was that flows over the flip bucket in the model landed on the ground surface or the concrete slab downstream of the flip bucket. Figure 3-3 shows the flip bucket trajectory hitting the ground surface in the physical model. Without sufficient tailwater, the concrete slab will be damaged, leading to further instability of the area. Due to this risk, the total project release (flow from the auxiliary spillway and main dam) should be the maximum that can be achieved prior to making releases from the emergency spillway. This provides the best chance for developing a tailwater cushion to minimize damages to the area downstream of the flip bucket.

Figure 3-3: Emergency Spillway Flip Bucket Trajectory, 1:48 model

3.3.6 Main Dam Tainter Gates – Transition Zone

The “transition zone” describes an unstable region on the rating curve where flow transitions back and forth from freeflow discharge to gated (orifice) flow (see Figure 3-1). Operation should not take place within this zone because of the potential for highly erratic flow behavior including potential for development of vortices. Operating in this zone could also lead to loss of gate control and ultimately the inability to limit reservoir releases. The transition zone was established in the 2010 TM and analyzed further as part of the Folsom Dam Raise Study, though differences were not appreciable.

3.3.7 Flow Split between the Main Dam and Auxiliary Spillway

The primary concerns associated with managing the flow split between the main dam and auxiliary spillway are 1) potential erosion of the right bank directly across from the auxiliary spillway exit, and 2) how the confluence area of the American River and the auxiliary spillway exit affect the performance of the auxiliary spillway stilling basin.

Results of the 1:48 scale model (see references cited in Section 3.3.5) showed that the optimal split of flows from the main dam and the auxiliary spillways occurs when flows from the two structures are about equal. But the study also concluded that flows up to 160 kcfs coming from only the main dam or auxiliary spillway are still acceptable (though it was observed that the hydraulic jump is barely contained in the stilling basin of the auxiliary spillway at that discharge). The modeling further showed that flows from the main dam create better tailwater conditions for the auxiliary spillway stilling basin, making it less likely for the hydraulic jump to sweep out of the basin.

While flows split evenly between the spillways are ideal, it will not always be practical to achieve an equal split (especially with the pool elevation below crest 420.34 feet NAVD88 (418.00 feet NGVD29)). As part of the JFP, portions of the right bank are being stabilized with rock bolting to lessen the potential of erosion. Also, physical model results of various flow combinations from the main dam and auxiliary spillway did show that relatively small releases from the main dam, in combination with large releases from the auxiliary spillway, improve the hydraulic performance at the confluence

In the event an even flow split between the main dam and the auxiliary spillway is not practical or achievable for total project discharges up to 115 kcfs, releases coming only from the auxiliary spillway are acceptable unless adverse conditions are observed or arise during such an operation. For total project discharges exceeding 115 kcfs, it is recommended a minimum of 25 kcfs should be released from the main dam. Once 25 kcfs is reached for release from the main dam, and increased releases are still required, the increased releases should be divided up equally between the main dam and auxiliary spillway.

3.3.8 Auxiliary Spillway – Unbalanced Tainter Gate Operations and Minimum Gate Openings

Balanced operations, or simultaneously operating all gates at the same opening, is highly recommended to reduce the magnitude of cross waves and the potential for overtopping of chute walls. (Corps, 2016a). A minimum tainter gate opening of 2 feet is recommended when the pool elevation is above elevation 379.34 feet NAVD88 (377.00 feet NGVD29). If the pool elevation is lower, a minimum gate opening of 1 foot is recommended. If unbalanced tainter gate operations are unavoidable because of the malfunctioning of one or more gates, or minimum gate openings cannot be met to achieve the target discharge, the preferred operation would be to aim for balanced gate operations for the gates that are being utilized. Under this scenario, the gates on the outside should be opened first and the middle gates opened last. Opening the center gates without opening the outer gates would result in high velocity flows spreading from the control structure, impacting the chute walls, riding up the walls, and possibly overtopping the walls.

3.3.9 Main Dam Service Tainter Gate – Unbalanced Operations

Balanced operations for the service spillway tainter gates are needed to provide optimal energy dissipation in the stilling basin and limit circulatory and basin return flows in the area downstream of the basin. Considerable erosion and cavitation damages have occurred to the stilling basin invert in the past due to circulatory flows in the area downstream of the basin returning flows carrying rocks that grind on the concrete surfaces and results in major abrasion damage. To minimize this, Reclamation grouted the rocky area downstream of the stilling basin. However, balanced operation of the service spillway tainter gates will also help to minimize the damages to the stilling basin by reducing the probability of circulatory flows bringing rock back into the basin. If possible, the service gates should be operated with each gate being opened an equal amount. If for some reason this cannot be done, discharge from the gates that are being used on each side of the basin should be as balanced as is practical. Existing procedures in place to account for this should be followed.

3.3.10 Closure of Auxiliary Spillway Bulkhead under Flow Conditions

The bulkheads for the auxiliary spillway are not intended to serve as regulating gates and are normally only to be operated under balanced head conditions. If one or more of the submerged tainter gates fails or malfunctions, it may be necessary to block off the reservoir with a bulkhead before repairs can be made. Though design criteria called for operation of the bulkhead gates in an unbalanced fashion at pool elevations at or below 420.34 feet NAVD88 (418.0 feet NAVD29), analyses conducted during design and post-construction contract award did not fully resolve uncertainties in the performance of the bulkhead gates in an unbalanced head situation. For example, it was not known with certainty that the gate would fully close under unbalanced head. Further information on risks, limitations, and cautions in regards to unbalanced operation of the bulkheads is discussed in *Folsom JFP, Phase III Design Document Report.*

To verify these conclusions and help plot a path forward to resolve the issue, the Corps retained the services of Dr. Henry T. Falvey in January of 2017. Dr. Falvey concluded the current configuration of the bulkhead gate is not sufficient for them to act as emergency closure gates and that further study is required through CFD and physical modelling to understand how they would perform. Dr. Falvey's discussion and recommendations are included in a technical memorandum, *Review and Analysis of Folsom Dam Auxiliary Spillway Bulkhead Gate*, dated 15 February 2017 (Falvey, 2017). Operability of the bulkhead gates will be known with more certainty once the additional modelling efforts are completed, currently slated for completion in late 2017.

3.4 Summary of Existing and Alternative Flood Operations

The analysis of flood operations considered the two existing operations listed in Table 3-1 and the two alternative operations listed in Table 3-2. As indicated in column 1 of the tables, more than one name is used in the WCM Update to refer to an operation. The usage of names was driven by context and often by space limitations in plots in tables. The major components of the each flood operation are identified in columns 3, 4, 5, and 6 of the tables.

Table 3-1: Existing Flood Operations

Existing Operation Names (1)	Operation ID (2)	Guide Curve (3)	Winter Flood Space (KAF) (4)	Variable Flood Space Method of Computation (5)	ESRD (6)
"Existing Corps"	E503	1987 Corps	400 fixed, variable (< 400) in Feb. to Apr.	Precipitation index	1987 Corps
"Existing Interim" (NEPA/CEQA "No action" alternative)	E504	2004 Reclamation/ SAFCA	Variable 400 to 670	Upstream storage credit	1987 Corps

The Existing Corps operation represents the flood operation as defined by the December 1987 Folsom WCM. The Existing interim operation represents the 2004 SAFCA Interim Operation Plan, including the SAFCA WCD, minimum allowable release diagram, and ESRD from the Corps 1987 WCM. The Existing interim operation also serves as the "no action" alternative for the purpose of NEPA/CEQA evaluation. Additional adaptations of the operations in Table 3-1 were developed to support evaluation of NEPA/CEQA requirements. These included J604, which is the E504 operation but with future level of demand reflected in non-flood operations in period of record (POR) simulations.

Table 3-2: Alternative Flood Operations (with JFP)

Alternative Operation Names (1)	Operation ID (2)	Guide Curve (3)	Winter Flood Space (KAF) (4)	Variable Flood Space Method of Computation (5)	ESRD (6)
"Alternative 1," or "Credit-based"	J602P	Early spring refill	Variable 400 to 600	Updated upstream storage credit + basin wetness credit	Updated for this study
"Alternative 2," or "Forecast-based"	J602F	Early spring refill	Variable 400 to 600	Inflow forecast-based	Updated for this study

Alternatives 1 and 2 were the two final flood operation alternatives under consideration. Depending on context, the term “operation” or “alternative” may be used in conjunction with the names Credit-based and Forecast-based. Alternatives 1 and 2 share the same guide curve, which, compared to the two existing operations, has an early (higher) spring refill curve (see Figure 3-8). The two alternatives also share the same ESRD. The two alternatives differ in how required variable TOC is computed, as indicated in column 5 of Table 3-2.

Operation IDs (column 2 in Table 3-1 and Table 3-2) were used by the study team to name HEC-ResSim models and track data handoffs and subsequent analysis results. For example, the E503 operation was originally modeled with an HEC-ResSim model by the same name. Later in the study, the model was improved and renamed to E503P. Similarly, HEC-ResSim models J602P3 and J602F3 were the third (and final) iteration of operational rules reflecting these operations. HEC-ResSim model IDs are found in some tables and figures in this report.

Not all operations considered in the study are listed in Table 3-2. For example, early in the study operation J602 was developed, which reflected a truncated version of the E504 WCD with an updated upstream storage credit relationship designed to support 400 to 600 KAF variable flood space at Folsom. This was a preliminary operation, and was later dropped in favor of Alternative 1, which uses both basin wetness and upstream storage credit in the computation of variable TOC.

3.5 Existing and Alternative Guide Curves

Normal flood operations are defined in the WCM by the WCD. The WCD defines the method for computing variable TOC. During much of the year, the TOC is defined entirely by a seasonal guide curve, in which the TOC storage value is specified as a function of date. When storage is greater than this value, the flood pool is encroached and releases are made to evacuate the flood space. When storage is less than this value, non-flood releases are made. The WCD can also specify variable TOC. In this case, the seasonal guide curve defines the upper and lower envelope of possible variable TOC values. The specific value of variable TOC is computed based on information reflecting the state of the watershed. The purpose of variable TOC is to require less flood space when, based on watershed conditions, FRM performance will not be reduced.

The Existing Corps operation is the official Corps operation and is defined in the 1987 WCM for Folsom Dam and Lake (Corps, 1987a). With this operation, the maximum required winter flood space is 400 KAF. From 8 February to 21 April, the operation transitions from winter to spring refill operations. During this time, the variable TOC can vary depending on basin wetness. Basin wetness is given by the precipitation index, which is dependent on estimates of basin-wide precipitation as indicated by precipitation gages. The guide curve for the Existing Corps operation is shown in Figure 3-4, with hatched area indicating variable flood space.

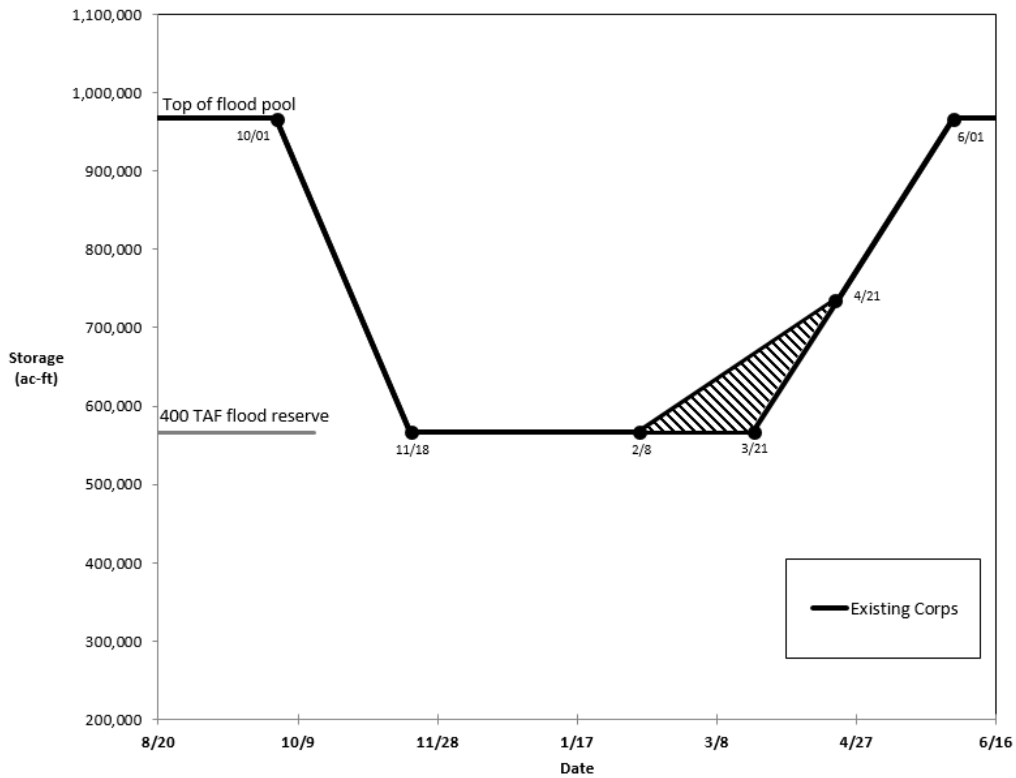


Figure 3-4: Guide Curve – Existing Corps

The Existing Interim operation was developed by SAFCA in recognition that greater FRM performance would be achieved if the required winter flood space were allowed to vary from 400 to 670 KAF, depending on upstream reservoir storage conditions. The operation includes a relationship between upstream creditable space (ranging from 0 to 200 KAF) to credited space at Folsom (ranging from 0 to 270 KAF respectively). Upstream reservoirs considered in the computation of credit storage are: French Meadows, Hell Hole, and Union Valley. An upstream creditable storage of 200 KAF means that 270 KAF will be subtracted from the maximum variable space requirement of 670 KAF to require only 400 KAF flood space at Folsom. An illustration of the guide curve for the Existing Interim operation is shown in Figure 3-5, with the shaded area indicating variable flood space.

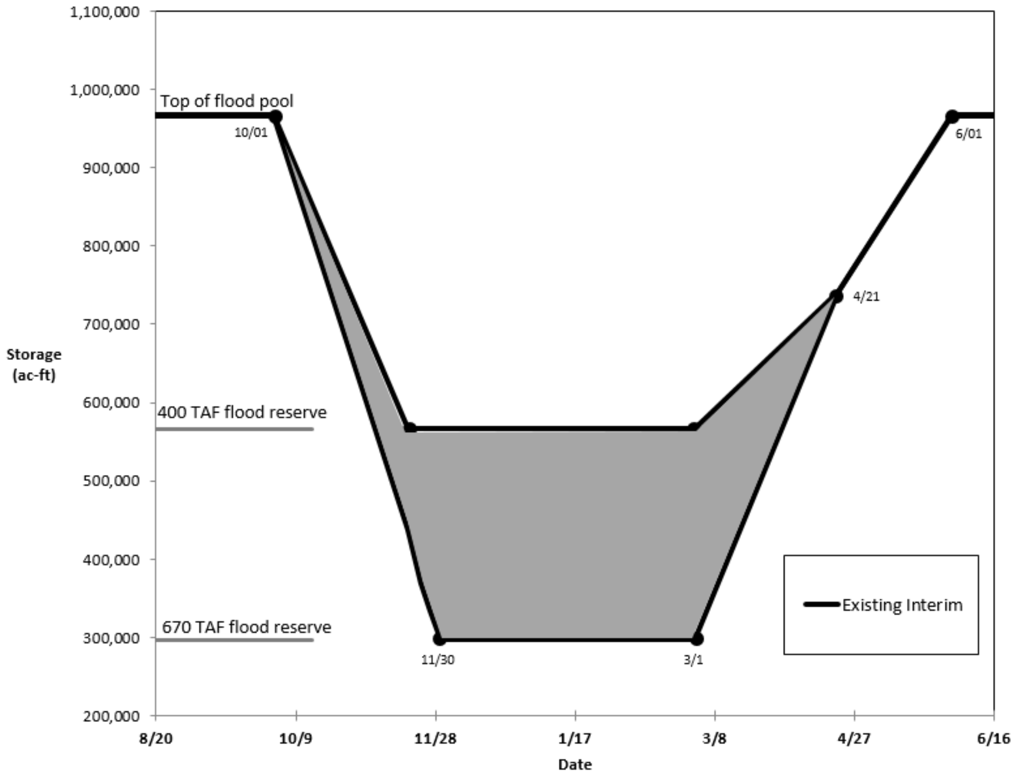


Figure 3-5: Guide Curve – Existing Interim

In the Manual Update, the current standard for flood protection is the Existing Interim operation. Therefore, comparisons of FRM performance and NEPA/CEQA affects analyses to the existing condition will reference the Existing Interim operation as the baseline, or No Action/No Project, condition. However, the Existing Corps operation is being used as a past baseline operation condition to evaluate cumulative effects in the NEPA/CEQA analysis. As such, both existing operations are carried forward in the WCM Update. For comparison, both existing condition guide curves are shown in Figure 3-6.

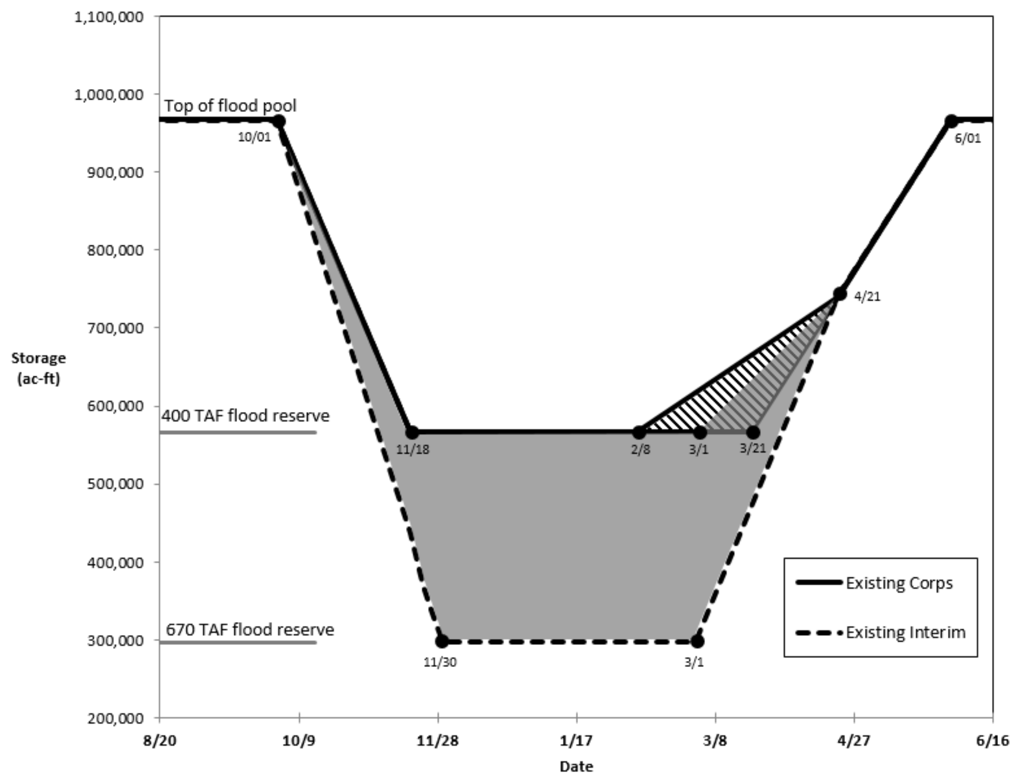


Figure 3-6: Guide Curves – Existing Corps and Existing Interim

A new guide curve was developed for Alternatives 1 and 2. It is shown in Figure 3-7 and tabulated in Table 3-3. For comparison with existing guide curves it is also shown in Figure 3-8. The new guide curve defines a winter variable flood space requirement ranging from 400 to 600 KAF. This range was required by authorization language in WRDA 99.

The fall portion of the new guide curve, from 1 October to 18 November, is coincident with the Existing Corps diagram. The new spring refill curve (1 March to 1 June) allows for earlier refilling of the reservoir than the existing guide curve, though the new curve does not allow refilling to begin until 1 March. Variable flood space of 400 to 600 KAF is in effect from 19 November to 28 February. The methods of computing variable TOC for Alternatives 1 and 2 are described in Section 3.6.

The spring refill portion of the proposed (Alternatives 1 and 2) guide curve was developed by simulating reservoir operations for seasonal events. Development of these events is described in Chapter 7. Simulations of ACE = 1/100 and 1/200 seasonal events were made and Folsom starting storage values tested that resulted in releases not exceeding 40, 60, 90, and 115 kcfs. The maximum acceptable starting storage for each maximum release was plotted versus event start date to suggest candidate refill curves supporting hypothetical maximum release values. The shape of the spring refill curve was based on results from this exercise. Later in the study, once the seasonal frequency curves, ESRD, and Alternative 1 and 2 operations were solidified, the seasonal events were again tested. As seen in Table 6-11 and Table 6-12, all ACE=1/100 and ACE=1/200 seasonal (March, April, and May) events were routed at 115 kcfs or less. This confirmed that the selected spring refill curve satisfied FRM performance requirements.

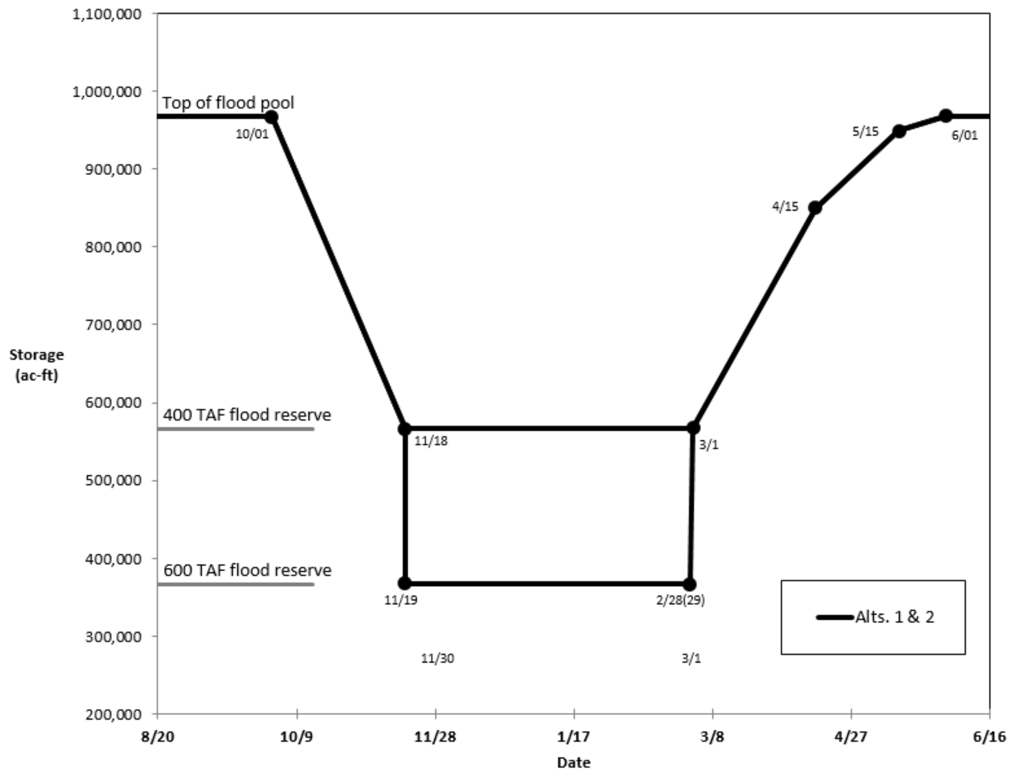


Figure 3-7: Guide Curve – Alternatives 1 and 2

Table 3-3: Guide Curve – Alternatives 1 and 2

Date	Non-Flood Storage (KAF)	Flood Space (KAF)
01 October	967	0
18 November	567	400
19 November	367 to 567 (variable)	400 to 600 (variable)
28 (29) February	367 to 567 (variable)	400 to 600 (variable)
01 March	567	400
14 April	850	117
15 May	950	17
01 June	967	0

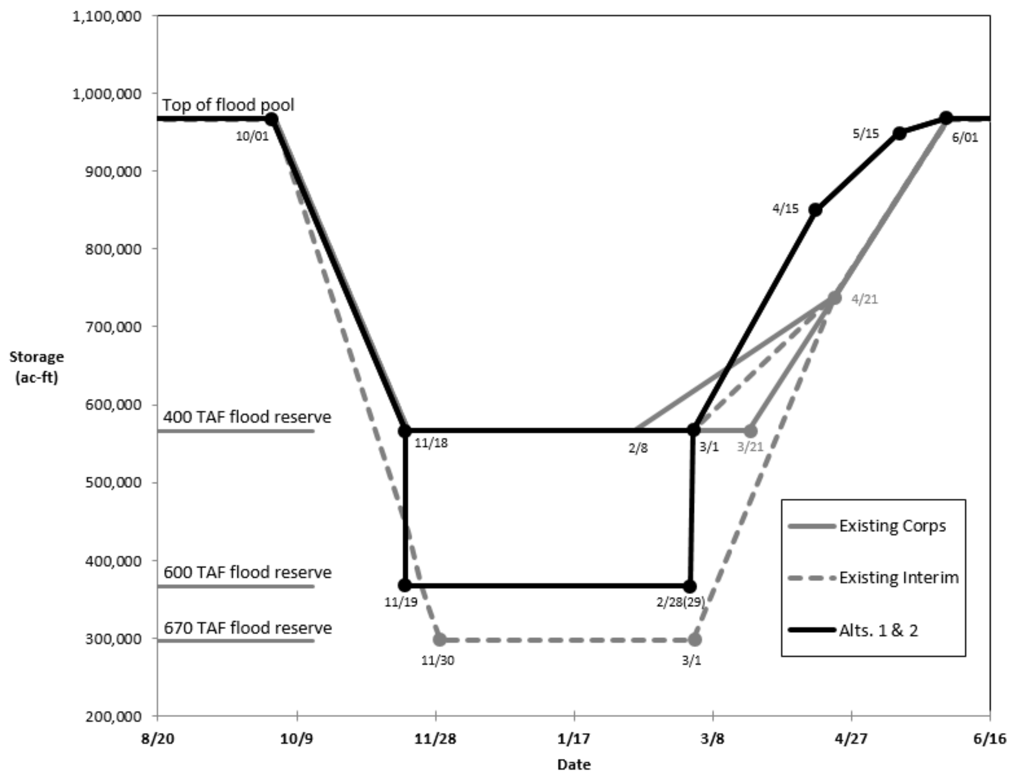


Figure 3-8: Guide Curves – Existing Corps, Existing Interim, and Alternatives 1 and 2

3.6 Alternatives 1 and 2 – Variable Top of Conservation

The primary difference between Alternatives 1 and 2 is the computation of variable TOC, which is in effect from 19 November to 28 February. During this winter period, variable flood space ranges from 400 to 600 KAF, which corresponds to 567 to 367 KAF storage. The only other difference between the two alternatives is that Alternative 2 also includes a forecast-based release schedule, which is described in Section 5.4.

With Alternative 1, separate credit volumes, one based on creditable space in upstream reservoirs and the other based on basin wetness, are combined to obtain the combined storage credit at Folsom. The total upstream creditable space is based on space available at French Meadows, Hell Hole, and Union Valley reservoirs. Basin wetness will be computed by the CNRFC and is provided as a parameter referred to in the Manual Update as the "RFC index." RFC index values of 0 and 1 reflect dry and saturated watershed conditions, respectively. Equation 3-1 provides the relationship used to compute the combined storage credit at Folsom.

Equation 3-1:
Combined storage credit (KAF) =

$$[1{,}100 \text{ KAF}] * (1 - \text{RFC index}) + (200/221)*[\text{total upstream creditable space (KAF)}]$$

Thus, a greater credit yields a higher TOC. The combined storage credit is added to 367 KAF, and the lesser of that result and 567 KAF is the variable TOC value for the day.

With Alternative 2, variable TOC is computed from inflow forecast information. 24-, 48-, 72-, and 120-hour inflow forecast volumes are used as input for the computation. The computation is done using the "drawdown curves" in Figure 3-9. Inflow forecast volumes for the four durations are used to enter the diagram from the X axis, and corresponding TOC storage values for each volume duration are located on the Y axis. Of the four TOC storage values obtained, the smallest value is the variable TOC value until the next forecast is issued. Required flood space will only be greater than 400 KAF if the 120-hour inflow forecast volume is greater than 300 KAF.

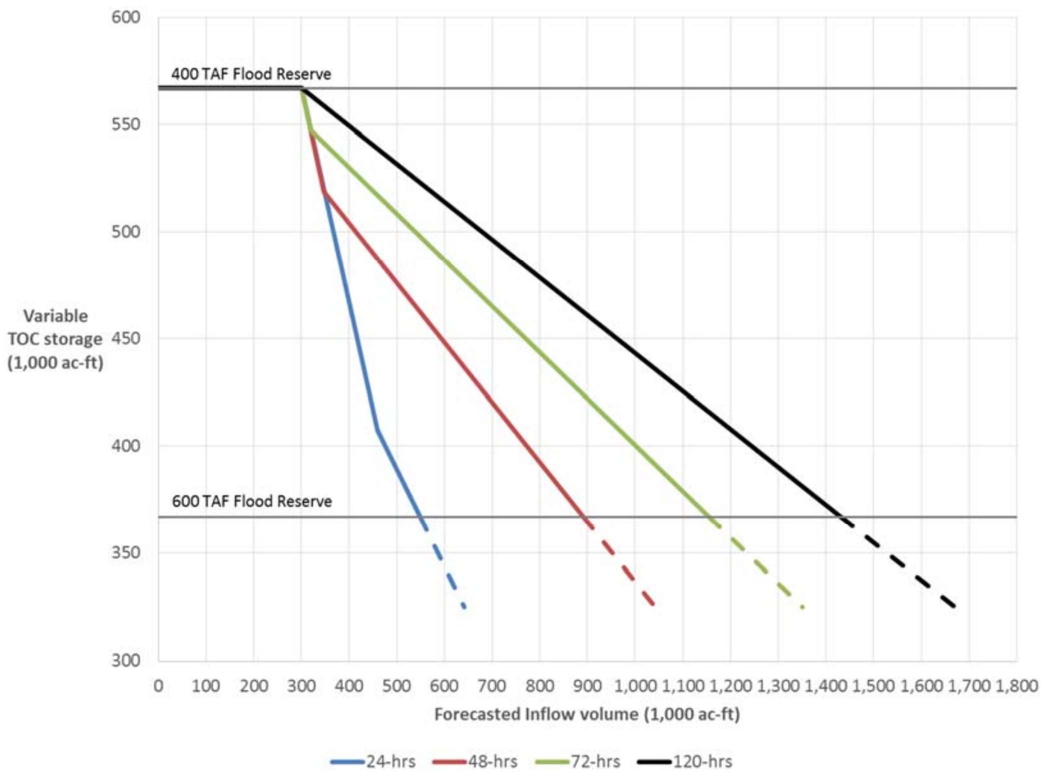


Figure 3-9: Drawdown Curves for Alternative 2 Variable TOC Computation

3.7 Alternatives 1 and 2 – ESRD

The ESRD developed for Alternatives 1 and 2 is documented in Appendix F. The development process is described and routing results for PMF and other events are provided. The ESRD routes all PMF events with at least 3 feet of freeboard.

3.8 Alternatives 1 and 2 – Stepped Releases

Alternatives 1 and 2 both feature stepped releases during normal flood operations. Release steps provide a framework for making well-behaved and predictable releases. This supports Reclamation's role in coordinating with other agencies to prepare and evacuate the downstream flood channel during an event. The stepped release values are shown in Table 3-4, and were selected considering downstream coordination and general channel and levee erosion potential.

Table 3-4: Stepped Releases – Alternatives 1 and 2

Release Step (kcfs)	Downstream Floodway Consideration
8	None
25	At 10 kcfs, low-lying park areas inundated At 15 kcfs, areas of Campus Commons Golf Course and segments of the American River Parkway bike trail are inundated At 20 kcfs, areas of Discovery Park are inundated
50	At 30 kcfs, Arcade Water District must turn off their river intake At 45 kcfs, the Sacramento County bike bridge is inundated and damaged At 50 kcfs, Carmichael Water District access road is damaged
80	At 65 kcfs Significant stretches of the American River Parkway bike trail are damaged
115	Damage occurs at the Nimbus Fish Hatchery with bank erosion occurring in many places along the LAR channel

Table 3-5 lists conditions which must be satisfied before increasing releases to the next release step. At any time during the year in which the flood pool is encroached, Alternative 1 stepped releases are achieved as a result of making releases to pass inflow, subject to rate of increase constraints. Outside the period of variable TOC (19 November to 28 February), Alternative 2 operation is identical to Alternative 1. During the period of variable TOC, Alternative 2 stepped releases are made in response to the forecasted inflow volume. Forecast-based stepped releases are intended to evacuate the variable flood space, thus drawing the reservoir down while current inflow values are relatively small but expected to increase substantially.

Table 3-5: Stepped Release Thresholds – Alternatives 1 and 2

Release Step (kcfs)	Condition Requiring Increasing Release to Indicated Release Step	
	Alternative 1 (1 Oct to 1 Jun), and Alternative 2 (1 Oct to 18 Nov and 1 Mar to 1 Jun)	Alternative 2 (19 Nov to 28 Feb)
25	Release maximum event inflow	Forecast-based (see Table 5-1)
50	Release maximum event inflow	Forecast-based (see Table 5-1)
80	Release maximum event inflow	Forecast-based (see Table 5-1)
115	Release maximum event inflow	Forecast-based (see Table 5-1)

3.9 Alternative 2 – Inflow Forecast Variability and Uncertainty

Alternative 2 depends directly on forecast information for events forecast to have a 120-hour volume greater than 300 KAF. Accounting for uncertainty in forecast information is important for both simulations and operations. Inflow volumes required by the forecast-based operation are computed from the CNRFC ensemble forecast product of 60+ inflow hydrographs. Variability among the inflow hydrographs of an ensemble reflects uncertainty in precipitation and temperature forecasts. In order to assess operational robustness, or vulnerability, to forecast variability, robustness testing (Section 6.5.2) was performed. To support these tests, CNRFC generated synthetic ensembles to correspond with 1/100 and 1/200 ACE scaled inflow events having the water year (WY) 1986 and WY 1997 historical event patterns.

3.10 Alternatives 1 and 2 – Performance Metrics Guiding Development

Metrics for evaluating FRM performance were defined by the two FRM goals listed in Section 1.1. These goals were to route ACE=1/100 and 1/200 events without exceeding 115 and 160 kcfs peak release respectively. In terms of formulating the operational rules of Alternatives 1 and 2, the degree to which these goals were satisfied guided development of the flood operation. Specifically, development of Alternatives 1 and 2 considered:

1. The largest (smallest ACE) synthetic event that can be routed at 115 kcfs peak release
2. The largest (smallest ACE) synthetic event that can be routed at 160 kcfs peak release

In addition to tabulating these ACE values (Table 6-24), regulated frequency curves spanning a range of ACE were plotted (Figure 6-14) and compared (Table 6-23).
These metrics for FRM performance do not consider any effects of the operation other than peak release. Effects of Alternative 2, the selected alternative, were subsequently considered in additional analyses as described in Chapter 8.

Evaluation of dam safety, as considered in the formulation of Alternatives 1 and 2, considered minimum freeboard to top of dam when routing the PMF events. The requirement of maintaining at least 3 feet freeboard was found to be entirely dependent on configuration of the ESRD. The ESRD, documented in Appendix F, was found to satisfy the dam safety requirement in Alternatives 1 and 2, with nearly exactly 3 feet of freeboard to top of dam in both cases. As such, beyond configuration of the ESRD, dam safety did not play a role in developing operations for Alternatives 1 and 2.

4 Alternative 1 – Credit-based Flood Operation

4.1 Development of Credit-based Flood Operation

The Existing Interim upstream credit storage relationship was updated to support routing 1/200 ACE events with peak release not exceeding 160 kcfs. Starting storage conditions, ranging from 400 to 600 KAF flood space at Folsom Lake, and upstream starting conditions defined by candidate relations were used in simulations of the 1/200 ACE 1986 and 1997 event patterns. These two event patterns are from the two largest events of the period of record. The WY 1986 event was a colder double-peaked event, with a greater portion of total basin precipitation falling downstream of the headwater reservoirs. The WY 1997 event was a warmer single-peaked event, with a larger portion of total basin precipitation falling above the headwater reservoirs.

The updated relationship for credited storage at Folsom (Y axis) as a function of total upstream creditable space (X axis) is shown in Figure 4-1. Total upstream creditable space, as a function of storage in the three credit reservoirs, is given by Equation 4-1.

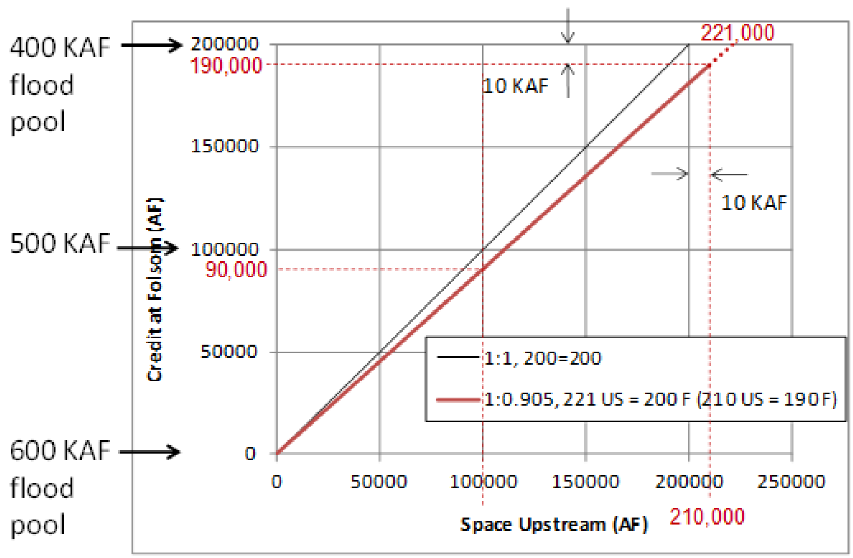


Figure 4-1: Updated Storage Credit Relationship

Equation 4-1:
Total upstream creditable space (KAF) = min[55 KAF, max(0, 110.7 KAF - French Meadows storage)] + min[91 KAF, max(0, 207.6 KAF - Hell Hole storage] + min[75 KAF, max(0,225.1 KAF - Union Valley storage)]

The relationship for storage credited to Folsom, shown in Figure 4-1 is given by Equation 4-2.

Equation 4-2:
Credit at Folsom (KAF) = (200/221)*[total upstream creditable space (KAF)]

The other credit component of Alternative 1 is basin wetness. The first step in computing the basin wetness credit will be done by the CNRFC, which computes the "RFC index" value. The method of computing the RFC index is described in Appendix C. The range of the index is 0 to 1, reflecting dry to saturated watershed conditions. The daily variation of the RFC index over an example year (WY 1981) is illustrated in Figure 4-2. The upper graph indicates daily basin average precipitation and the lower graph indicates the RFC index.

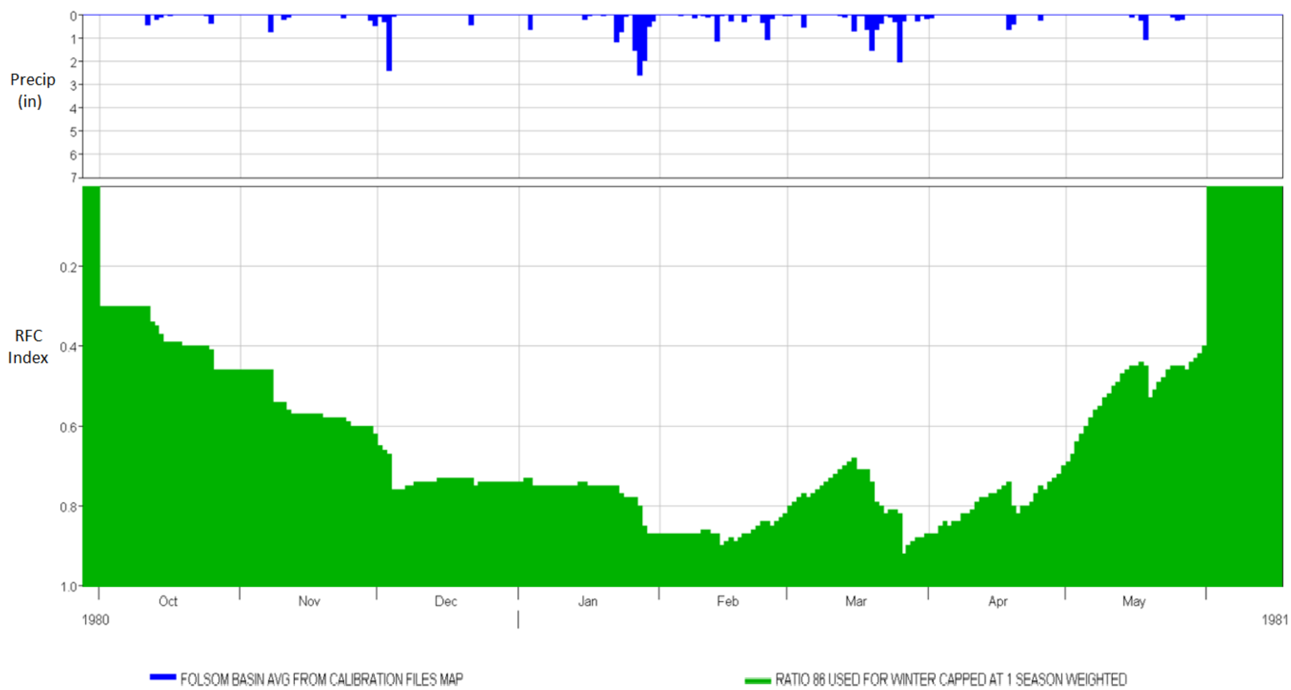


Figure 4-2: Water Year 1981 Daily Variation of Basin Wetness (RFC index)

An analysis was undertaken to define the relationship between the RFC index and required event starting flood space needed at Folsom to route ACE=1/200 events scaled from the historical events of WY 1956, 1964, 1986, and 1997. Two analysis approaches were taken: Determine the amount of starting space at Folsom needed to successfully route these events 1) within the flood pool ("max elev < 466" in Figure 4-3), and 2) with maximum release (QR) less than 160 kcfs ("max QR < 160 kcfs" in Figure 4-3). Results of these simulations are shown in Figure 4-3. The dashed line shown in both plots is the adopted lower envelope of results, and is the adopted linear relationship between required flood and RFC index. The relationship is provided in Equation 4-3, and takes on values of 400 and 600 KAF for RFC index values of 0.82 and 1.0. The slope of the relationship is 1,110 KAF per unit RFC index.

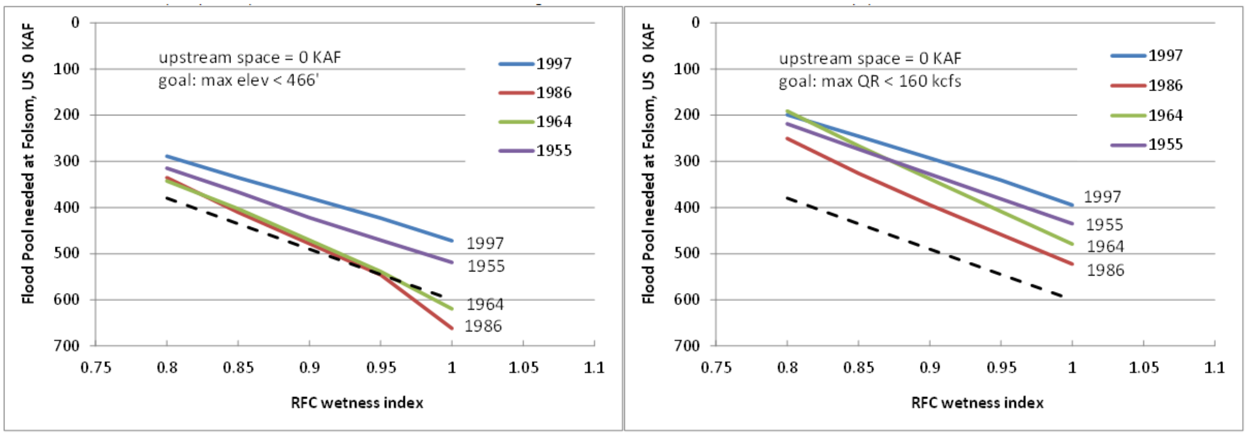


Figure 4-3: Required Flood Space vs. RFC Index

Equation 4-3:
Required flood space (KAF) = 1,110 KAF *RFC index – 510 KAF

Subtracting both sides of Equation 4-3 from 600 KAF, and identifying the left side of the equation as credited storage at Folsom as a function of RFC index gives:

Equation 4-4:
Basin wetness storage credit at Folsom = 1,110 KAF – 1110 * (RFC index)
= 1,110 KAF * (1 – RFC Index)

Equations 4-2 and 4-4 are combined to give the combined storage credit at Folsom Lake resulting from upstream storage and basin wetness:

Equation 4-5:
Combined storage credit at Folsom (KAF) =
(200/221)*[total upstream creditable space (KAF) + [1,100 KAF] * (1 - RFC index)

The combined storage credit is added to 367 KAF (storage corresponding to maximum required flood space of 600 KAF), and the lesser of that result and 567 KAF (corresponding to minimum required flood space of 400 KAF) is the daily-updated variable TOC. This computation was scripted in the HEC-ResSim model of Alternative 1 and would be used in a real-time operation if Alternative 1 were to become the selected operation.

5 Alternative 2 – Forecast-based Flood Operation

Folsom Dam has substantial downstream channel capacity (115 kcfs normal objective release) relative to the size of the reservoir (400 to 600 KAF winter variable flood space). With the JFP spillway, the flood pool no longer must be significantly encroached before flood releases near the objective release can be made. The watershed upstream of Folsom is steep, and excess precipitation on the watershed enters the reservoir quickly. At any time, the volume of water in the watershed that will eventually flow into Folsom Lake is comprised of snowpack, excess precipitation, and upstream reservoir storage. The Corps' best practice of operating to "rain on ground" is of limited utility at Folsom for informing flood operations, as the majority of excess precipitation and snowmelt will enter the reservoir within 12 hours. Thus, a rain on ground operation allows only hours for operational decisions to be made and implemented. Use of forecast information and real-time hydroclimate information provide potential for greater lead time to act. Further, forecast information provided by the CNRFC uses the current state of the watershed as the initial condition for forecasts. As a result, rain on ground is included in the forecast. The current WCM contains general language indicating that forecast information should be considered in the process of making release decisions. The forecast-based alternative formalizes rules for computing the required winter variable flood space as a function of forecasted inflow volume.

5.1 Overview of the Forecast-based Alternative

Alternative 2, the forecast-based operation, relies on forecast information generated by CNRFC, which supports the use of this information for defining flood operations at Folsom. The information is used for two purposes: 1) to compute a forecast-based TOC during the portion of the year in which variable TOC is in effect, and 2) if the reservoir is encroached above the forecast-based TOC, to compute the required release. The intended effect of this approach is to initiate releases greater than inflow in advance of the flood regulation portion of the event, while inflows are still sufficiently low enough to allow a controlled drawdown of the reservoir leading into the event.

A potential benefit to water supply is that the variable TOC is allowed to remain at the highest storage level (or minimum required flood space of 400 KAF), except immediately preceding and during a large event. Unlike alternatives relying on basin wetness and/or upstream storage credit, the TOC returns to the highest allowed level once the event has passed, providing improved opportunity for the reservoir to refill up to or higher than the pre-event storage level. This operation is consistent with the operation specified in the 1987 WCM.

The CNRFC already operates a comprehensive precipitation runoff model of the watershed upstream of Folsom Lake. The model is updated with observed data including measured precipitation, current storage levels at headwater reservoirs, and the current inflow into Folsom Lake. It is further supplied with an ensemble of precipitation and temperature forecasts. As such, the resulting CNRFC inflow forecasts reflect both current and forecasted inflow, basin wetness, and upstream reservoir storage conditions. The resulting forecast products do not require further routing or transformation by the Corps to obtained Folsom impaired inflow hydrographs.

Inflow forecasts present unique challenges in developing a reservoir operation scheme. The primary challenge is the simple fact that forecasts are not perfect. While forecast skill has been improving over the years, and will continue to improve, understanding and accounting for the degree of variability in forecasts is a requirement. A second challenge is that given the variability of forecasts, and variability of inflows even if forecasts were perfect, there is a need to make well-behaved (non-erratic) releases. This is an important consideration for dam operations as well as minimizing downstream impacts and supporting coordination efforts.

5.2 Ensemble Inflow Forecast Product

CNRFC ensemble inflow forecasts will be used to support the forecast-based operation. These will be issued once daily during normal (non-flood) operations, and every 6 hours during forecast-informed flood operations. For Folsom, an ensemble is presently comprised of 62 forecasted inflow hydrographs. With each water year, the number of inflow hydrographs comprising the ensemble increases by one. The inflow hydrographs are on an hourly time step. These are impaired inflow hydrographs, with each hydrograph reflecting upstream storage conditions and basin wetness. The product ID of the impaired (regulated) inflow hydrograph to Folsom Lake is FOLC1R. Appendix D further describes ensemble forecasts and their development.

CNRFC began issuing ensemble inflow forecasts in December 2010, but has been producing the underlying data required to create an ensemble forecast since January 1985. In 2015, CNRFC performed a simulation synthesizing hypothetical ensemble forecasts with the historical information from 1985 through 2010 in order to extend the ensemble forecast dataset. These forecasts are referred to as hindcasts. These are useful for allowing testing of the forecast-based operation on events that occurred prior to the implementation of the ensemble forecast system at CNRFC. The WY 1986 and 1997 storm events were the largest events to occur during the hindcast period of record. Hindcasts and inflows for these two events, scaled up to reflect 1/200 ACE runoff volumes, were used to test the forecast-based operation.

Figure 5-1 and Figure 5-2 show the progression of the inflow hindcast ensemble for the 1997-pattern event scaled to 1/200 ACE. The vertical line in each plot illustrates the time of forecast. The date of hindcast is indicated in the top right of each plot. Vertical gridlines correspond to days on the X axis. The solid black line in each figure represents the synthetic inflow for the 1997-based 1/200 ACE event and remains unchanged between figures. The series of daily plots illustrate that the general magnitude and duration of a large event can be detected multiple days in advance of its arrival. As the event approaches, the individual member hydrographs cluster more closely together. Also, as the event approaches, white space below the ensemble of hydrograph appears, showing that even the smallest forecasted inflows indicate that an event is approaching (see 12/30/1996 plot for example). When in the middle of the event and inflows are greatest, all hydrographs signal the end of the event (see 1/2/1997 plot). While there is variability among the individual inflow hydrographs, taken as a whole, the signal of a large event approaching is strong and actionable.

The forecast-based operation is described further in Section 6.5. In that operation, inflow forecast volumes are computed for four durations of interest. Each duration begins at the time of forecast, and the corresponding duration volume is computed for each ensemble member.

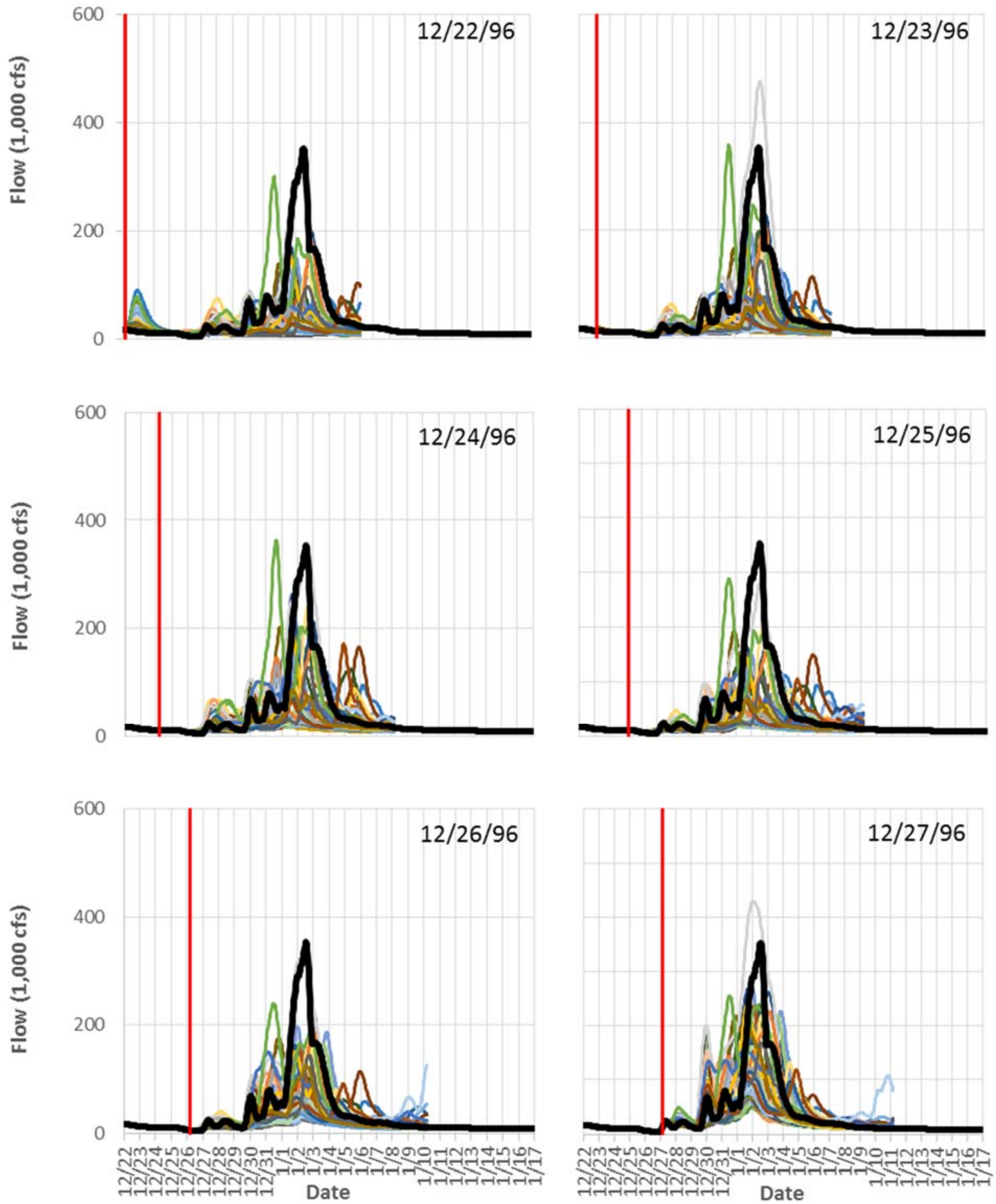


Figure 5-1: Daily Hindcasts for the 1997 ACE=1/200 event: 12/22/1996 through 12/27/1996 Issued at 4 a.m. PST (12 p.m. GMT)

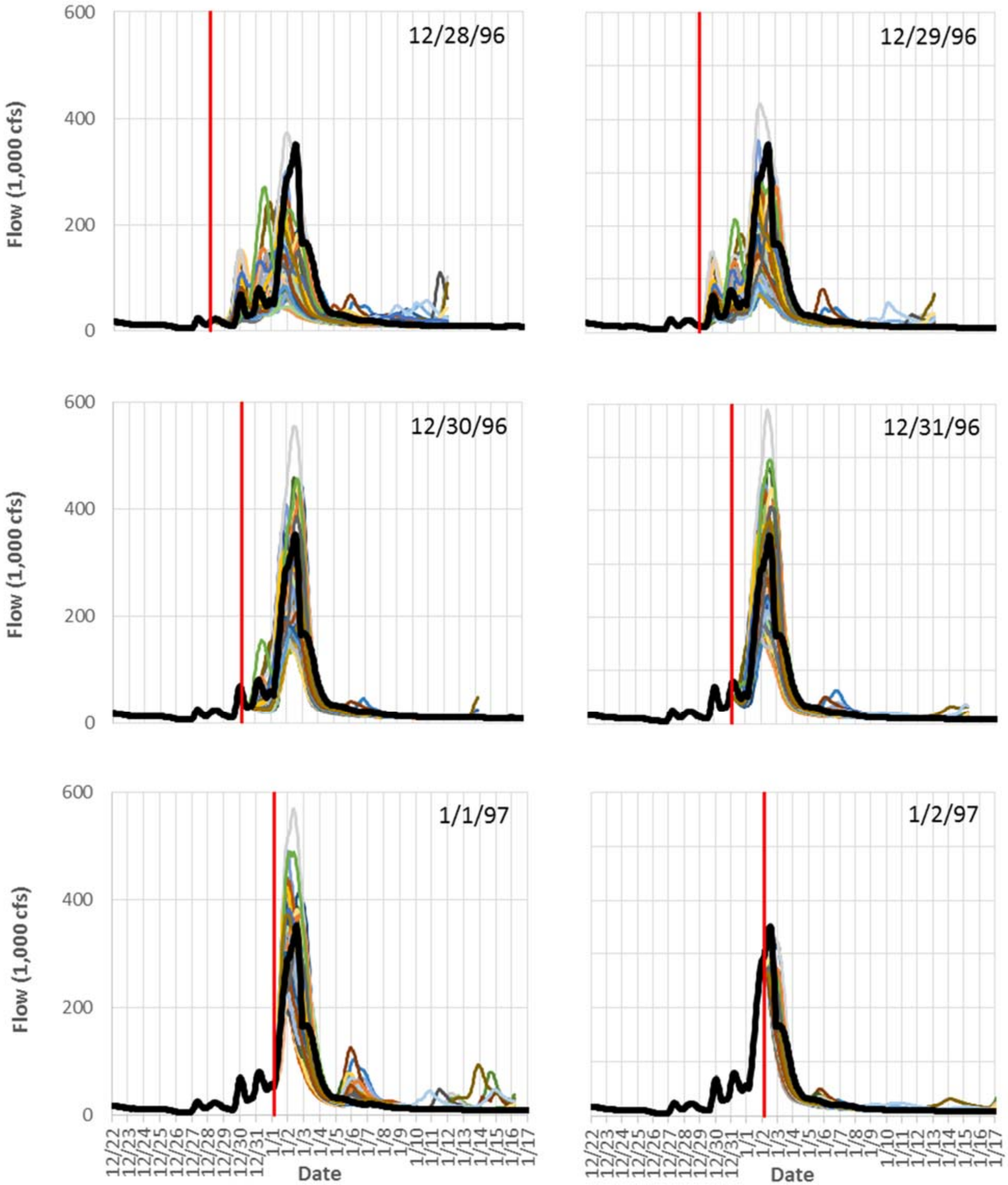


Figure 5-2: Daily Hindcasts for the 1997 ACE=1/200 event: 12/28/1996 through 1/2/1997 Issued at 4 a.m. PST (12 p.m. GMT)

5.3 Forecast-based Variable TOC

Variable TOC, is in effect from 19 November to 28 February. During this time, required flood space varies from 400 to 600 KAF (567 KAF to 367 KAF TOC storage). A method was sought in which TOC could be made to vary based on inflow forecasts such that:

1. When a 1/200 ACE event is in the forecast, then the TOC would drop, causing the reservoir to be drawn down ahead of the event arriving, so as to maximize the amount of available flood space available for routing the event.

2. When no event, or small events are forecast, then the minimum flood space of 400 KAF would be required.

3. When an event sufficiently large to trigger forecast-based drawdown of the reservoir does occur, that the TOC would recover so as to allow the reservoir to refill during the inflow hydrograph's recession.

4. The inflow volume threshold for triggering a drop in forecast-based TOC would be sufficiently large, such that the forecast-based releases would be triggered relatively infrequently, about once per 5 years on average, or less frequent.

Drawdown curves were used to specify variable TOC as a function of inflow forecast volume. The final drawdown curves are shown in Figure 5-3 and were the result of several iterations of testing and modification. Note that the method developed was adjusted to assure that goals 1 and 2, above, were achieved. Goals 3 and 4 were confirmed after testing of the forecast-based operation. In Figure 5-3 there are four curves, each corresponding to one of the four inflow forecast volume durations of 24, 48, 72, and 120 hours. When a forecast is issued, the four required inflow volumes are computed, and their values located on the X axis of the diagram. For each inflow volume and corresponding duration curve, a candidate TOC volume is located on the Y axis. The smallest of the four candidate TOC storage values is then adopted. In Figure 5-3, it can be seen that TOC will never drop below the maximum allowable variable storage as long as the 120-hour forecast volume is less than 300 KAF. Shorter duration volumes will always be less than longer duration volumes.

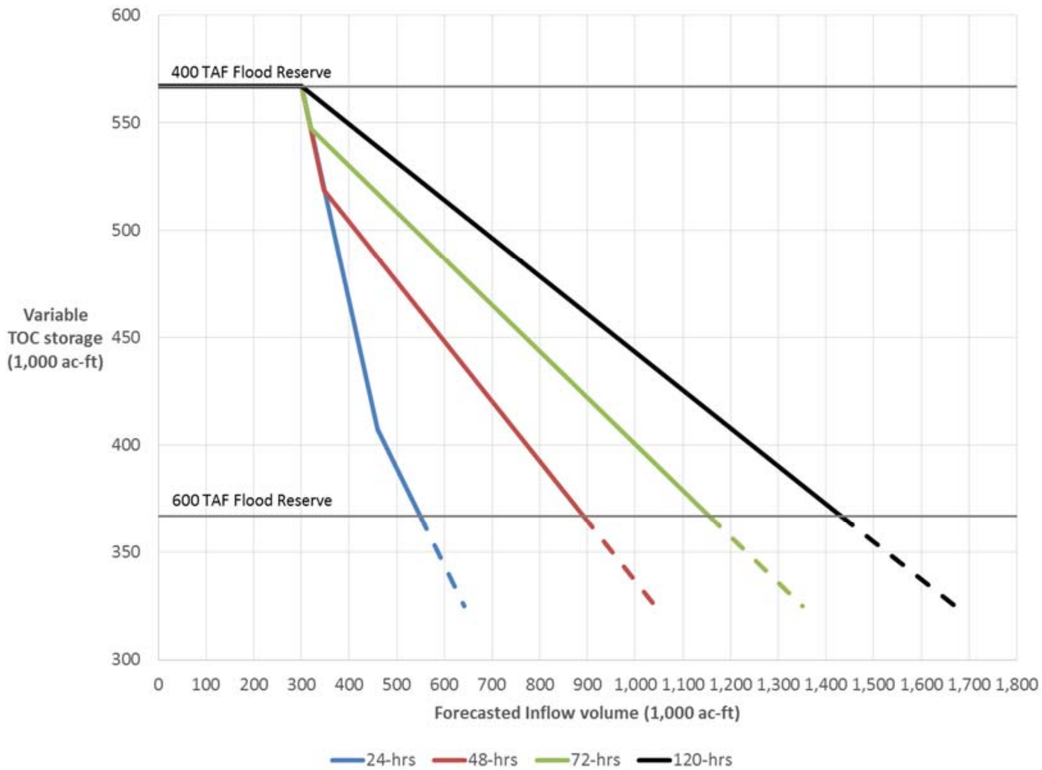


Figure 5-3: Drawdown Curves for Forecast-based Operation

5.4 Forecast-based Releases

Forecast-based releases are made when the TOC drops below the maximum TOC value shown on the Water Control Diagram, and the actual storage is above the TOC. In this condition, the storage is encroached into the flood space, and forecast-based flood releases are required. A rules-based release approach was sought that would support well-behaved releases made to evacuate flood space as required by the computed variable TOC. Should the reservoir be drawn down below the computed TOC, then the computed release is no longer required, allowing the operator to reduce releases to allow the reservoir to refill to TOC.

Table 5-1 lists the stepped releases identified to support downstream coordination needs and awareness of general downstream erosion concerns. Forecast-based thresholds were assigned to these releases. Once the 120-hour volume increases above 300 KAF, releases will be increased, subject to rate of increase restrictions, to 25 kcfs. The next release steps of 50 and 80 kcfs are triggered when the 72- hour and 48- hour volumes exceed 300 KAF, respectively. The next release step of 115 kcfs, the normal objective release, is the triggered when the 24-hour volume exceeds 300 KAF and the current inflow is at least 115 kcfs. Stepped releases of 25, 50, and 80 kcfs are maintained until the trigger for the next stepped release is satisfied. Releases above 115 kcfs are governed by the ESRD and are a function of current pool elevation and current inflow. Note that stepped releases are only made or maintained while storage is above the variable TOC. When this is not the case, the operator has latitude to hold or cutback releases, as the reservoir is no longer encroached into flood space.

Table 5-1: Forecast-based Releases

Forecast-based Trigger	Stepped Release
120-hour inflow volume > 300 KAF	25 kcfs
72- hour inflow volume > 300 KAF	50 kcfs
48- hour inflow volume > 300 KAF	80 kcfs
24- hour inflow volume > 300 KAF and current inflow >= 115 kcfs	115 kcfs

The concepts of forecast-based TOC and forecast-based drawdown are illustrated in Figure 5-4. The plot shows results of an HEC-ResSim routing of the 1/100 1986 pattern event, using a draft set of drawdown curves and perfect inflow forecast volume series (forecast volumes back-computed directly from the inflow series). The volume series are shown in the upper plot. They appear as dashes because they are hourly series of forecast volume values that are updated only once every 6 hours. The grey horizontal dashed line is 300 KAF volume, which is the volume threshold used to trigger the stepped release values of 25, 50, 80, and 115 kcfs, based on 120-, 72-, 48-, and 24-hour (labeled as “5-day”, “3-day”, “2-day”, and “1-day” in Figure 5-4) inflow forecast volume durations, respectively.

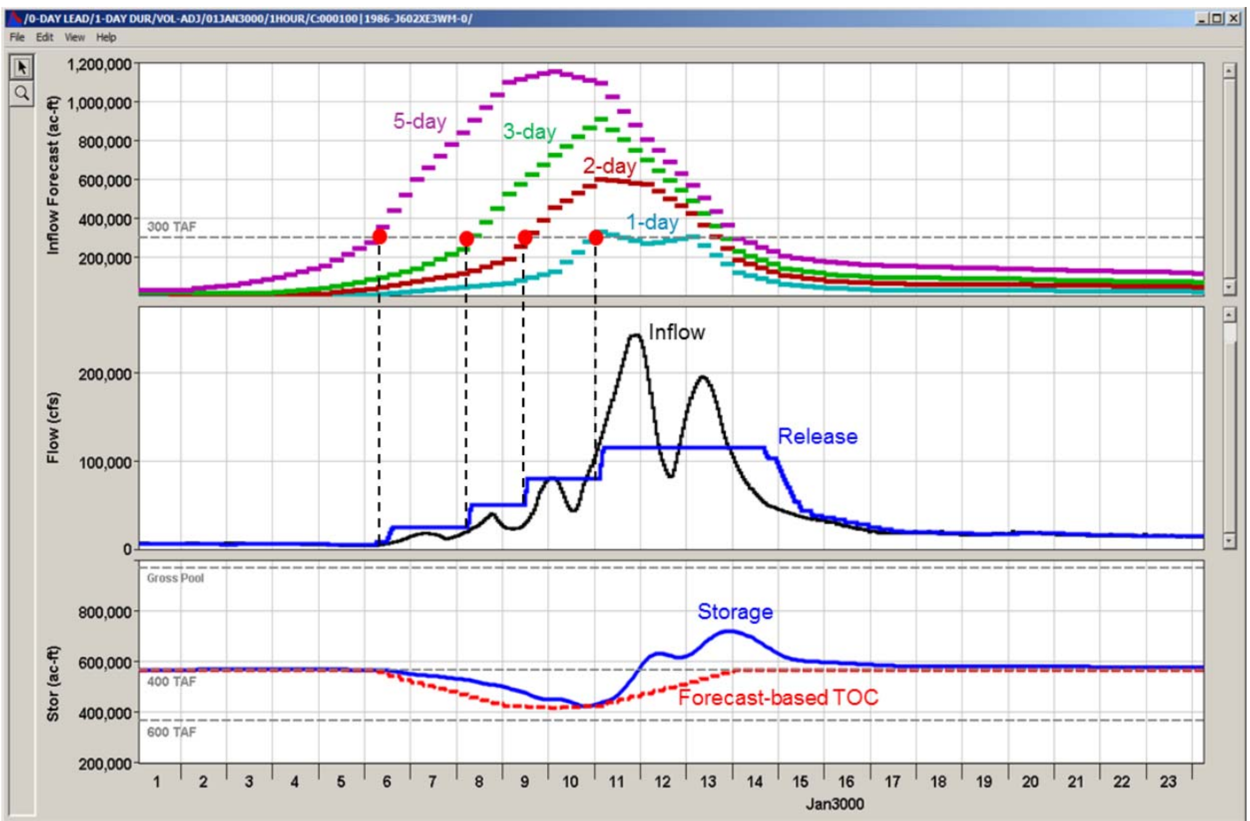


Figure 5-4: Forecast-based Drawdown Concept – WY 1986 pattern – 1/100 ACE

The red points in the upper plot and vertical dashed lines are used to indicate that the stepped releases in the middle plot correspond to each inflow volume series exceeding 300 KAF. The forecast-based stepped releases result in releasing more than inflow prior to the arrival of the main event. As a result, the reservoir is drawn down in the days preceding arrival of the greatest runoff intensity. This is seen in the lower plot. Had the reservoir not been drawn down in advance of the event, the peak storage and possibly the peak outflow would have been greater. The lower plot also illustrates that encroachment into the flood pool began when the forecast-based TOC dropped from its maximum value to a lower value on 6 January. When this occurred, the mode of reservoir operation changed from non-flood releases to forecast-based flood releases.

5.5 Sensitivity of Forecast-based Operation to Folsom Starting Storage

For the same event and operation as depicted in Figure 5-4, Figure 5-5 shows HEC-ResSim results for five different starting storage conditions at Folsom Lake. The forecast-based TOC is the same for all simulations, as it is a function of forecasted inflow volume only. The figure illustrates that lower starting storages at Folsom Lake result in later initiation of forecast-based releases. For the routings shown, the effects of starting stepped releases later due to lower starting storage are offsetting. The resulting peak pool elevations and releases are similar.

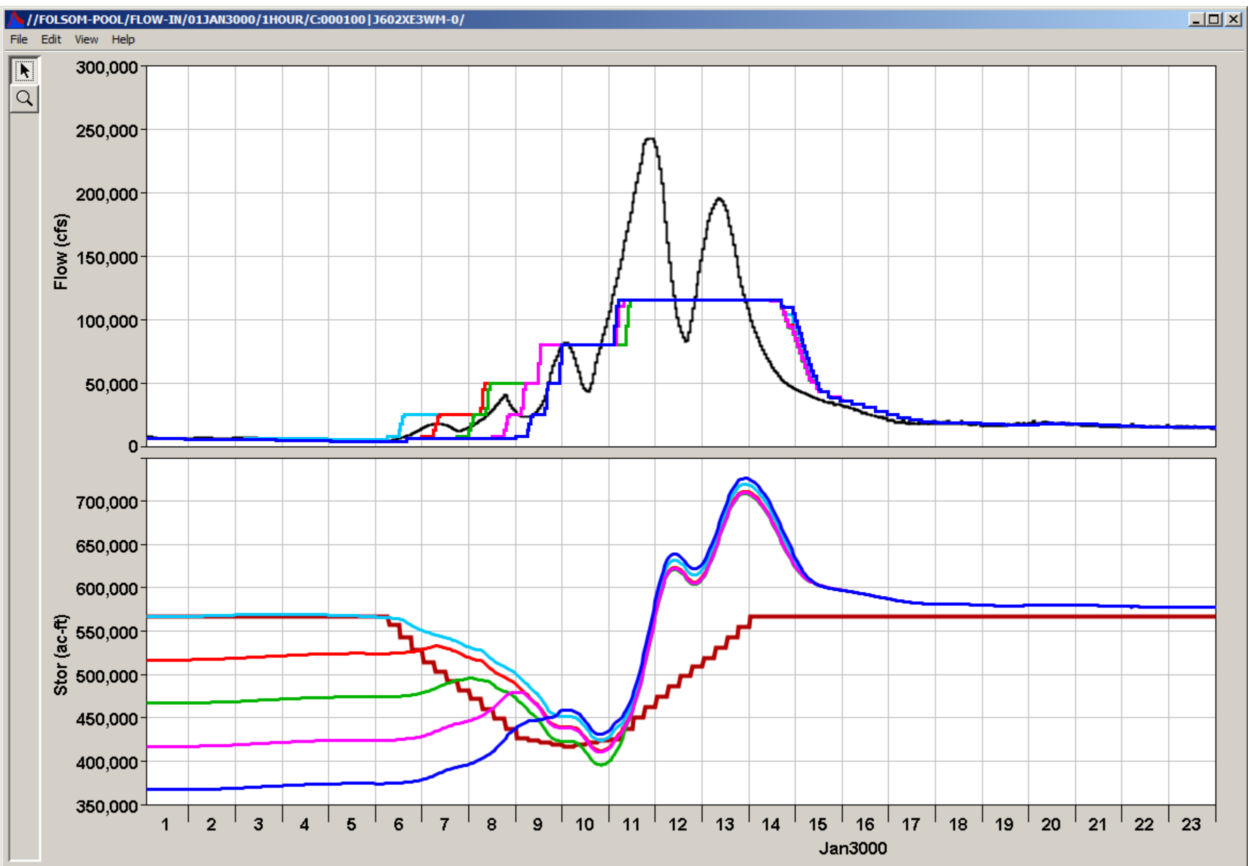


Figure 5-5: Forecast-based Drawdown Concept – Vary Folsom Starting Storage

Additional simulations for a range of event magnitudes were performed, again using perfect forecast inputs. Hydrograph plots from simulations of the WY 1986 and 1997 event patterns for 1/2, 1/5, 1/10, 1/50, 1/100, 1/200, 1/250, 1/300, and 1/500 ACE scaled winter events are shown in Figures G-17 through G-36. In these simulations, a range of starting conditions at Folsom were considered, varying in 50 KAF increments from 567 KAF storage (400 KAF flood space) to 317 KAF (650 KAF flood space) storage. These figures illustrate the potential to maintain the downstream flood protection by drawing down the reservoir prior to the arrival of a major event. ACE=1/100 and ACE=1/200 peak releases were held to 115 kcfs and 160 kcfs, respectively, for all Folsom starting storage values, for both WY 1986 and 1997 event patterns.

5.6 Consideration of Folsom Ending Storage

5.6.1 Short-Term Potential for Not Refilling

It is possible that the forecast-based operation will draft down the reservoir and that the reservoir does not refill to the pre-event storage level. While this is not problematic from the perspective of FRM performance, it is undesirable from a water management and supply perspective. Two factors contribute to the likelihood of not refilling to the pre-event storage level: 1) over-forecast of inflow volume, and 2) over-drafting to maintain well-behaved release.

As long as the actual inflow volume to the reservoir is not less than (to the left of) the 24-hour drawdown curve in Figure 5-3, the inflow volume will be sufficient to fill the reservoir (with zero releases) to at least 867 KAF storage (100 KAF below the top of flood pool). In other words, if the inflow forecast is “good,” then releases will be required to satisfy the drawdown requirement, with sufficient inflow remaining to refill to 567 KAF storage (400 KAF flood space). If the pre-event starting storage is less than 567 KAF (400 KAF flood space), the end of event storage can be as high as 567 KAF, which would be a net increase in storage. If the reservoir is drawn down to the forecast-based TOC and not further, a substantial over-forecast of inflow volume would be necessary to result in not refilling to the 567 KAF storage level.

Drawing down the reservoir to the forecast-based TOC is a required flood operation while the flood pool is encroached. Over-drafting the reservoir may occur as a result of getting to TOC but being limited by rate of decrease constraints, or the desire to maintain a constant release, to avoid over drafting. Making releases during forecast-based operations when storage only marginally greater than TOC can become a balance between getting to TOC in a timely fashion, but not so rapidly as to result in a significant over-draft. Dynamics of the operation are sensitive to the current inflow and inflow volume that enters the reservoir before the next forecast is issued. If the reservoir has been drawn down below the forecast-based TOC, the operator has latitude to cut back releases to minimize over-drafting of the reservoir. This operation requires assessment of the situation by the operator, and is not reflected in the HEC-ResSim model simulations in this report.

Answering the question of how likely it is that short term (end-of-event) refill does not occur is challenging because:

1. The supporting analysis should be based on the hindcast period of record (1986 to 2010), as that record can be reviewed for over-forecasted events. However, the hindcast period of record is limited to 24 years of data.

2. The likelihoods of drawdown being required, and then not refilling, are dependent on event starting storage. The event starting storage is a function of inflow to that point in time, and conservation and flood operations to that point in time. A simulation of the hindcast period of record should therefore incorporate realistic (to the extent possible) flood and conservation operations. As previously mentioned, the simulated operation will not reflect cutting back of releases to prevent over-drafting the reservoir below the TOC.

3. Forecasts are updated every six hours. Therefore, changes in runoff conditions would be detected quickly.

In order to provide some assessment of the likelihood of short term non-refill, the CalSim period of record (1986 to 2002) was simulated using an HEC-ResSim model of the forecast-based operation. 75 percent NEP hindcast inflow volumes (described in Section 6.5.4) were used to compute TOC. In the model, CalSim II end of month storage targets were used to influence conservation pool operations once all other conservation rules had been satisfied. The simulated storage and TOC values were inspected over the 17 year period of record. The TOC was found to have dropped from its maximum value (567 KAF storage or 400 KAF flood space) for five events. Of those five events, two resulted in a negligible drawdown. The January 1995 event resulted in a small amount of drawdown, and the WY 1986 and 1997 events resulted in significant drawdown. All of these events refilled to the maximum allowable storage (400 KAF flood space). To summarize, no occurrences of short-term non-refill were found in the 17-year hindcast period of record.

As an update to the previous paragraph, three recent events occurred during WY 2017 that were sufficiently large to have triggered forecast-based releases had the operation been in effect. Hydrograph plots of six events, including the three WY 2017 events, are described and provided in 6.5.5. In all six events the reservoir refills to the pre-event starting storage.

Another effort to better understand the susceptibility of the forecast-based operation to not refilling in the short term was undertaken in the form of a sensitivity analysis. Using an earlier version of the HEC-ResSim model, 1/n ACE inflow hydrographs for a specific event pattern were coupled with forecast volumes for the same pattern but different 1/n ACE volumes. Simulations of combinations of inflow and perfect forecast volumes were made, and the resulting end of event storages recorded. In these model simulations, operational rules in the conservation pool were removed such that the model would not drawdown below the TOC if inflows were sufficient. Table 5-2 and Table 5-3 contain results of these simulations, providing end of pool elevations. In all simulations, the starting pool elevation was 428.00 feet NGVD29, which corresponds to 400 KAF flood space. The tables show that a substantial under-forecast is required to cause the reservoir to not refill at the end of an event.

Table 5-2: WY 1986 Pattern Inflow-Forecast Combinations, End of Event Pool Elevation (feet, NAVD88)

		Inflow 1/ACE					
		2	10	50	100	200	500
Perfect Forecast 1/ACE	5	428.00	428.00	428.00	428.00	428.00	428.00
	10	428.00	428.00	428.00	428.00	428.00	428.00
	20	428.00	428.00	428.00	428.00	428.00	428.00
	50	428.00	428.00	428.00	428.00	428.00	428.00
	100	428.00	428.00	428.00	428.00	428.00	428.00
	130	426.79	428.00	428.00	428.00	428.00	428.00
	200	422.76	428.00	428.00	428.00	428.00	428.00
	300	418.98	428.00	428.00	428.00	428.00	428.00
	500	414.96	428.00	428.00	428.00	428.00	428.00

Further analysis would be required to determine the probability of not refilling after an event, as this would require developing relationships between the spread of ensemble inflow hydrographs and the perfect forecast 1/ACE values shown in Table 5-2 and Table 5-3.

Table 5-3: 1997 Pattern Inflow-Forecast Combinations, End of Event Pool Elevation (feet, NAVD88)

		Inflow 1/ACE					
		2	10	50	100	200	500
Perfect Forecast 1/ACE	5	428.00	428.00	428.00	428.00	428.00	428.00
	10	428.00	428.00	428.00	428.00	428.00	428.00
	20	428.00	428.00	428.00	428.00	428.00	428.00
	50	428.00	428.00	428.00	428.00	428.00	428.00
	100	424.48	428.00	428.00	428.00	428.00	428.00
	130	420.46	428.00	428.00	428.00	428.00	428.00
	200	416.71	428.00	428.00	428.00	428.00	428.00
	300	414.93	428.00	428.00	428.00	428.00	428.00
	500	413.16	425.59	428.00	428.00	428.00	428.00

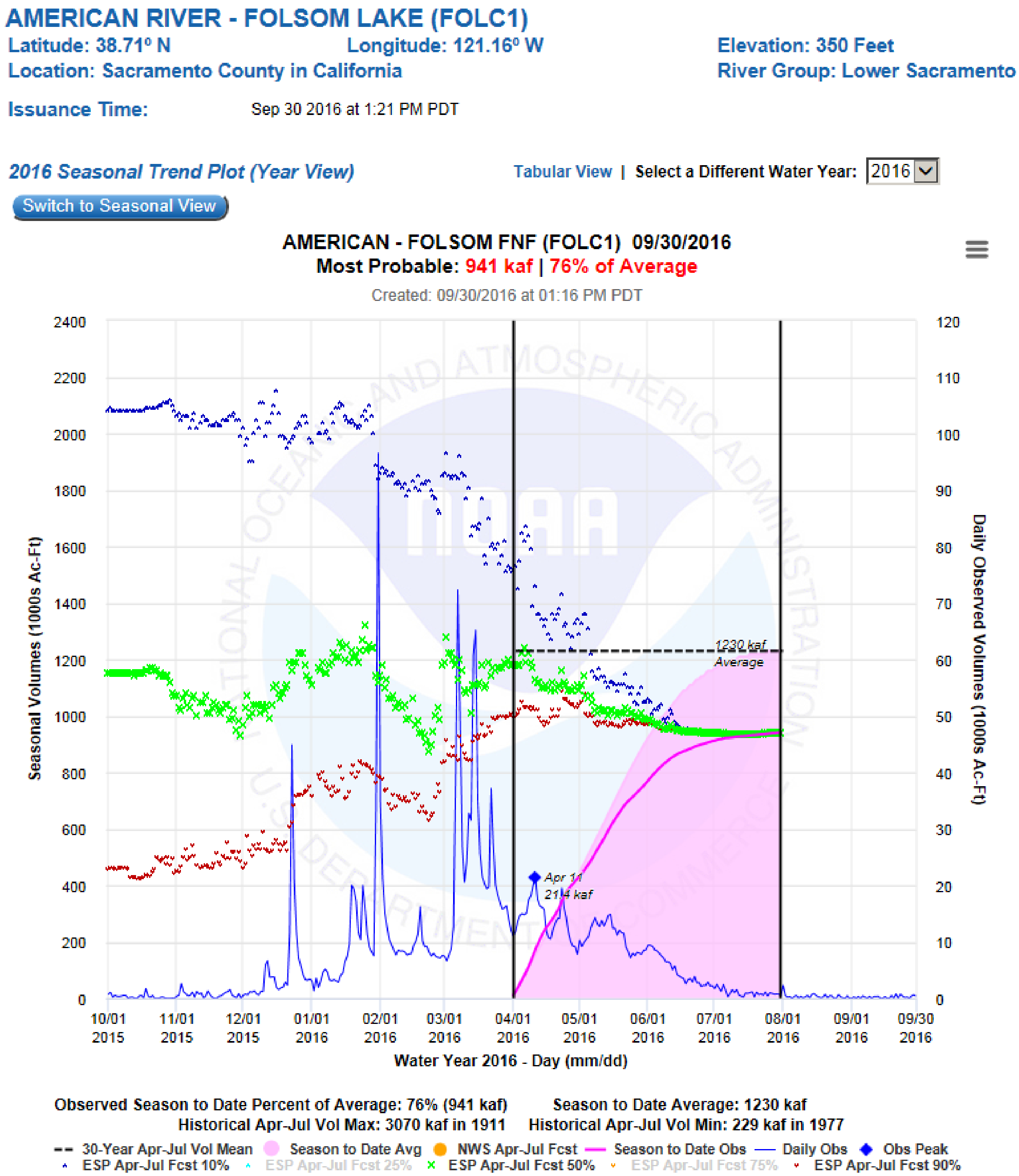
5.6.2 Long-Term Potential for Not Refilling

Should the forecast-based operation draw down the reservoir as required by the drawdown curves shown in Figure 5-3, and the reservoir does not achieve short term refill as discussed in the previous section, then there exists the potential that the end of season storage may be less as a result. While this scenario is not desirable from a water supply perspective, it is a possibility if the target level of downstream flood protection is to be maintained through the end of February, the end of variable TOC operations. It is worth noting, however, that it is the forecast-based operation that, given no major event is in the forecast, will allow storage at the top of the variable flood pool in the end of February, even if the watershed is saturated and upstream reservoirs are full.

Folsom Dam is part of the Central Valley Project (CVP) and is operated by Reclamation. As part of operating the CVP, Reclamation manages the reservoir to satisfy agricultural, downstream, and local delivery obligations. Reclamation must also manage the CVP to manage in-stream flow requirements, delta salinity, and in-stream temperature requirements. Throughout the winter season, seasonal cumulative runoff is an important part of planning operations in the conservation pool. This information is generated by the California Department of Water Resources (DWR) and is issued in the monthly (starting in February of each year) Bulletin 120, which includes probabilistic estimates of seasonal runoff based primarily on Sierra snowpack surveys. CNRFC generates a similar product also used by Reclamation. An example web page displaying this information for WY 2016 is shown in Figure 5-6. The blue, green, and red series are the 10, 50, and 90 percent chance exceedence forecast values of cumulative runoff for the period 1 April to 31 July. These values are issued daily, and as the season progresses, the three values converge toward the actual value on July 31. While this information does not change the flood operation rules, it contributes to the management of water in the conservation pool. In wet years, this information can be used by Reclamation to keep the spring pool lower than required by the season refill curve, to provide additional downstream flood protection, because it is known that the coming snowpack runoff will be sufficient to fill the reservoir.

5.7 Forecast-based Operation Water Control Diagram

The proposed Water Control Diagram for the forecast-based operation is shown in Figure 5-7.



[http://www.cnrfc.noaa.gov/ensembleProduct.php?id=FOLC1&prodID=7&year=2016&briefing=0]

Figure 5-6: CNRFC Seasonal Runoff Forecast Example Plot

- DRAFT -

FIGURE A - FLOOD CONTROL DIAGRAM

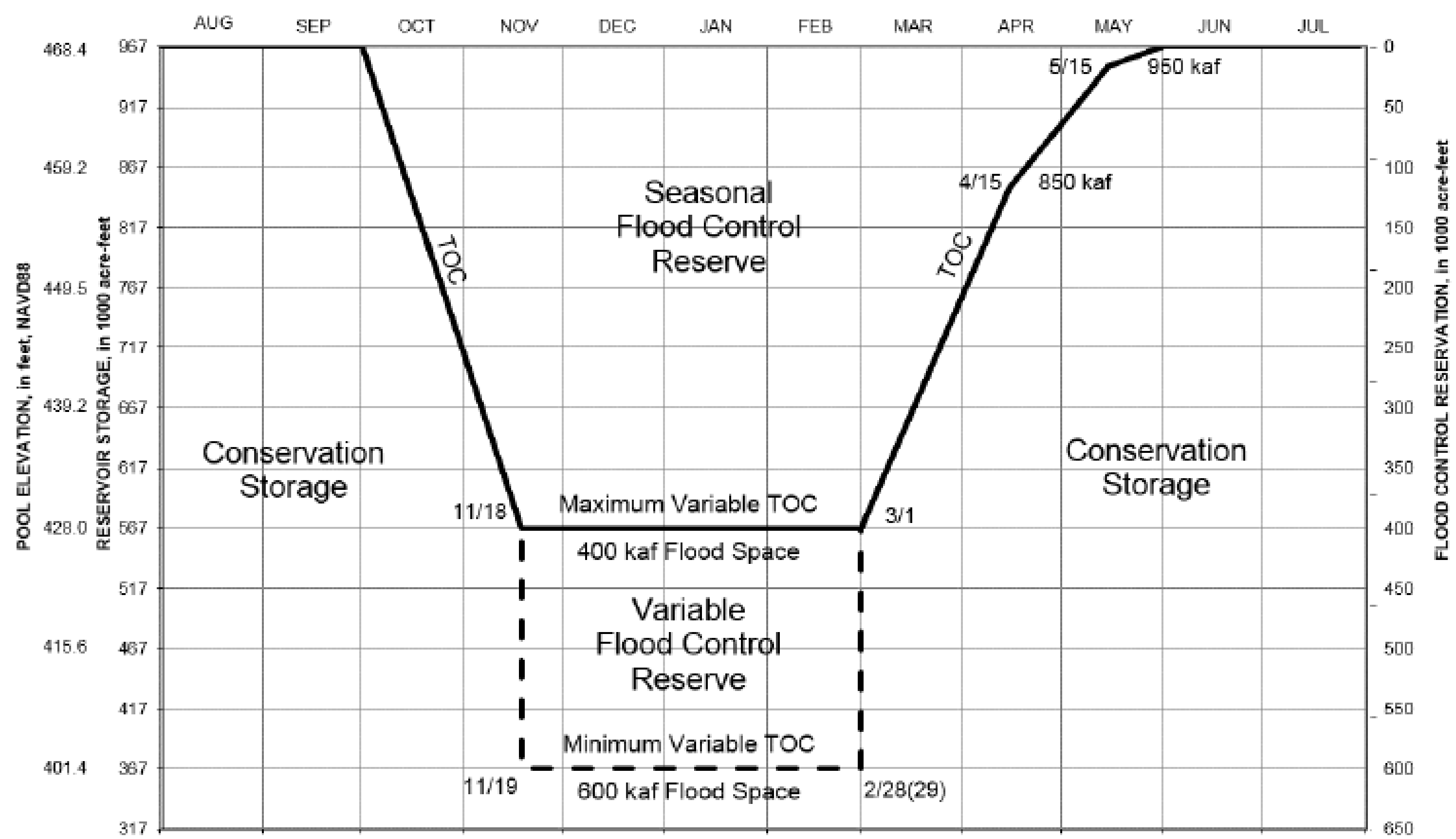
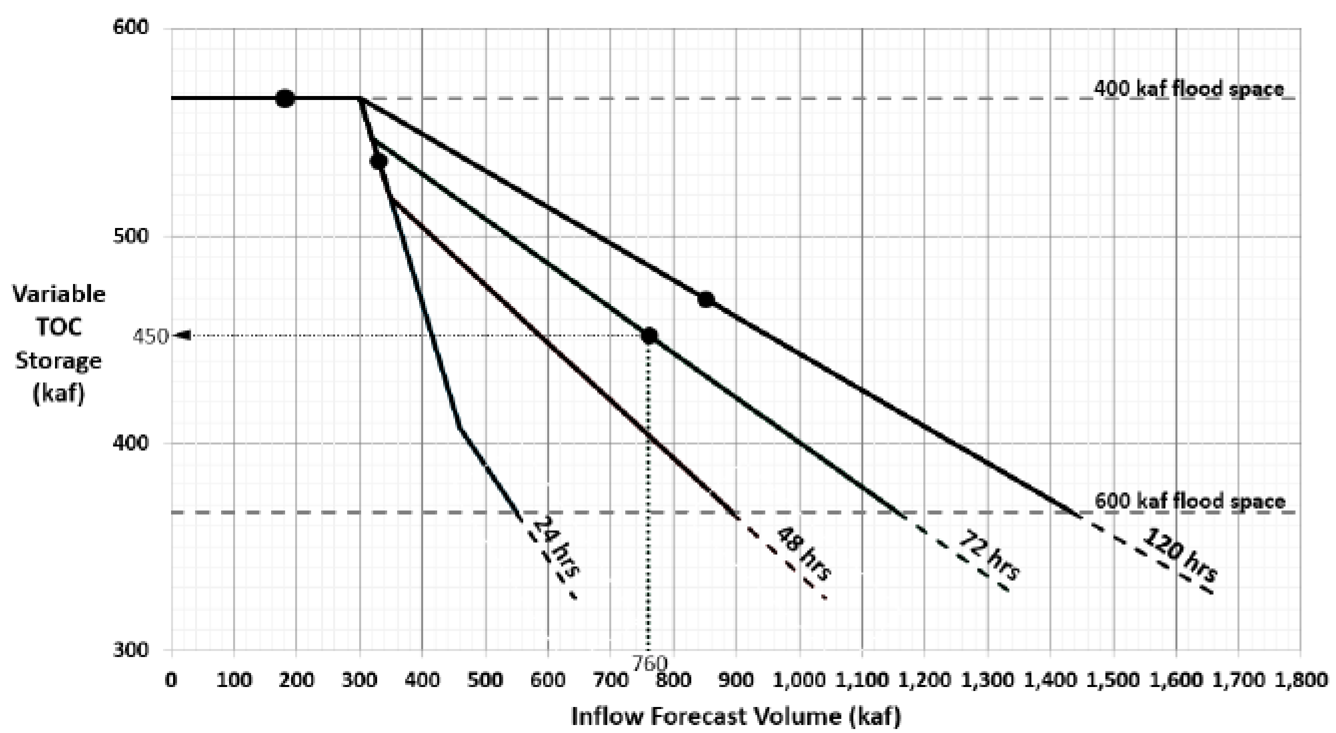


FIGURE B - VARIABLE TOP OF CONSERVATION



USE OF FLOOD CONTROL DIAGRAM (FIGURE A)

Folsom Dam and Lake shall be operated for flood control in accordance with the Flood Control Diagram and the accompanying Emergency Spillway Release Diagram (ESRD). Water stored within Flood Control Reserve (FCR) space shall be released as rapidly as possible subject to the Release Schedule (Table A), except when releases greater than 115 kcfs are required by the ESRD. The Corps of Engineers may direct flood releases to be increased or descreased from the prescribed release when warranted by existing conditions or by high confidence forecast information provided by NWS-CNRFC.

COMPUTATION OF VARIABLE TOP OF CONSERVATION (FIGURE B)

From 18 Nov to 28/29 Feb the Top of Conservation (TOC) storage will vary based on forecasted inflow volumes. These are developed by the NWS-CNRFC for the purpose of supporting Folsom Dam flood operations, will reflect forecasted inflows over the next 24, 48, 72, and 120 hours, and will reflect a value of non-exceedance probability (NEP) specified by the Corps. Volumes will be provided once per day during normal operations, and once per six hours once the 120-hour volume exceeds 300 kaf. Figure B provides relationships relating inflow forecast volume to variable TOC storage for each duration.

FIGURE B - INSTRUCTIONS: Locate each of the four forecast volumes on the horizontal axis. Place the four forecast volumes on the respective duration curves. For each forecast volume, identify the corresponding candidate TOC storage value on the vertical axis. Of the four candidate TOC storage values, the lowest value is the adopted variable TOC storage value. The corresponding FCR value is given by: FCR = 966.9 kaf - variable TOC storage.

FIGURE B - EXAMPLE: Inflow forecast volumes of 180, 330, 760 and 850 kaf are provided, corresponding to 24, 48, 72, and 120 hours respectively. As shown in Figure B, the volumes are located on the horizontal axis, and placed on the corresponding curves (indicated by large dots). Corresponding candidate TOC storage values are read from the vertical axis. The lowest value is given by the 72-hour volume. This value (450 kaf) is therefore the adopted variable TOC storage value. The corresponding FCR value is: FCR = 966.9 kaf - 450 kaf = 546.9 kaf.

TABLE A - FOLSOM RELEASE SCHEDULE

Date	Storage Condition	Description
Mar. 1 to Nov. 18	Storage > TOC	EVACUATE SEASONAL FLOOD CONTROL RESERVE Release peak inflow for current event.
Nov. 19 to Feb. 28/29	Storage > TOC	EVACUATE VARIABLE FLOOD CONTROL RESERVE Release greater of peak inflow for current event or Table B
All year	Storage < TOC	Non-flood operations.

RAMPING RATES

Releases between 8 kcfs and 30 kcfs will not be increased by more than 10 kcfs during any 2-hour period.
Releases between 30 kcfs and 115 kcfs will not be increased by more than 30 kcfs during any 2-hour period.
Releases between 8 kcfs and 115 kcfs will not be *decreased* by more than 10 kcfs during any 2-hour period.

TABLE B - FORECAST-BASED RELEASES

INFLOW FORECAST VOLUMES	RELEASE
120-hr volume < 300 kaf	8 kcfs
120-hr volume > 300 kaf	25 kcfs
72-hr volume > 300 kaf	50 kcfs
48-hr volume > 300 kaf	80 kcfs
24-hr volume > 300 kaf and inflow >= 115 kcfs	115 kcfs

1 kcfs = 1,000 cfs, 1 kaf = 1,000 acre-feet

FOLSOM DAM AND LAKE
American River, California

WATER CONTROL DIAGRAM

APPROVED ______________________

APPROVED ______________________

Effective Date ____________ File No. ____________

Figure 5-7: Forecast-based Operation Water Control Diagram

6 Reservoir Models and Simulation Results

6.1 Reservoir Models

Multiple HEC-ResSim models were developed to simulate flood operations at Folsom Dam. All models were configured to operate on a 1-hour time step, and all models included Folsom Dam and Lake and five upstream reservoirs. Elevation data in the models, and their output, are in feet NGVD29. All storage data in the models, and their output, reflect elevation-capacity from the Reclamation Technical Service Center (TSC) September 2005 Folsom lake survey.

There were two groups of models:

1. Models (Table 6-1 and Table 6-3) configured to route scaled synthetic events and PMF event for the purpose of evaluating FRM and dam safety performance. These models used HEC-ResSim version 3.2.148. Performance results from these model simulations are presented in this chapter.

2. Models (Table 6-4) configured to route the 81-year POR (WY 1922-2002) to support analyses of downstream effects and effects on other project purposes. These models used HEC-ResSim version 3.2.54 with zone boundary logic disabled. Effects analyses and results are documented in Chapter 8.

The use of two versions of HEC-ResSim within the study was not by design, but is reflective of the timelines in which the FRM and dam safety and the CalSim-consistent models evolved and challenges in coordinating the two model efforts. Comparisons of FRM and dam safety performance were made on a consistent model version basis, as were comparisons of effects.

In the “Events Simulated” column of the following tables, scaled synthetic events and PMF events are followed by “winter only” or “winter and seasonal” in parentheses. Winter events are those scaled based on the winter unregulated flow-frequency curve. Winter events were used to test performance of operations during the period of variable TOC. Seasonal events were scaled based on seasonal unregulated flow-frequency curves. Seasonal events were used to test operations outside the period of variable TOC (fall drawdown and spring refill). Unregulated frequency curves used for event scaling are described in Section 7-2.

Table 6-1: Reservoir Models for FRM and Dam Safety – Existing Conditions

Flood Operation	Model ID	Model Date	Events Simulated
Existing Corps	E503	09-25-2015	PMF (winter only)
Existing Corps	E503P	11-24-2015	Scaled synthetic (winter only),
Existing Interim	E504	12-07-2016	Scaled synthetic (winter only), PMF (winter only)

Note: E503P includes spring basin wetness functionality, E503 does not.

FRM and dam safety models of existing condition operations are listed in Table 6-1. FRM and dam safety models of Alternatives 1 and 2 were developed later in the study. Due to the computational overhead of routing ensemble hindcast datasets required for Alternative 2, and the desire to not repeat the upstream routing of these datasets every time an adjustment was made to the operation, an upstream-downstream model strategy was adopted. With this approach, a set of models were configured to perform routing of upstream inflow hydrographs from their input locations, through the headwater reservoirs and routing reaches, to Folsom Lake. These models are listed in Table 6-2. In these models, operational rules were removed from Folsom Dam to decrease simulation times. Models J602X and J602N2 were used to generated impaired (reflecting upstream regulation) inflow hydrographs needed for evaluation of Alternatives 1 and 2. Model J602N is listed for reference, and was used to generate unimpaired inflow hydrographs at Folsom, which were used to check and confirm proper configuration of upstream boundaries.

Table 6-2: Reservoir Models for Upstream Routing

Configuration	Model ID	Model Date	Events Simulated
With upstream storage	J602X	10-23-2015	WY 1986-2002 Hindcast POR for *impaired* hindcast Folsom inflow
No upstream storage	J602N	02-22-2016	Scaled synthetic (winter and seasonal) for *unimpaired* Folsom inflow
With upstream storage	J602N2	02-22-2016	Scaled synthetic (winter and seasonal) for *impaired* Folsom inflow

Table 6-3 lists models used to evaluate Alternatives 1 and 2. In these models, Folsom inflow hydrographs, previously computed using models J602X and J602N2, were read externally. Upstream routing reaches and reservoirs remain in these models as placeholders, having inflow and releases set to 0 cfs.

Table 6-3: Reservoir Models for FRM and Dam Safety – Alternatives 1 and 2

Flood Operation	Model ID	Model Date	Events Simulated
Alternative 1	J602P3	05-01-2017	Scaled synthetic (winter and seasonal), PMF (winter and seasonal)
Alternative 2	J602F3	05-01-2017	[Perfect forecast-based simulations] Scaled synthetic (winter and seasonal), PMF (winter and seasonal)
			[Hindcast-based simulations] for robustness testing of Pattern-specific ACE=1/100 & 1/200, 1986 & 1997 winter events

Models supporting analyses of downstream effects and effects on other project purposes are listed in Table 6-4. POR inflows to these models were scaled on a monthly basis to be consistent with CalSim monthly inflows. These models also included CalSim-consistent end of month storage targets. Storage targets reflected either existing level of demand (ELD) or future level of demand (FLD) depending on analysis requirement.

Table 6-4: Reservoir Models for Effects Analyses

Flood Operation	Model ID	Model Date	Events simulated
Existing Corps	E503P	08-10-2016	WY 1922-2002 POR (CalSim-ELD)
Existing Interim	E504	07-29-2016	WY 1922-2002 POR (CalSim-ELD)
Alternative 1	J602P3	10-13-2016	WY 1922-2002 (CalSim-ELD)
Alternative 2	J602F3	09-13-2016	WY 1922-2002 (CalSim-ELD and FLD)

6.2 Physical Constraints

Maximum release capacity resulting from limitations to gate openings to maintain controlled flow are described in section 3.3.6 and are reflected by the right-bounding curve of the ESRD. Constraints on how long the main and emergency gates may be kept shut before freeboard to the top of the gates is encroached are described in sections 3.3.3 and 3.3.4. These make up the upper-left bounding curve of the ESRD and include 2 feet freeboard on the five service gates and 1 foot freeboard on the three emergency gates. Constraints reflected on the ESRD are reflected in all simulated event routings, as the ESRD is part of the specified operation in the HEC-ResSim models.

6.3 Operational Constraints

HEC-ResSim models are a tool for simulating the required total release sequence through time. In other words, if the model determines that adequate capacity exists to make the release, subject to the specified flood release rules, it will make that release. Important operational details, such as how the release is distributed among gates and spillways are not considered, nor whether challenges exist in transitioning from one gate configuration to another as the pool rises. The Corps relies on the operating agency, Reclamation, to identify operational challenges which cannot be reflected in the reservoir routing models.

Constraints specified on the WCD and ESRD must be adhered to operationally and are reflected in reservoir operations models. These take the form of maximum allowable release rate of change values in the existing and alternative operations. Table 6-5 and Table 6-6 list maximum allowable release rates of change for both existing condition operations. Table 6-7 and Table 6-8 list maximum allowable rates of release change for Alternatives 1 and 2.

Table 6-5: Maximum Rates of Release Increase – Existing Operations

Pertinent Diagram	Release Range	Will not be *increased* by more than this amount
WCD	up to 115 kcfs	15 kcfs per 2 hours
ESRD	115 kcfs to 160 kcfs	15 kcfs per 2 hours

Table 6-6: Maximum Rate of Release Decrease – Existing Operations

Pertinent Diagram	Release Range	Will not be *decreased* by more than this amount
WCD	up to 115 kcfs	10 kcfs per 2 hours

Table 6-7: Maximum Rates of Release Increase – Alternatives 1 and 2

Pertinent Diagram	Release Range	Will not be *increased* by more than this amount
WCD	8 kcfs to 30 kcfs	5 kcfs per 2 hours
WCD	30 kcfs to 160 kcfs	30 kcfs per 2 hours
ESRD	160 kcfs to 360 kcfs	100 kcfs per hour
ESRD	360 kcfs and greater	200 kcfs per hour

Table 6-8: Maximum Rate of Release Decrease – Alternatives 1 and 2

Pertinent Diagram	Release Range	Will not be *decreased* by more than this amount
WCD	8 kcfs to 160 kcfs	10 kcfs over any 2-hour period

Model simulations of Alternatives 1 and 2 include delays in implementing stepped releases. These delays are not part of the flood operation, but are included to reflect a more realistic operation in which delays may occur to support downstream coordination efforts. Delays of 18, 6, 3, and 3 hours were enforced before ramping up to stepped releases of 25, 50, 80, and 115 kcfs respectively. Based on coordination with Reclamation, a 6-hour delay at 8 kcfs release and a 12-hour delay (avoid increasing releases at night time) before ramping up to 25 kcfs can occur to support efforts to evacuate and secure the downstream flood channel. In the model, these two delays are reflected as a single 18-hour delay before ramping up to 25 kcfs.

Even with the above operational constraints reflected in the reservoir routing models, simulations reflect no hardware failures (non-operational gates for example), and effectively reflect hourly gate changes as needed to maintain the specified release. The models use current values of elevation and inflow as input to the ESRD to compute dam safety releases. In actual operations, current elevation may be known, but inflow must be estimated from change in storage and release over time, which can result in an underestimate of inflow.

For both existing and both alternative operations, scaled synthetic event and period of record simulations allowed releases to transition from lower outlets to the spillways as a function of pool elevation. Releases were not allowed through the river outlets and the power house during PMF event simulations.

6.4 Scaled Event Routings

This section documents simulations of event patterns, scaled based on unregulated runoff volume, to obtain events having ACE values of interest. Events having ACE values ranging from ½ to 1/100 were simulated using HEC-ResSim. In simulations of Alternative 2, inflow forecast volumes required for the operation were developed by computing directly from the inflow time series.

Starting storage values for Folsom Lake are listed in Table 6-9. Starting storage values for the five modeled headwater reservoirs are listed in Table 6-10. HEC-ResSim simulation start dates and times are listed in parentheses in the column headers. For all events, Folsom Lake was started at TOC. For winter events, the TOC is variable for the Existing Interim operation and Alternatives 1 and 2. Winter event starting storage/TOC at Folsom Lake reflects headwater reservoir storage for the Existing Interim operation and headwater storage and basin wetness for the Alternative 1 operation. Starting storage at Folsom for seasonal events is the seasonally varying TOC for the indicated date. The storm mass of each seasonal event was placed to occur at the start of the indicated month for consistency with effective dates of the seasonal unregulated flow frequency curves used to scale those events. Seasonal events were not simulated for the Existing Corps and Existing Interim operations. Headwater starting storage values were most likely values computed from the historical period of record as having 80 percent chance non-exceedence probability (NEP) for the indicated simulation start date. The 80 percent value was considered representative of the average starting historical storage conditions for the WY 1986 and 1997 events.

Table 6-9: Scaled Event Simulations - Starting TOC and Storage at Folsom Lake

Operation	Folsom Lake Starting TOC and Storage (ac-ft)			
	Winter (03 Jan 02:00)	March (22 Feb 02:00)	April (23 Mar 02:00)	May (22 Apr 02:00)
Existing Corps	566,934	not simulated	not simulated	not simulated
Existing Interim	507,035	not simulated	not simulated	not simulated
Alternative 1	498,286	566,934	705,846	873,611
Alternative 2	566,934	566,934	705,846	873,611

Table 6-10: Scaled Event Simulations – Starting Storage at Headwater Reservoirs

Reservoir	Headwater Starting Storage (ac-ft)			
	Winter (03 Jan 02:00)	March (22 Feb 02:00)	April (23 Mar 02:00)	May (22 Apr 02:00)
Union Valley	186,240	201,990	213,990	238,350
French Meadow	76,000	87,320	93,500	110,340
Hell Hole	137,880	146,750	153,950	176,840
Ice House	28,235	28,839	30,263	36,121
Loon Lake	46,100	44,800	44,241	54,591

Figure 6-1 through Figure 6-4 display simulation results of the Alternative 1 and Alternative 2 operations against ACE=1/100 and ACE=1/200 winter events. Simulation results from the Existing Interim operation are also shown for comparison. Alternatives 1 and 2 both route the ACE=1/100 event at 115 kcfs, with Alternative 1 operation using about 100 KAF more flood space than Alternative 2. Both alternatives route the ACE=1/200 event at less than 160 kcfs, with Alternative 2 routing at a lower peak release for both event patterns. The figures also illustrate TOC changing ahead of and during the event. Both the Existing Interim and Alternative 1 operations have starting storage lower than Alternative 2, because these two operations are dependent on sufficient space existing in upstream credit reservoirs in order to allow Folsom to store up to the 400 KAF flood space level. All three operations in all four events result in end-of-event storage at TOC for the specific operation. The highest end-of-event TOC is given by Alternative 2, the forecast-based operation, which allows storage at 400 KAF flood space when the 120-hour inflow forecast volume no longer exceeds 300 KAF.

Table 6-11 and Table 6-12 summarize results from the ACE=1/100 and ACE=1/200 events for winter and spring simulations. Existing Interim, Alternative 1, and Alternative 2 operations are included in these tables. Simulated event patterns include the historical events of WY 1956, 1964, 1986, and 1997, and the SPF and PMF event temporal distributions. The SPF and PMF patterns were used to further test the operations, but were not used to define regulated peak flow-frequency curves in Section 6.6. Even though the majority of attention in the report has been focused on the winter operation, in which variable flood space is in effect for the Alternatives 1 and 2, it is important to recognize that outside this period (19 November through 28/29 February) that the "seasonal guide curve" operation is in effect. This refers to evacuating the flood pool as rapidly as possible without violating rate of increase restrictions. Neither credit-based nor forecast-based operations are in effect during this part of year. As a result, Alternatives 1 and 2 produce identical results (when using the same starting storage) for fall drawdown and spring refill operations, as they share the same seasonally varying guide curve.

Table 6-13 through Table 6-16 summarize results for Existing Interim, Alternative 1, and Alternative 2 operations for synthetic winter events for the four historical event patterns of WY 1956, 1964, 1986, and 1997. There is one table for each pattern. Each table provides results for conditional ACE values ranging from 1/2 to 1/1000. The term "conditional" indicates that the probabilities are valid given the condition that the indicated event has occurred. The relative likelihood of a specific event pattern occurring is not reflected in the tabulated probabilities. All ACE events in each table were simulated using the same starting storage configuration indicated in Table 6-9 and Table 6-10.

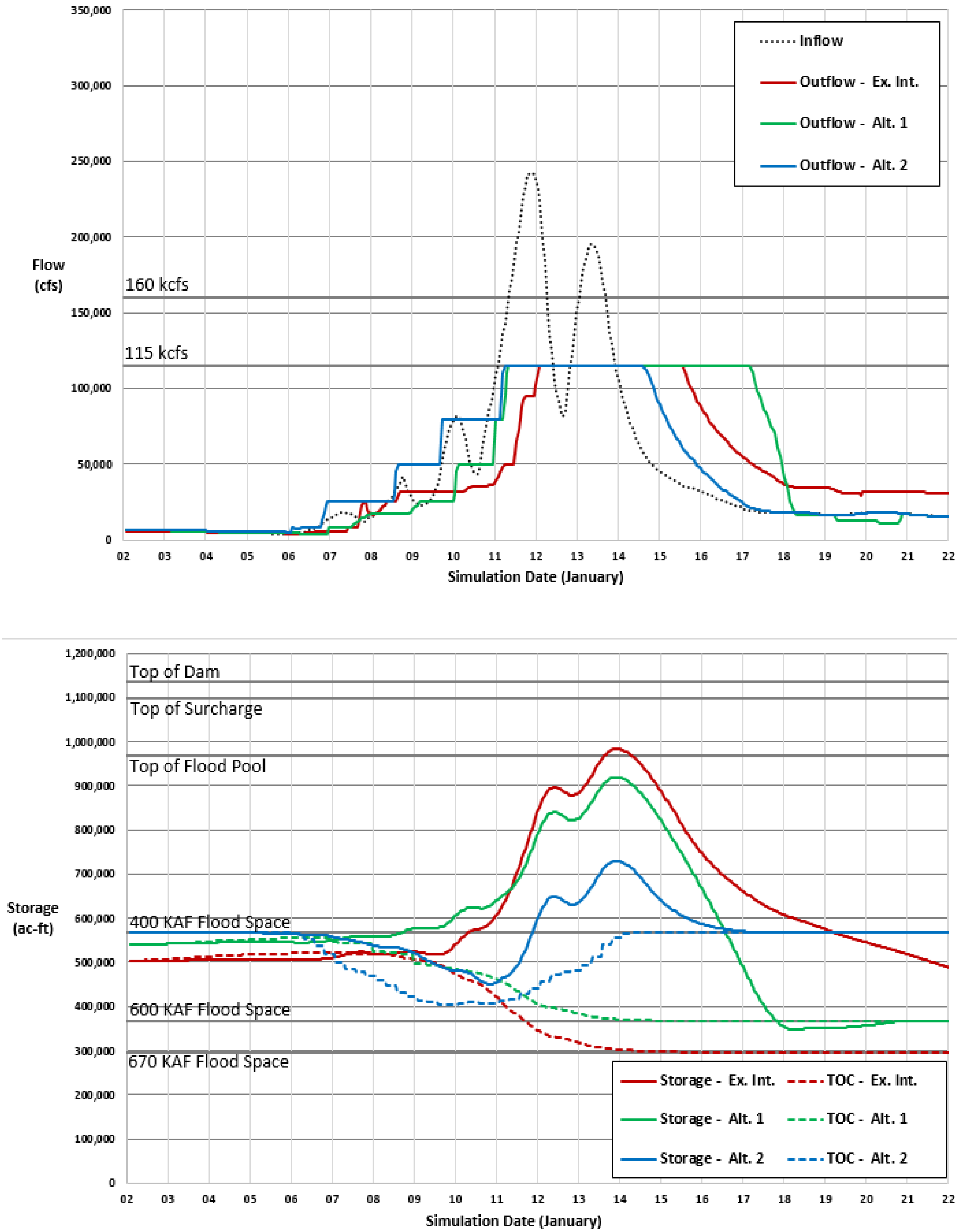


Figure 6-1: Scaled WY 1986 Event Pattern, ACE=1/100

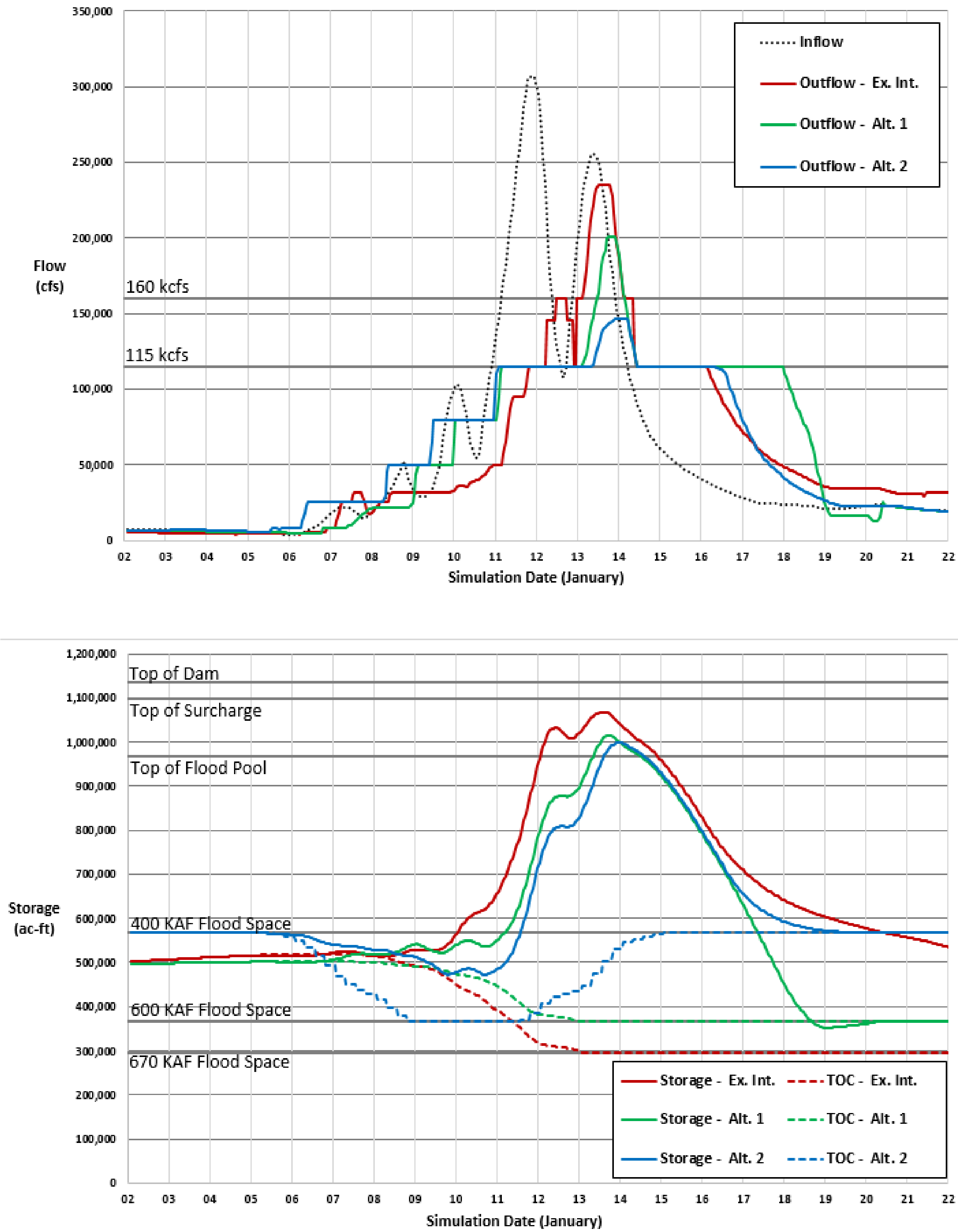


Figure 6-2: Scaled WY 1986 Event Pattern, ACE=1/200

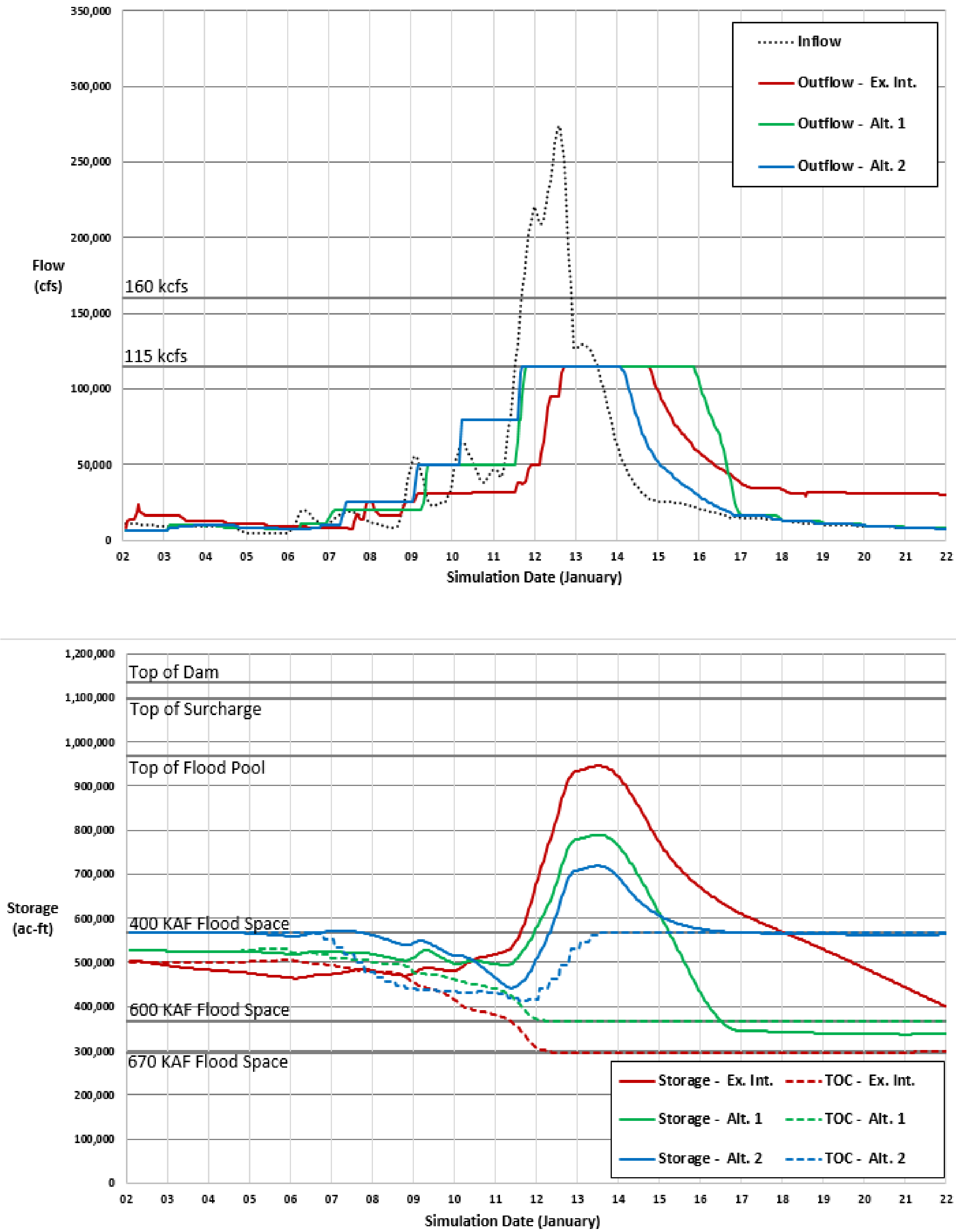


Figure 6-3: Scaled WY 1997 Event Pattern, ACE=1/100

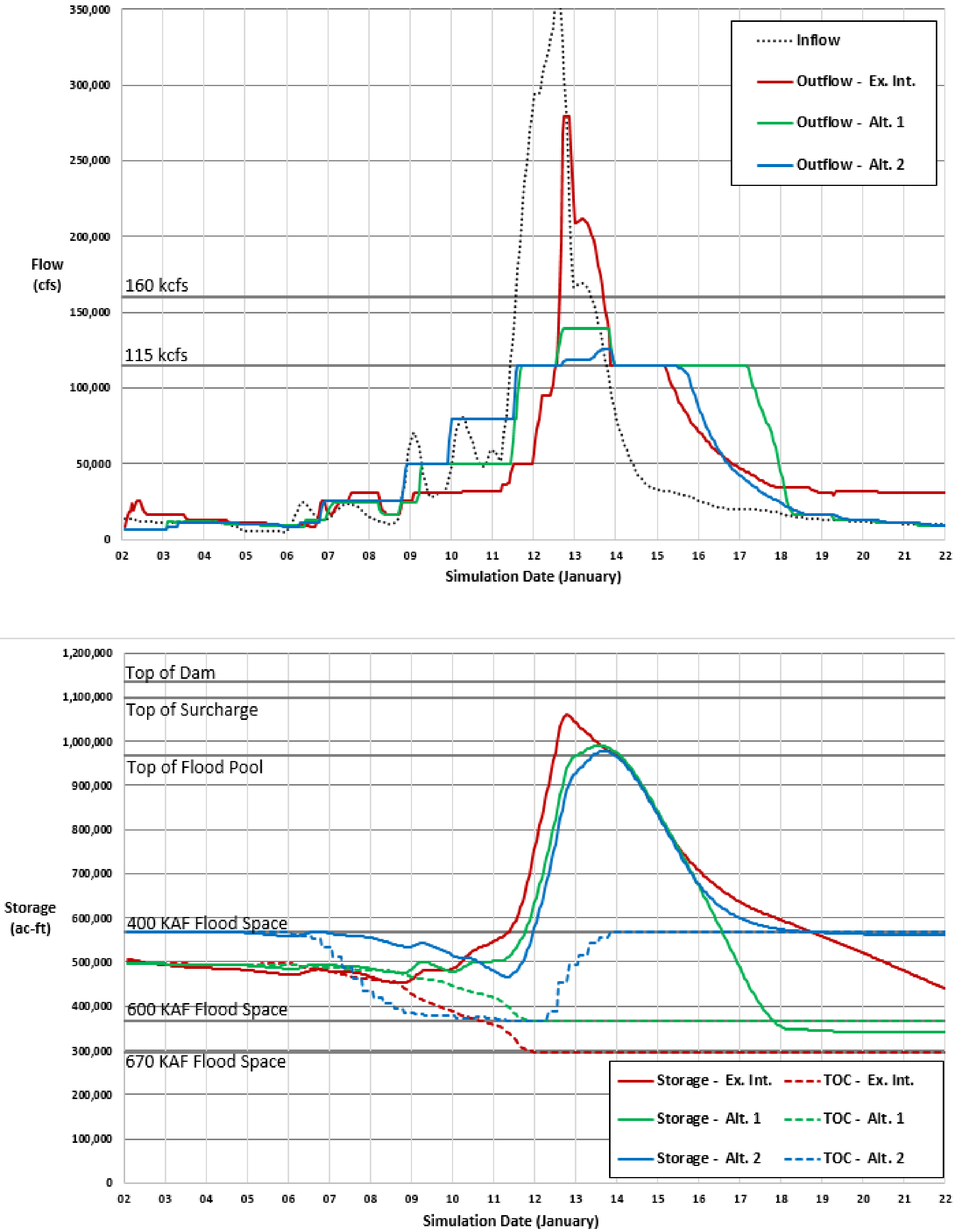


Figure 6-4: Scaled WY 1997 Event Pattern, ACE=1/200

Table 6-11: Simulation Results Comparison – ACE=1/100 Scaled Synthetic Events

Flood operation (1)	Pattern-specific ACE (2)	Event Pattern (3)	Critical Duration for Event Scaling (days) (4)	HEC-ResSim Simulation (5)	HEC-ResSim Alternative (6)	Peak Inflow (cfs) (7)	Peak Outflow (cfs) (8)	Peak Storage (ac-ft) (9)	Peak Pool Elevation (ft, ngvd29) (10)
Existing Interim (no JFP spillway) (E504)	1/100	PMF	3	E5R4-Scaled-WM	E504-A3WM	303,387	160,000	1,028,499	471.46
		SPF	2		E504-B2WM	306,587	115,000	952,410	464.70
		WY 1956	2		E504-C2WM	246,212	115,000	962,514	465.61
		WY 1964	3		E504-D3WM	262,553	115,000	975,281	466.76
		WY 1986	3		E504-E3WM	237,682	115,000	983,204	467.47
		WY 1997	2		E504-F2WM	268,484	115,000	945,012	464.03
Alternative 1 Credit-based (J602P)	1/100	PMF	3	P-Winter-scaled	J602p3A3wM	303,387	115,000	885,901	458.60
		SPF	2		J602p3B2wM	306,587	115,000	762,248	446.67
		WY 1956	2		J602p3C2wM	246,212	115,000	919,440	461.70
		WY 1964	3		J602p3D3wM	262,553	115,000	886,669	458.67
		WY 1986	3		J602p3E3wM	237,682	115,000	919,608	461.72
		WY 1997	2		J602p3F2wM	268,484	115,000	788,932	449.32
Alternative 2 Forecast-based (J602F)	1/100	PMF	3	F-Winter-scaled	J602f3A3wM	303,387	115,000	846,469	454.88
		SPF	2		J602f3B2wM	306,587	115,000	749,847	445.43
		WY 1956	2		J602f3C2wM	246,212	115,000	690,997	439.38
		WY 1964	3		J602f3D3wM	262,553	115,000	671,010	437.26
		WY 1986	3		J602f3E3wM	237,682	115,000	728,615	443.27
		WY 1997	2		J602f3F2wM	268,484	115,000	718,523	442.24
Seasonal Guide Curve Alternatives 1 and 2	1/100	WY 1986	3	F-March	J602f3E3pM	194,302	115,000	636,399	433.53
		Mar-95	3		J602f3H3pM	207,967	115,000	656,166	435.67
		WY 1997	2		J602f3F2pM	164,137	115,000	693,621	439.65
		WY 1986	3	F-April	J602f3E3rM	133,301	115,000	801,981	450.60
		Mar-95	3		J602f3H3rM	108,020	115,000	830,714	453.38
		WY 1997	2		J602f3F2rM	135,360	108,020	831,391	453.44
		WY 1986	3	F-May	J602f3E3mM	99,270	99,270	921,790	461.92
		Mar-95	3		J602f3H3mM	80,426	105,570	935,196	463.14
		WY 1997	2		J602f3F2mM	105,570	80,000	940,000	463.58

Table 6-12: Simulation Results Comparison – ACE=1/200 Scaled Synthetic Events

Flood operation (1)	Pattern-specific ACE (2)	Event Pattern (3)	Critical Duration for Event Scaling (days) (4)	HEC-ResSim Simulation (5)	HEC-ResSim Alternative (6)	Peak Inflow (cfs) (7)	Peak Outflow (cfs) (8)	Peak Storage (ac-ft) (9)	Peak Pool Elevation (ft, ngvd29) (10)
Existing Interim (no JFP spillway) (E504)	1/200	PMF	3	E5R4-Scaled-WM	E504-A3WM	393,166	355,818	1,068,691	474.93
		SPF	2		E504-B2WM	400,404	292,278	1,060,114	474.19
		WY 1956	2		E504-C2WM	311,949	243,456	1,060,490	474.23
		WY 1964	3		E504-D3WM	333,552	224,214	1,069,575	475.00
		WY 1986	3		E504-E3WM	300,749	234,928	1,067,893	474.86
		WY 1997	2		E504-F2WM	351,181	278,885	1,059,953	474.18
Alternative 1 Credit-based (J602P)	1/200	PMF	3	P-Winter-scaled	J602p3A3wM	393,166	139,397	991,282	468.18
		SPF	2		J602p3B2wM	400,404	125,459	957,710	465.18
		WY 1956	2		J602p3C2wM	311,949	134,885	985,130	467.64
		WY 1964	3		J602p3D3wM	333,552	217,544	1,015,331	470.31
		WY 1986	3		J602p3E3wM	300,749	200,756	1,014,379	470.22
		WY 1997	2		J602p3F2wM	351,181	139,212	990,763	468.14
Alternative 2 Forecast-based (J602F)	1/200	PMF	3	F-Winter-scaled	J602f3A3wM	393,166	149,559	1,001,176	469.06
		SPF	2		J602f3B2wM	400,404	124,918	957,141	465.13
		WY 1956	2		J602f3C2wM	311,949	115,000	902,111	460.10
		WY 1964	3		J602f3D3wM	333,552	115,014	944,548	463.99
		WY 1986	3		J602f3E3wM	300,749	146,178	997,865	468.77
		WY 1997	2		J602f3F2wM	351,181	125,354	977,081	466.92
Seasonal Guide Curve Alternatives 1 and 2	1/200	WY 1986	3	F-March	J602f3E3pM	241,552	115,000	786,549	449.08
		Mar-95	3		J602f3H3pM	215,675	115,000	735,753	444.00
		WY 1997	2		J602f3F2pM	271,032	115,000	775,903	448.03
		WY 1986	3	F-April	J602f3E3rM	161,149	115,000	820,979	452.44
		Mar-95	3		J602f3H3rM	132,937	115,000	878,767	457.93
		WY 1997	2		J602f3F2rM	181,216	115,000	831,391	453.44
		WY 1986	3	F-May	J602f3E3mM	116,406	110,370	921,039	461.85
		Mar-95	3		J602f3H3mM	97,688	115,000	963,483	465.70
		WY 1997	2		J602f3F2mM	135,583	90,868	940,000	463.58

Table 6-13: Simulation Results Comparison – WY 1956 Event Pattern, ACE=1/2 to 1/1000

Scaled WY 1956 event 2-day critical duration		Operation: Existing Interim (no JFP) HEC-ResSim model: E504				Operation: Alternative 1 - Credit-based HEC-ResSim model: J602P3				Operation: Alternative 2 - Forecast-based HEC-ResSim model: J602F3			
Pattern-specific 1/ACE (years) (1)	Peak Inflow (cfs) (2)	Peak Outflow (cfs) (3)	Minimum TOC Storage (ac-ft) (4)	Peak Storage (ac-ft) (5)	Peak Pool Elevation (ft, ngvd29) (6)	Peak Outflow (cfs) (7)	Minimum TOC Storage (ac-ft) (8)	Peak Storage (ac-ft) (9)	Peak Pool Elevation (ft, ngvd29) (10)	Peak Outflow (cfs) (11)	Minimum TOC Storage (ac-ft) (12)	Peak Storage (ac-ft) (13)	Peak Pool Elevation (ft, ngvd29) (14)
2	30,143	31,622	566,823	549,856	423.63	25,000	566,934	586,946	427.99	30,143	566,934	574,759	426.58
5	62,779	37,286	551,269	607,903	430.37	50,000	516,997	639,981	433.92	62,779	566,934	581,610	427.38
10	93,385	57,673	535,714	669,666	437.12	80,000	482,928	632,480	433.10	80,000	545,234	624,989	432.27
15	116,569	71,010	533,352	704,960	440.83	115,000	466,818	635,649	433.45	111,823	522,612	578,644	427.03
20	130,293	78,491	531,988	726,158	443.02	115,000	457,175	640,217	433.95	80,000	508,755	637,976	433.70
25	143,673	88,191	531,539	747,779	445.22	115,000	447,530	665,887	436.72	111,056	497,784	600,276	429.51
30	156,611	98,132	531,237	769,230	447.37	115,000	438,175	648,460	434.84	85,387	487,177	618,853	431.59
35	166,572	105,809	531,024	787,126	449.14	115,000	430,942	674,795	437.67	90,745	479,010	640,080	433.93
40	174,463	113,004	530,865	799,135	450.32	115,000	425,206	693,312	439.62	94,989	472,540	650,183	435.03
45	183,138	115,000	530,699	814,705	451.83	115,000	419,076	721,993	442.59	115,000	465,428	601,097	429.60
50	190,392	115,000	530,565	828,675	453.18	115,000	413,803	746,221	445.06	115,000	459,480	621,027	431.84
60	207,275	115,000	530,265	862,735	456.42	115,000	401,519	803,789	450.77	115,000	445,547	603,002	429.82
70	220,180	115,000	530,042	892,388	459.20	115,000	393,747	848,974	455.12	115,000	434,852	615,257	431.19
80	230,392	115,000	529,875	918,794	461.64	115,000	390,206	885,104	458.52	115,000	426,380	644,922	434.46
90	239,003	115,000	529,740	941,067	463.67	115,000	387,332	907,001	460.56	115,000	419,520	668,519	437.00
100	246,212	115,000	529,630	962,514	465.61	115,000	384,935	919,440	461.70	115,000	413,864	690,997	439.38
110	256,965	115,000	529,460	990,505	468.11	115,329	381,141	944,802	464.01	115,000	405,042	708,411	441.19
120	266,305	160,000	529,315	1,016,767	470.43	118,515	378,165	962,293	465.59	115,000	397,395	739,811	444.41
130	274,476	160,000	529,189	1,030,279	471.61	125,540	377,170	972,894	466.54	115,000	390,701	768,332	447.28
140	281,705	160,939	529,081	1,038,269	472.31	133,389	376,293	980,955	467.26	115,000	384,796	794,321	449.85
150	288,201	176,701	528,984	1,045,009	472.89	140,662	375,514	988,746	467.96	115,000	379,508	818,290	452.18
160	293,962	189,406	528,898	1,050,130	473.33	150,076	374,818	998,891	468.86	115,000	374,687	840,468	454.31
170	299,134	207,880	528,822	1,053,297	473.61	161,026	374,192	1,006,275	469.51	115,000	370,312	853,100	455.51
180	303,833	219,636	528,753	1,055,516	473.80	175,561	373,627	1,010,573	469.89	115,000	366,934	870,348	457.14
190	308,102	232,280	528,690	1,058,247	474.03	128,877	373,113	979,096	467.10	115,000	366,934	886,775	458.68
200	311,949	243,456	528,633	1,060,490	474.23	134,885	372,644	985,130	467.64	115,000	366,934	902,111	460.10
210	319,172	255,898	528,527	1,062,261	474.38	146,702	371,770	996,509	468.65	115,073	366,934	930,926	462.75
220	325,894	282,038	528,430	1,062,365	474.39	160,025	370,957	1,005,200	469.42	117,595	366,934	955,399	464.97
230	332,129	301,610	528,340	1,064,259	474.55	175,946	370,200	1,010,518	469.88	120,517	366,934	964,056	465.75
240	337,967	312,500	528,256	1,065,420	474.65	196,618	369,487	1,012,460	470.05	129,542	366,934	976,971	466.91
250	344,105	327,221	528,177	1,067,423	474.82	217,136	368,814	1,014,080	470.20	134,077	366,934	981,147	467.28
275	359,272	352,607	528,006	1,070,227	475.06	262,023	367,300	1,018,130	470.55	154,387	366,934	1,001,516	469.09
300	373,808	372,315	527,858	1,074,474	475.42	299,281	366,934	1,019,938	470.71	183,925	366,934	1,009,144	469.76
350	398,409	400,683	527,622	1,081,502	476.02	343,104	366,934	1,022,663	470.95	241,680	366,934	1,015,121	470.29
400	416,671	418,480	527,439	1,086,692	476.46	375,842	366,934	1,024,433	471.10	299,146	366,934	1,019,763	470.69
450	430,831	432,194	527,292	1,089,466	476.70	406,150	366,934	1,025,624	471.21	328,093	366,934	1,022,679	470.95
500	442,072	443,152	527,176	1,092,481	476.95	419,572	366,934	1,027,505	471.37	356,814	366,934	1,023,100	470.99
550	461,153	462,521	526,967	1,097,318	477.36	447,975	366,934	1,029,559	471.55	398,974	366,934	1,025,503	471.20
600	477,227	478,635	526,788	1,104,890	478.00	473,489	366,934	1,030,730	471.65	429,479	366,934	1,028,046	471.42
700	502,911	504,522	526,501	1,115,592	478.90	501,408	366,934	1,031,798	471.74	475,128	366,934	1,030,994	471.67
800	522,612	524,336	526,276	1,122,179	479.46	518,962	366,934	1,033,576	471.90	511,179	366,934	1,032,924	471.84
900	538,340	540,050	526,100	1,127,029	479.86	535,141	366,934	1,034,571	471.99	528,081	366,934	1,034,529	471.98
1000	551,203	554,550	525,955	1,130,070	480.12	550,636	366,934	1,035,325	472.05	549,442	366,934	1,035,292	472.05

Table 6-14: Simulation Results Comparison – WY 1964 Event Pattern, ACE=1/2 to 1/1000

Scaled WY 1964 event 3-day critical duration		Operation: Existing Interim (no JFP) HEC-ResSim model: E504				Operation: Alternative 1 - Credit-based HEC-ResSim model: J602P3				Operation: Alternative 2 - Forecast-based HEC-ResSim model: J602F3			
Pattern-specific 1/ACE (years) (1)	Peak Inflow (cfs) (2)	Peak Outflow (cfs) (3)	Minimum TOC Storage (ac-ft) (4)	Peak Storage (ac-ft) (5)	Peak Pool Elevation (ft, ngvd29) (6)	Peak Outflow (cfs) (7)	Minimum TOC Storage (ac-ft) (8)	Peak Storage (ac-ft) (9)	Peak Pool Elevation (ft, ngvd29) (10)	Peak Outflow (cfs) (11)	Minimum TOC Storage (ac-ft) (12)	Peak Storage (ac-ft) (13)	Peak Pool Elevation (ft, ngvd29) (14)
2	31,880	27,732	543,114	533,710	421.67	25,000	566,934	586,585	427.95	27,191	566,934	578,320	427.00
5	66,652	35,797	534,050	586,350	427.92	50,000	462,184	619,493	431.66	65,000	566,934	584,579	427.72
10	99,227	49,144	531,985	642,293	434.17	80,000	422,072	643,830	434.34	65,000	544,270	590,009	428.34
15	123,921	62,125	530,739	681,583	438.39	115,000	396,598	659,363	436.02	74,545	526,059	628,584	432.67
20	138,534	70,580	530,109	703,702	440.70	115,000	383,862	671,509	437.32	83,154	515,182	590,449	428.39
25	152,792	78,337	529,536	725,856	442.99	115,000	374,272	687,017	438.96	80,000	504,519	601,106	429.60
30	166,586	87,261	529,019	745,269	444.96	115,000	370,840	683,485	438.59	80,000	493,917	609,627	430.56
35	177,207	93,316	528,639	759,472	446.39	115,000	368,183	697,035	440.01	80,000	485,138	631,797	433.02
40	185,620	99,287	528,347	771,676	447.61	115,000	366,934	707,145	441.06	80,000	477,747	625,567	432.34
45	194,874	103,943	528,035	783,917	448.82	115,000	366,934	724,074	442.81	80,000	469,557	655,924	435.65
50	202,613	109,861	527,780	794,445	449.86	115,000	366,934	738,974	444.33	95,000	462,689	621,968	431.94
60	220,635	115,000	527,203	825,507	452.88	115,000	366,934	777,023	448.14	115,000	446,305	588,723	428.20
70	234,491	115,000	526,773	871,709	457.27	115,000	366,934	832,804	453.58	115,000	433,580	612,403	430.87
80	245,820	115,000	526,440	914,875	461.28	115,000	366,934	870,653	457.17	115,000	423,537	609,316	430.53
90	255,099	115,000	526,172	946,556	464.17	115,000	366,934	892,077	459.17	115,000	415,403	643,415	434.29
100	262,553	115,000	525,956	975,281	466.76	115,000	366,934	886,669	458.67	115,000	408,652	671,010	437.26
110	274,142	160,000	525,618	1,018,513	470.59	115,000	366,934	920,536	461.80	115,000	398,097	716,557	442.03
120	284,269	160,000	525,331	1,034,617	471.99	115,017	366,934	947,004	464.21	115,000	388,934	752,279	445.67
130	293,119	168,589	525,080	1,049,261	473.26	118,880	366,934	970,386	466.32	115,000	380,914	784,932	448.92
140	301,000	178,091	524,862	1,057,115	473.94	130,054	366,934	981,848	467.34	115,000	373,833	815,498	451.91
150	307,939	186,130	524,669	1,061,601	474.32	144,191	366,934	995,805	468.58	115,000	367,540	842,888	454.54
160	314,161	199,274	524,497	1,064,934	474.61	157,398	366,934	1,007,093	469.58	115,000	366,934	868,690	456.99
170	319,765	206,968	524,343	1,066,608	474.75	170,312	366,934	1,012,647	470.07	115,000	366,934	887,351	458.73
180	324,782	213,694	524,205	1,067,742	474.85	198,731	366,934	1,013,975	470.19	115,000	366,934	908,006	460.65
190	329,363	219,485	524,080	1,068,807	474.94	211,796	366,934	1,014,389	470.22	115,000	366,934	926,871	462.38
200	333,552	224,214	523,966	1,069,575	475.00	217,544	366,934	1,015,331	470.31	115,014	366,934	944,548	463.99
210	341,343	234,366	523,752	1,071,204	475.14	232,975	366,934	1,016,213	470.38	120,899	366,934	972,629	466.52
220	348,618	242,779	523,554	1,072,166	475.23	241,970	366,934	1,016,705	470.43	135,582	366,934	987,381	467.84
230	355,406	247,747	523,368	1,072,202	475.23	248,430	366,934	1,016,987	470.45	148,344	366,934	999,868	468.94
240	361,702	249,832	523,195	1,071,581	475.18	253,749	366,934	1,017,297	470.48	162,213	366,934	1,009,933	469.83
250	367,629	249,713	523,037	1,070,309	475.07	258,210	366,934	1,017,807	470.52	187,805	366,934	1,013,888	470.18
275	380,990	241,091	522,673	1,060,472	474.22	268,426	366,934	1,018,420	470.58	230,277	366,934	1,016,047	470.37
300	392,561	264,805	522,372	1,062,562	474.40	277,077	366,934	1,018,916	470.62	255,889	366,934	1,017,734	470.52
350	412,435	324,468	521,884	1,065,159	474.63	291,210	366,934	1,019,583	470.68	288,995	366,934	1,019,472	470.67
400	429,497	356,041	521,507	1,067,579	474.83	302,346	366,934	1,019,875	470.70	302,346	366,934	1,019,875	470.70
450	445,384	384,569	521,204	1,070,283	475.06	305,347	366,934	1,020,486	470.76	308,888	366,934	1,020,781	470.78
500	459,174	413,020	520,957	1,072,594	475.26	285,814	366,934	1,019,344	470.66	317,803	366,934	1,021,400	470.84
550	484,178	455,530	520,521	1,074,675	475.44	323,559	366,934	1,022,214	470.91	330,497	366,934	1,022,122	470.90
600	505,871	480,894	520,144	1,078,946	475.80	358,974	366,934	1,023,751	471.04	341,805	366,934	1,022,624	470.94
700	540,432	499,393	519,533	1,088,816	476.64	415,909	366,934	1,026,005	471.24	360,442	366,934	1,023,040	470.98
800	566,025	510,819	519,054	1,094,946	477.16	452,813	366,934	1,029,817	471.57	331,573	366,934	1,022,654	470.95
900	585,775	517,514	518,673	1,098,653	477.48	492,940	366,934	1,031,561	471.72	371,536	366,934	1,024,392	471.10
1000	602,326	535,266	518,354	1,108,392	478.30	509,220	366,934	1,033,162	471.86	405,137	366,934	1,025,340	471.18

Table 6-15: Simulation Results Comparison – WY 1986 Event Pattern, ACE=1/2 to 1/1000

Scaled WY 1986 event 3-day critical duration		Operation: Existing Interim (no JFP) HEC-ResSim model: E504				Operation: Alternative 1 - Credit-based HEC-ResSim model: J602P3				Operation: Alternative 2 - Forecast-based HEC-ResSim model: J602F3			
Pattern-specific 1/ACE (years) (1)	Peak Inflow (cfs) (2)	Peak Outflow (cfs) (3)	Minimum TOC Storage (ac-ft) (4)	Peak Storage (ac-ft) (5)	Peak Pool Elevation (ft, ngvd29) (6)	Peak Outflow (cfs) (7)	Minimum TOC Storage (ac-ft) (8)	Peak Storage (ac-ft) (9)	Peak Pool Elevation (ft, ngvd29) (10)	Peak Outflow (cfs) (11)	Minimum TOC Storage (ac-ft) (12)	Peak Storage (ac-ft) (13)	Peak Pool Elevation (ft, ngvd29) (14)
2	28,599	31,516	556,003	539,248	422.34	25,000	566,934	589,579	428.29	28,599	566,934	572,815	426.35
5	60,334	35,619	529,532	587,037	428.00	50,000	482,339	618,710	431.58	60,334	564,969	579,493	427.13
10	90,089	52,236	527,013	651,757	435.20	80,000	456,378	636,266	433.51	71,357	538,777	595,645	428.99
15	112,644	67,300	525,763	694,082	439.70	80,000	435,992	669,674	437.12	80,000	518,913	607,826	430.36
20	125,992	74,768	525,072	715,677	441.94	115,000	423,799	664,999	436.62	80,000	507,158	603,225	429.84
25	138,926	84,470	524,417	737,762	444.20	115,000	411,004	680,096	438.23	107,674	495,710	575,575	426.68
30	151,436	93,041	523,795	758,984	446.34	115,000	402,746	658,145	435.89	108,802	484,668	609,724	430.57
35	161,067	100,824	523,385	774,847	447.93	115,000	396,639	676,198	437.82	111,272	476,063	586,760	427.97
40	168,697	105,317	523,176	786,839	449.11	115,000	391,627	692,910	439.58	115,000	469,198	606,857	430.25
45	177,090	112,372	522,955	798,600	450.27	115,000	386,315	710,081	441.36	95,000	461,608	648,058	434.80
50	184,109	115,000	522,774	807,810	451.16	115,000	383,537	723,593	442.76	115,000	455,229	567,031	425.67
60	200,453	115,000	522,367	842,065	454.46	115,000	376,889	771,572	447.60	115,000	439,893	581,090	427.32
70	213,019	115,000	522,045	883,192	458.34	115,000	371,819	824,649	452.79	115,000	428,017	627,121	432.51
80	222,965	115,000	521,811	921,823	461.92	115,000	367,810	868,578	456.97	115,000	418,606	655,371	435.59
90	231,024	115,000	521,625	956,190	465.04	115,000	366,942	903,475	460.23	115,000	410,929	697,521	440.06
100	237,682	115,000	521,473	983,204	467.47	115,000	366,943	919,608	461.72	115,000	404,575	728,615	443.27
110	248,064	160,000	521,238	1,024,810	471.14	115,000	366,934	906,159	460.48	115,000	394,416	781,239	448.56
120	257,064	160,000	521,038	1,041,772	472.61	115,000	366,934	938,234	463.42	115,000	385,269	830,227	453.33
130	264,937	173,166	520,863	1,053,028	473.58	116,614	366,934	962,602	465.62	115,000	377,215	836,116	453.89
140	271,878	185,766	520,710	1,058,760	474.08	127,857	366,934	979,657	467.15	115,000	370,089	875,259	457.60
150	278,042	199,766	520,577	1,063,006	474.44	142,143	366,934	994,099	468.43	115,000	366,934	908,582	460.70
160	283,552	213,936	520,458	1,066,532	474.74	157,095	366,934	1,006,885	469.56	115,000	366,934	941,394	463.70
170	288,504	219,697	520,351	1,066,953	474.78	180,208	366,934	1,013,402	470.14	117,887	366,934	968,220	466.13
180	292,979	223,562	520,254	1,067,072	474.79	208,843	366,934	1,014,831	470.26	127,167	366,934	978,946	467.09
190	297,043	229,544	520,166	1,067,649	474.84	223,815	366,934	1,015,710	470.34	136,616	366,934	988,426	467.93
200	300,749	234,928	520,087	1,067,893	474.86	200,756	366,934	1,014,379	470.22	146,178	366,934	997,865	468.77
210	307,671	240,425	519,941	1,067,251	474.81	231,995	366,934	1,016,132	470.38	159,928	366,934	1,008,945	469.75
220	314,197	244,014	519,806	1,066,516	474.74	254,821	366,934	1,017,434	470.49	204,379	366,934	1,014,213	470.21
230	320,318	246,754	519,679	1,065,677	474.67	268,321	366,934	1,018,421	470.58	226,914	366,934	1,015,906	470.36
240	326,073	232,720	519,562	1,061,855	474.34	277,799	366,934	1,018,954	470.62	245,880	366,934	1,016,863	470.44
250	331,455	230,283	519,454	1,058,171	474.03	284,682	366,934	1,019,298	470.65	255,471	366,934	1,017,715	470.52
275	343,502	265,416	519,213	1,059,493	474.14	297,666	366,934	1,019,686	470.69	281,901	366,934	1,019,166	470.64
300	353,929	285,889	519,005	1,062,013	474.36	307,758	366,934	1,020,253	470.74	306,758	366,934	1,020,202	470.73
350	372,057	326,853	518,673	1,066,332	474.73	323,268	366,934	1,021,713	470.87	323,268	366,934	1,021,716	470.87
400	386,703	355,880	518,414	1,069,051	474.96	334,929	366,934	1,022,365	470.92	334,930	366,934	1,022,365	470.92
450	397,744	378,517	518,210	1,071,668	475.18	344,702	366,934	1,022,675	470.95	344,706	366,934	1,022,675	470.95
500	406,824	394,054	518,043	1,072,644	475.27	353,336	366,934	1,022,797	470.96	353,090	366,934	1,022,798	470.96
550	423,126	421,535	517,746	1,074,600	475.43	355,723	366,934	1,023,041	470.98	366,377	366,934	1,023,308	471.00
600	437,288	438,616	517,494	1,076,099	475.56	334,375	366,934	1,022,469	470.93	375,163	366,934	1,024,422	471.10
700	460,737	462,041	517,066	1,087,005	476.49	409,279	366,934	1,025,756	471.22	377,288	366,934	1,024,488	471.11
800	482,556	483,959	516,636	1,093,787	477.07	446,420	366,934	1,029,391	471.54	379,180	366,934	1,024,632	471.12
900	502,024	503,260	516,003	1,097,868	477.41	478,944	366,934	1,031,190	471.69	417,520	366,934	1,027,375	471.36
1000	517,584	518,863	515,483	1,105,098	478.02	487,267	366,934	1,031,533	471.72	451,085	366,934	1,029,833	471.57

Table 6-16: Simulation Results Comparison – WY 1997 Event Pattern, ACE=1/2 to 1/1000

Scaled WY 1997 event 2-day critical duration		Operation: Existing Interim (no JFP) HEC-ResSim model: E504				Operation: Alternative 1 - Credit-based HEC-ResSim model: J602P3				Operation: Alternative 2 - Forecast-based HEC-ResSim model: J602F3			
Pattern-specific 1/ACE (years) (1)	Peak Inflow (cfs) (2)	Peak Outflow (cfs) (3)	Minimum TOC Storage (ac-ft) (4)	Peak Storage (ac-ft) (5)	Peak Pool Elevation (ft, ngvd29) (6)	Peak Outflow (cfs) (7)	Minimum TOC Storage (ac-ft) (8)	Peak Storage (ac-ft) (9)	Peak Pool Elevation (ft, ngvd29) (10)	Peak Outflow (cfs) (11)	Minimum TOC Storage (ac-ft) (12)	Peak Storage (ac-ft) (13)	Peak Pool Elevation (ft, ngvd29) (14)
2	29,052	31,545	557,529	536,526	422.01	25,000	566,934	593,926	428.79	29,052	566,934	570,133	426.04
5	59,832	34,381	525,952	570,769	426.11	50,000	471,481	613,664	431.02	59,832	566,934	577,446	426.89
10	88,591	39,017	521,757	612,198	430.85	80,000	428,430	607,574	430.33	80,000	553,082	625,181	432.30
15	110,377	51,541	518,935	650,156	435.03	80,000	400,198	643,193	434.27	109,580	535,829	612,077	430.84
20	124,053	58,213	518,203	671,014	437.26	115,000	384,793	662,412	436.35	115,000	520,755	572,020	426.26
25	136,982	67,799	517,503	695,505	439.85	115,000	373,196	656,163	435.67	115,000	509,484	574,089	426.50
30	149,700	76,161	516,827	719,621	442.35	115,000	369,239	682,064	438.44	104,205	499,619	604,758	430.02
35	159,396	83,318	515,539	735,025	443.92	115,000	366,943	707,627	441.11	110,000	491,101	625,390	432.32
40	167,059	88,535	514,528	748,708	445.31	115,000	366,934	726,865	443.09	110,000	483,863	589,534	428.29
45	175,506	95,000	513,416	763,879	446.83	115,000	366,934	740,276	444.46	87,940	475,734	597,641	429.21
50	182,883	102,590	512,453	781,706	448.60	115,000	366,934	758,464	446.29	115,000	468,738	574,608	426.56
60	206,949	115,000	510,272	823,674	452.70	115,000	366,934	748,348	445.27	115,000	452,286	598,549	429.31
70	229,718	115,000	508,627	862,949	456.44	115,000	366,934	779,270	448.36	115,000	439,574	631,510	432.99
80	246,633	115,000	507,330	890,822	459.06	115,000	366,934	750,061	445.45	115,000	429,401	659,328	436.01
90	259,190	115,000	506,280	923,030	462.03	115,000	366,934	771,375	447.58	115,000	421,136	687,018	438.96
100	268,484	115,000	505,412	945,012	464.03	115,000	366,934	788,932	449.32	115,000	414,276	718,523	442.24
110	282,578	115,000	504,061	982,974	467.44	115,000	366,934	821,309	452.47	115,000	399,994	769,190	447.36
120	294,590	160,000	502,889	1,020,683	470.77	115,000	366,934	846,592	454.89	115,000	390,129	810,739	451.45
130	305,034	160,000	501,973	1,024,720	471.13	115,000	366,934	881,044	458.14	115,000	381,455	842,767	454.53
140	314,069	160,000	501,973	1,031,525	471.72	115,000	366,934	913,476	461.15	115,000	373,779	876,248	457.69
150	322,038	176,167	501,973	1,039,233	472.39	115,000	366,934	939,647	463.55	115,000	366,934	889,668	458.95
160	329,142	199,589	501,973	1,047,284	473.09	116,174	366,934	960,988	465.48	115,000	366,934	898,348	459.75
170	335,499	225,884	501,973	1,051,870	473.48	122,356	366,934	974,083	466.65	115,000	366,934	923,616	462.08
180	341,243	246,467	501,973	1,055,894	473.83	130,224	366,934	982,011	467.36	115,000	366,934	946,714	464.19
190	346,405	260,974	501,973	1,056,874	473.91	135,012	366,934	986,797	467.79	116,118	366,934	966,492	465.97
200	351,181	278,885	501,973	1,059,953	474.18	139,212	366,934	990,763	468.14	125,354	366,934	977,081	466.92
210	360,117	306,368	501,973	1,063,324	474.47	132,804	366,934	984,607	467.59	134,526	366,934	986,313	467.74
220	368,520	326,664	501,973	1,065,395	474.65	139,414	366,934	991,333	468.19	141,998	366,934	993,852	468.41
230	376,264	343,468	501,973	1,067,311	474.81	146,457	366,934	998,256	468.80	149,428	366,934	1,000,785	469.03
240	383,454	358,126	501,973	1,069,489	475.00	154,355	366,934	1,003,004	469.22	161,469	366,934	1,003,303	469.25
250	390,186	368,963	501,973	1,070,504	475.08	164,747	366,934	1,003,516	469.27	172,792	366,934	1,001,000	469.04
275	405,177	399,387	501,973	1,074,287	475.41	203,339	366,934	1,001,184	469.06	205,481	366,934	1,001,888	469.12
300	418,015	419,696	501,973	1,078,811	475.79	189,691	366,934	999,082	468.87	242,992	366,934	1,010,966	469.92
350	438,882	440,411	501,973	1,084,762	476.30	256,483	366,934	1,011,745	469.99	303,392	366,934	1,020,385	470.75
400	455,324	456,914	501,973	1,088,526	476.62	311,939	366,934	1,020,626	470.77	342,832	366,934	1,023,499	471.02
450	468,612	470,199	501,973	1,092,137	476.93	367,611	366,934	1,024,029	471.07	398,662	366,934	1,025,523	471.20
500	479,484	481,072	501,973	1,095,285	477.19	399,913	366,934	1,025,877	471.23	415,199	366,934	1,027,239	471.35
550	498,751	500,293	501,973	1,099,416	477.54	446,470	366,934	1,029,488	471.54	466,715	366,934	1,030,311	471.62
600	515,199	512,616	501,973	1,100,759	477.65	485,183	366,934	1,031,783	471.74	501,053	366,934	1,031,847	471.75
700	541,784	525,824	501,973	1,103,260	477.87	508,922	366,934	1,033,128	471.86	519,222	366,934	1,033,892	471.93
800	562,567	536,679	501,973	1,109,152	478.36	550,105	366,934	1,035,746	472.09	551,474	366,934	1,035,837	472.10
900	579,256	551,852	501,973	1,117,334	479.05	575,468	366,934	1,036,921	472.19	575,877	366,934	1,036,926	472.19
1000	592,920	565,305	501,973	1,123,526	479.57	591,326	366,934	1,037,762	472.26	591,298	366,934	1,037,761	472.26

6.5 Alternative 2 – Forecast Uncertainty

Alternative 2 (forecast-based) simulations described in previous sections of this chapter reflect use of perfect forecast information, with required forecast volumes computed directly from the inflow hydrograph. This reflects the fact that drawdown curves (Figure 3-9) and stepped releases were developed based on routings using perfect forecast information. In this section, forecast uncertainty is considered, and a method identified for computing operational values of the four (24-, 48-, 72-, and 120-hour) inflow volumes.

Each forecast will consist of an ensemble of hourly inflow hydrographs, with the first ordinate of each reflecting current basin wetness and upstream storage conditions. It is expected that the forecast ensemble will include 61 hydrographs during WY 2018, the first year that the forecast-based operation would be in effect. The number of hydrographs in the ensemble will generally increase by one with each water year. Hydrographs of the ensemble are unique in temporal pattern and magnitude. Their variability reflects uncertainty associated with inputs to the forecast model, and as a whole, uncertainty associated with the resulting inflow forecast.

A hindcast is the forecast that would have been made had the forecast method been used at the time. Hindcasts can therefore only be developed for the period in which the required data inputs are available. CNRFC developed daily-issued hindcasts for the period January 1985 to December 2010. Each hindcast in the dataset consists of 62 members. The two largest events to occur in the hindcast record were the WY 1986 and 1997 events. ACE for these events are estimated as 1/70 and 1/90 respectively. Hindcast and inflow data for the these events were scaled up to obtain ACE=1/100 and ACE=1/200 events for both event patterns. These four scaled events are the focus of Sections 6.5.1 and 6.5.2. The selection of method for computing inflow volumes for use in operations is discussed in Section 6.5.4. Forecast-based simulations of historical (unscaled) events reflecting the selected method are presented in 6.5.5.

6.5.1 Inflow Forecast Volume Uncertainty

Figure G-53 through Figure G-56 in Appendix G provide scatter plot comparisons of hindcast and actual inflow volumes, computed once daily, from the hindcast period of record (January 1985 to December 2010). The record includes the WY 1986 and 1997 events, which yielded the nine greatest inflow pairs on each plot. In each plot, hindcast volumes reflecting 50 percent NEP and 75 percent NEP are shown along with best fit lines based on the greatest 2 percent of inflow volumes. The figures illustrate that, based on the hindcast period of record, 50 percent NEP inflow volumes underestimated actual inflow volumes.

Inflow volumes based on ensemble inflow hydrographs can span a wide range as an event approaches. Quartile plots of hindcast volume series for the WY 1986 and 1997 events scaled to 1/100 and 1/200 ACE are shown in Figures G-1 to G-16. In these plots, horizontal lines are shown indicating volume thresholds corresponding to 400 and 600 KAF flood space (shown as “0 and 200 KAF decrease in TOC” on the plots) as required by the drawdown curves (Figure 5-3). The plots illustrate that the frequency and magnitude of drawdown will be dependent on how the operational volume is computed from the ensemble members.

6.5.2 Robustness Tests

Robustness testing considers forecast uncertainty for ACE=1/100 and ACE=1/200 events as reflected by the WY 1986 and 1997 event hindcast ensembles. The four volumes required by the forecast-based operation can be computed for each of the 62 inflow hydrographs that comprise the hindcast ensemble. This provides a sample of 62 for each volume. Values of non-exceedence probability (NEP) based on the sample can be computed, with 0 percent, 50 percent, and 100 percent NEP corresponding to the minimum, median, and maximum values of the sample. Robustness testing is used to identify the NEP value of the four required volumes to be used during operations. A greater NEP value reflects using larger inflow forecast volumes and will result in a more aggressive flood operation. However, using 100 percent NEP volumes will result in more frequent triggering of the forecast-based operation. Further, the NEP=100 percent volume is defined by only the maximum ensemble member, making the operation sensitive to changes of just one ensemble member. Generally, a lower NEP value is expected to result in more well-behaved changes in computed inflow volume between forecasts.

In agreement with the CNRFC, if a daily-issued forecast is sufficiently large, the frequency of forecasts will increase to once per 6 hours. The capability to issue ensemble forecasts once per 6 hours is under development and will be implemented by start of WY 2018. As such, the robustness analysis was performed using daily hindcasts from the hindcast dataset, with volumes interpolated on a 6 hour interval between hindcasts.

The robustness analysis relied on simulations of the WY 1986 and 1997 events scaled to ACE=1/100 and ACE=1/200. Three types of simulations were performed, reflecting three types of forecast volume inputs:

1. Ensemble statistic (EST) simulations – Each event was simulated 21 times using the same inflow hydrograph, but with forecast volumes corresponding to a specific NEP value. NEP values ranged from 0 to 100 percent, on a 5 percent increment. For each event, 24-, 48-, 72-, and 120-hour forecast volumes were extracted from the 62-member inflow hydrographs. The 62 extracted 24-hour volumes were then ranked from highest to lowest. 24-hour volumes were then computed for each of the 21 NEP values. The process was repeated for the 28-, 72-, and 120-hour forecast volumes.

2. Ensemble member specific (EMS) simulations – Each event was simulated 62 times, using the same inflow hydrograph, but with forecast volumes extracted from a specific member hydrograph of the ensemble. EMS simulations were identified by the labels 1949 to 2010.

3. Perfect forecast simulations – Each event was simulated once, with 24-, 48-, 72-, and 120-hour volume inputs extracted directly from the synthetic inflow series. The synthetic inflow series is equivalent to the observed inflow series for an historical event.

EST simulations are of direct relevance to the actual operation, as these are the basis for selecting the operational NEP value. EMS simulations are informative in that they illustrate the degree of variability among the ensemble of inflow hydrographs. Perfect forecast simulations are relevant for validating the regulated flow frequency curve presented in Figure 6-14. Hydrograph plots of simulated storage and outflow, are provided Figure G-37 through Figure G-44.

Figure 6-5 shows distributions of peak outflow based on 42 (24 for each event pattern) EST and all 124 (62 for each event pattern) EMS simulations for ACE = 1/100 and ACE = 1/200 events. Results from simulations of the WY 1986 and 1997 pattern events have been grouped together, with contributions to the distributions noted by red and blue shading. In each plot, the total probability indicated by the red plus blue bars is one. Target release thresholds of 115 and 160 kcfs are indicated by dashed horizontal lines, and are bolded in plots of corresponding event ACE.

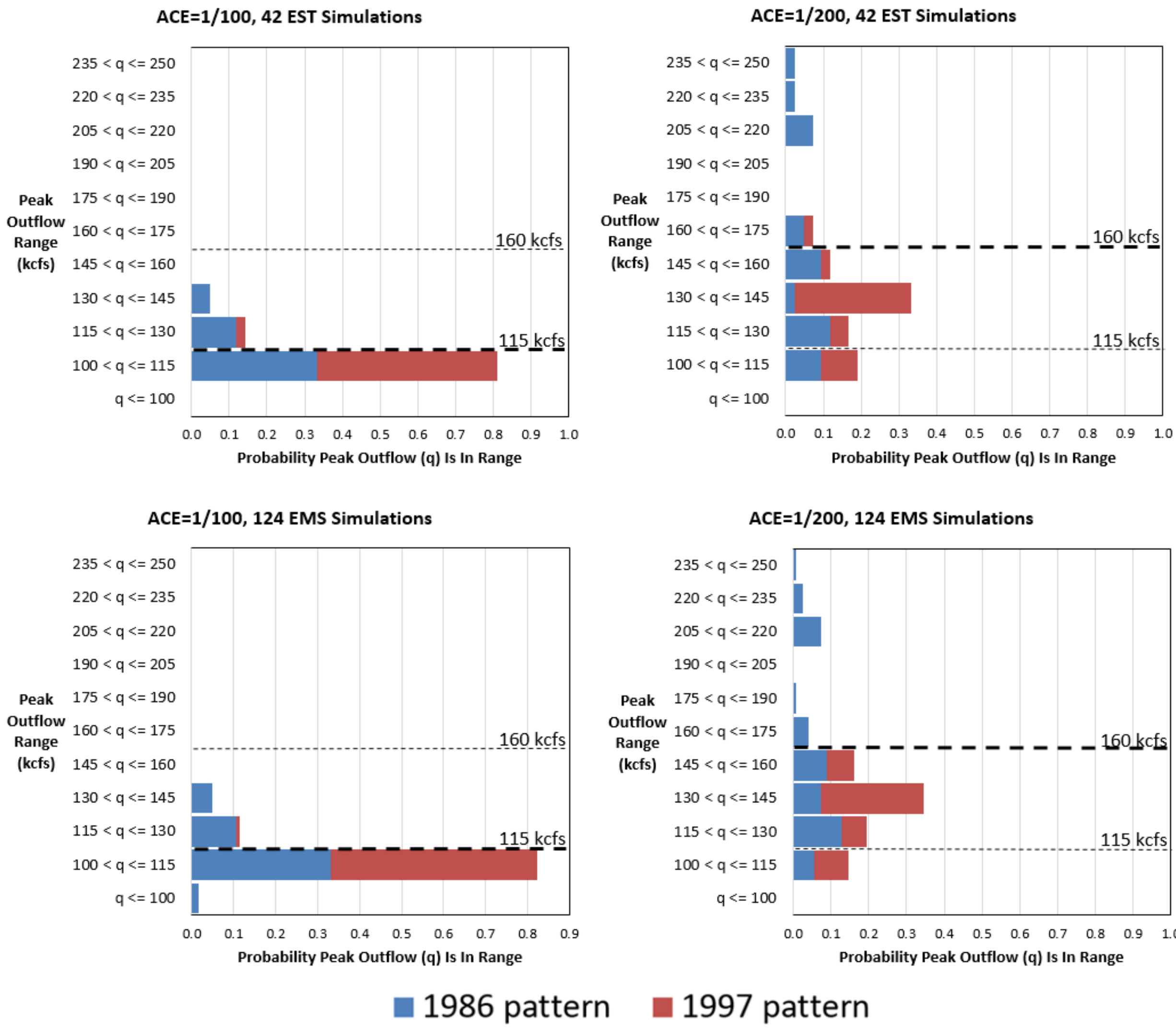


Figure 6-5: EST and EMS Peak Outflow Distributions for ACE=1/100 and 1/200

For both ACE = 1/100 and ACE = 1/200 events, the EST-based distribution indicates that most ensemble members, from which computed inflow forecast volumes were computed for each simulation, resulted in passing the event at, or less than, the target release value (115 kcfs and 160 kcfs for ACE = 1/100 and 1/200 events respectively). Similarly, most NEP values, when used to compute inflow forecast volumes, resulting in passing the event at, or less than, the target release value. The specific number of events passing and percent passing for simulations are provided in Table 6-17 through Table 6-20.

Table 6-17: EMS and EST ACE=1/100 Peak Release Summary

Event Pattern-specific ACE (1)	Event Pattern (2)	Number of **EST** Simulations having Peak Release of **115 kcfs** or Less (3)	Number of **EMS** Simulations having Peak Release of **115 kcfs** or Less (4)
1/100	WY 1986	14 of 21 (minimum passing NEP: 35%)	43 of 62 (percent passing: 69%)
	WY 1997	20 of 21 (minimum passing NEP: 5%)	61 of 62 (percent passing: 98%)

Table 6-18: EMS and EST ACE=1/200 Peak Release Summary

Event Pattern-specific ACE (1)	Event Pattern (2)	Number of **EST** Simulations having Peak Release of **160 kcfs** or Less (3)	Number of **EMS** Simulations having Peak Release of **160 kcfs** or Less (4)
1/200	WY 1986	14 of 21 (minimum passing NEP: 35%)	43 of 62 (percent passing: 69%)
	WY 1997	20 of 21 (minimum passing NEP: 5%)	62 of 62 (percent passing: 100%)

Table 6-17 shows that the operational NEP value could be as low as 35 percent to achieve FRM goals of routing 1/100 and 1/200 ACE events at 115 kcfs and 160 kcfs, respectively. Figure 6-6 shows peak outflow versus NEP from EST simulations. "PF" indicates perfect forecast-based release. The plots show that passing ACE = 1/200 events at 160 kcfs is more challenging than passing ACE = 1/200 events at 115 kcfs. The lower figure, showing ACE = 1/200, also shows peak outflow from perfect forecast simulations. While NEP = 35 percent is sufficient to satisfy the 160 kcfs release target, 75 percent NEP is necessary to route both event patterns at, or less than, the perfect forecast-based release of 125 kcfs (WY 1997 pattern). Simulations show that when 75 percent NEP volumes are used, that the resulting peak release was 116 kcfs.

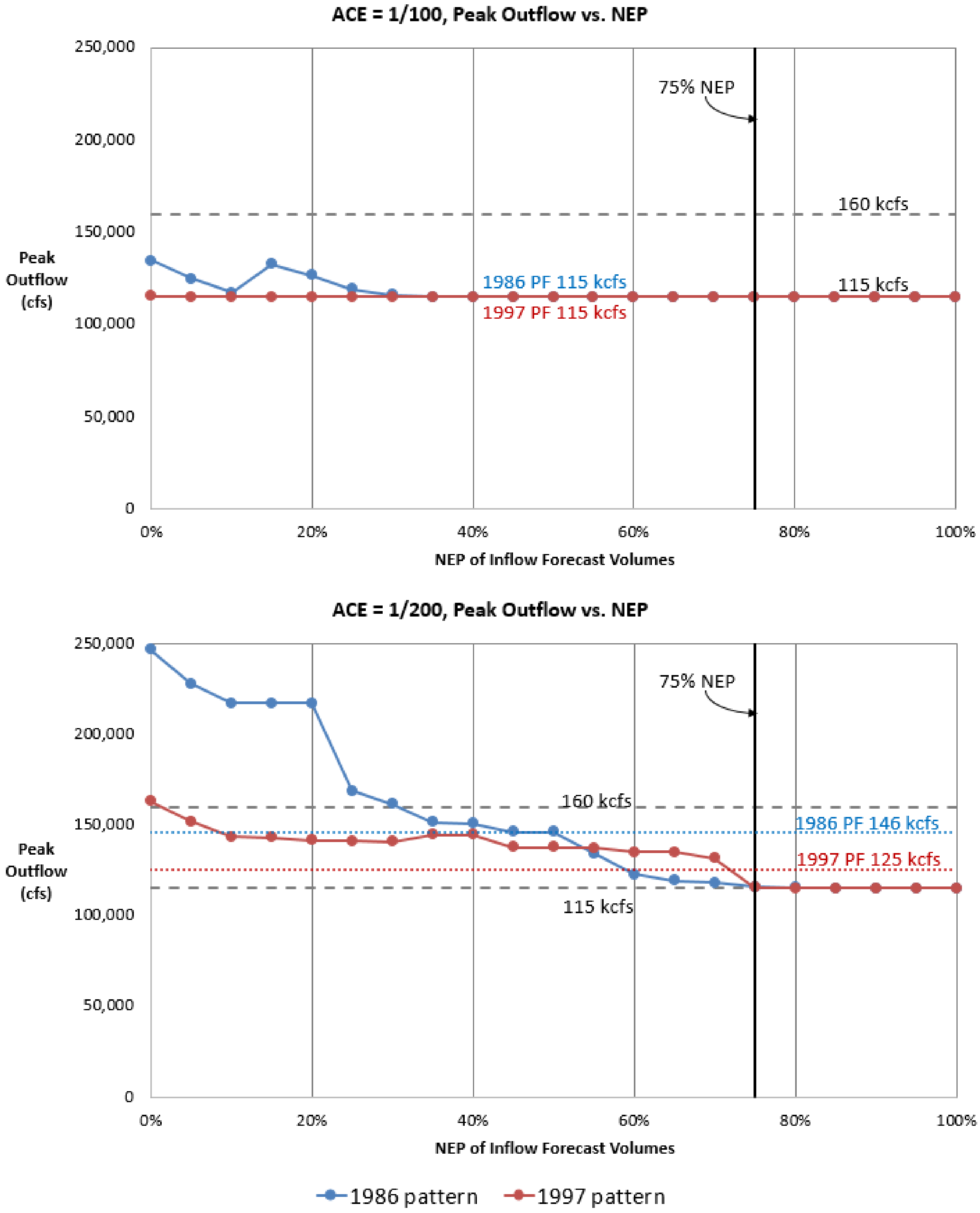


Figure 6-6: EST Simulations, Peak Outflow versus NEP

An additional test was conducted to investigate sensitivity of the operation to a systematic time shift, or under-forecast, by all ensemble members. This was simulated by configuring EST simulations with all inflow forecast volume series shifted 24 hours late. This is considered a severe test, because it does not reflect updating of forecasts to reflect the current inflow condition. Furthermore, the ensemble already reflects variability in among the members. Results from these simulations are provided in Table 6-19 and Table 6-20. Hydrograph plots are provided in Figures G-45 through G-48. Column 3 of the tables indicates that an NEP of 90 percent would be needed to ensure that FRM goals will be satisfied if all forecast volumes are 24 hours late throughout the event.

Table 6-19: EST ACE=1/100 Peak Release Summary (+24-hour Forecast Shift)

Event Pattern-specific ACE (1)	Event Pattern (2)	Number of EST Simulations having Peak Release of 115 kcfs or Less (Hindcast volumes shifted 24 Hours Late) (3)
1/100	WY 1986	4 of 21 (minimum passing NEP: 85%)
	WY 1997	7 of 21 (minimum passing NEP: 70%)

Table 6-20: EST ACE=1/200 Peak Release Summary (+24-hour Forecast Shift)

Event Pattern-specific ACE (1)	Event Pattern (2)	Number of EST Simulations having Peak Release of 160 kcfs or Less (Hindcast volumes shifted 24 Hours Late) (3)
1/200	WY 1986	4 of 21 (minimum passing NEP: 85%)
	WY 1997	3 of 21 (minimum passing NEP: 90%)

6.5.3 Change in current inflow

The EMS and EST robustness tests allow variations in inflow forecast volumes used by the operation to affect the peak outflow, but do not allow the actual inflow to vary from the scaled observed inflow. Another component of uncertainty in the forecast-based operation is how the current inflow forecasts vary with time. Figure 6-7 shows 59 hourly inflow forecast hydrographs (color lines) and the observed hydrograph (black line) for the WY 2017 February 2 through 11 event. Vertical solid grey lines indicate the times of forecast issuance (once per day at 4 AM PST). For each issued forecast, the inflow series up to the next forecast are plotted. At the time of forecast issuance, all forecast hydrographs have the same value as the observed inflow hydrograph. Between forecasts, variability among the inflow forecast hydrographs increases until the next forecast, when the inflow forecast hydrographs reset to take on the observed inflow value.

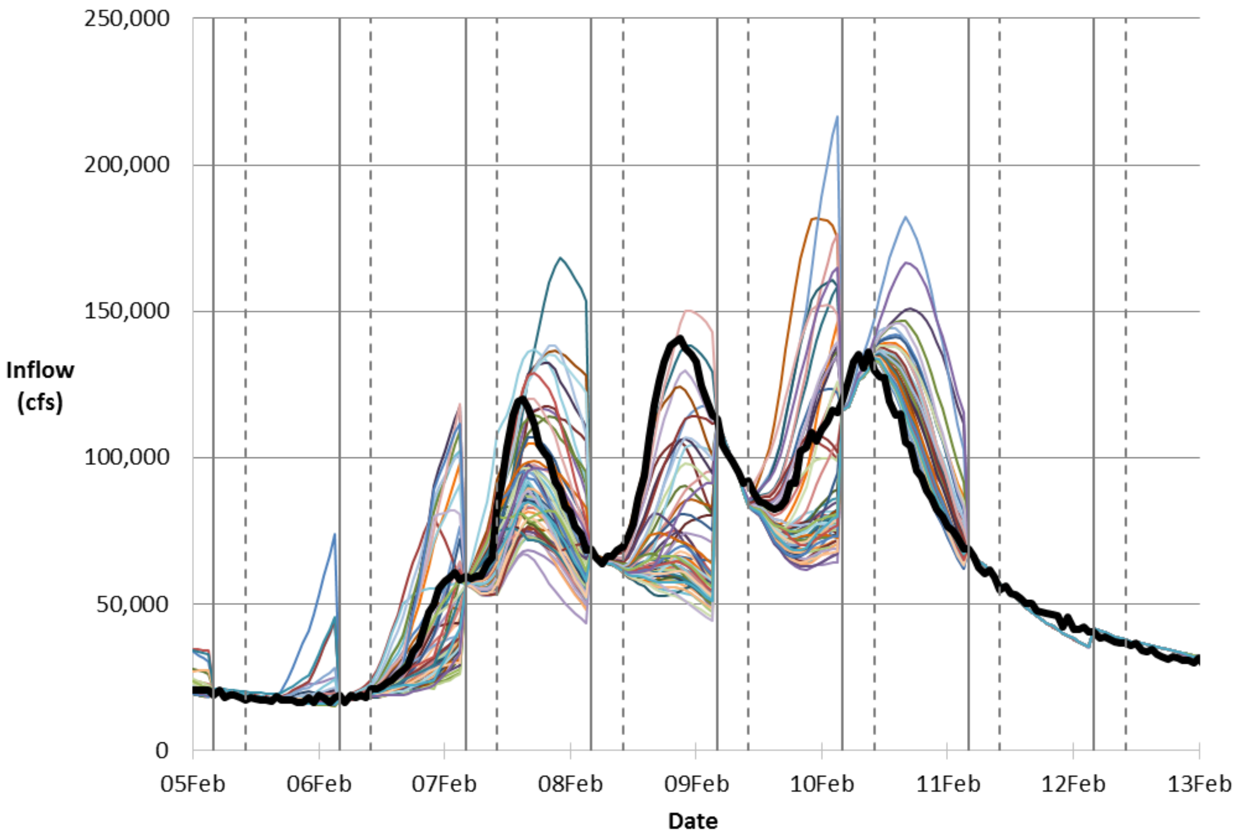


Figure 6-7: Alternative 2 – Inflow Forecast Uncertainty

While Figure 6-7 reflects daily forecast updates, the forecast-based operation requires updates to be made on a 6-hour interval, once the 120-hour 75 percent NEP inflow volume exceeds 300 KAF. Vertical dashed lines indicate the next (6 hours later) forecast would have been made under a 6-hour update interval. Inflow forecast variability is considerably reduced with the 6-hour interval. A pure forecast-based operation is vulnerable to changes in current inflow if those changes are ignored. The WCD operation therefore specifies to release the maximum of the forecast-based release or current inflow, while in a forecast-based release mode..

6.5.4 Selection of Inflow Forecast NEP Value for Operations

The approach for selecting the operational NEP value reflects the analysis steps taken in the study. The forecast-based operation was developed using perfect-forecast information, and later, robustness tests were used to identify an appropriate NEP value for operation. An alternate approach would have been to revisit and adjust the operation (drawdown curves and stepped release volume trigger) to provide target FRM performance when 50 percent NEP volumes are used. Due to study constraints, only the first approach was realistically an option. Furthermore, the approach taken, results in an effective operation.

Robustness simulations indicate NEP=35 percent is adequate (Table 6-17 and Table 6-18) to satisfy FRM goals of passing 1/100 and 1/200 ACE events at 115 kcfs and 160 kcfs respectively. Those results reflect variability of the ensemble members as reflected in the by hindcasts of the WY 1986 and 1997 events. It is possible that future forecasts of large events will under-forecast inflow volumes by more than indicated by these two events. In order to provide some degree of conservativism in the operation should this occur, an NEP value greater than 35 percent is desired. A value of 50 percent was considered. It reflects operating to median values of the four duration volumes computed from the ensemble. The median value is consistent with the concept of unbiased forecasts, with forecast volumes sometimes over- and sometimes under- predicting inflow volumes. Comparisons of hindcast and inflow volumes for the WY 1986 and 1997 events (Figure G-53 through Figure G-56) indicate inflow volumes were under-forecast. The figures also show that NEP=75 percent would have provided an improved prediction of actual inflow volumes. The robustness simulations reflecting forecasts shifted 24 hours late (Table 6-19 and Table 6-20) indicate 90 percent NEP is required to preserve FRM performance. However, this 24-hour time shift is considered severe and 90 percent NEP would place greater reliance on the most extreme ensemble members, which are expected to be less stable from forecast to forecast. Based on these considerations, NEP=75 percent is recommended for operations. This value may be updated in the future, to reflect refinements to either the operational rules or forecast skill.

Table 6-21 compares peak outflow for operations using 75 percent NEP hindcast volumes with the perfect forecast operation. Peak release values in column 4 are less than or equal to values in column 3. This result allows peak releases defined by perfect forecast simulations to conservatively be used for defining the outflow-frequency curve presented in Figure 6-14.

Table 6-21: Comparison of Perfect Forecast and 75 Percent NEP Simulated Peak Outflow

Pattern-specific ACE (1)	Event Pattern (2)	Peak Outflow Perfect Forecast Operation (kcfs) (3)	Peak Outflow 75 Percent NEP Operation (kcfs) (4)
1/100	WY 1986	115	115
1/100	WY 1997	115	115
1/200	WY 1986	146	116
1/200	WY 1997	125	116

6.5.5 Historical Event Simulations using 75 Percent NEP

Limited events exist in the hindcast period of record that were sufficiently large (75percent NEP 120-hour inflow forecast volume greater than 300 KAF) to trigger the forecast-based operation. Until the current water year, only three events fell into this category. The current water year (2017) has become the wettest on record for the Northern Sierra region. Three additional events, large enough to trigger a forecast-based operation, occurred during WY 2017. The resulting six historical events are listed in Table 6-22. Simulations of these events are valuable for testing the operation using real (unscaled) events. These events are smaller than the scaled events used to test ACE = 1/100 and 1/200 FRM performance, but are of magnitudes which will occur more frequently. Estimated event ACE values are provided in column 2 of Table 6-22, with the duration of volume used to estimate ACE indicated in parentheses. These events were simulated using 75 percent NEP inflow forecast volumes, observed inflow, and with Folsom starting at the top of variable flood space (400 KAF flood space). Hydrograph plots of these simulations are provided in Figure 6-8 through Figure 6-13.

Table 6-22: Unscaled Inflow Events Simulated using 75 Percent NEP Forecast-based operation

Event Peak Inflow Date	Event ACE	Event Volume Used for ACE Computation	Simulated Peak Release (kcfs)	Historical Peak Release (kcfs)
WY 1986 February	1/70	72-hours unimpaired inflow	104	130
WY 1995 January	1/ 5	48-hours unimpaired inflow	48	31
WY 1997 January	1/90	48-hours unimpaired inflow	115	116
09 Jan 2017	1/15[1]	72-hours observed inflow	80	59
08 Feb 2017	1/20[1]	72-hours observed inflow	80	84
21 Feb 2017	1/ 5[1]	72-hours observed inflow	50	34

[1] Estimate only. Unimpaired inflow and critical duration analysis not performed.

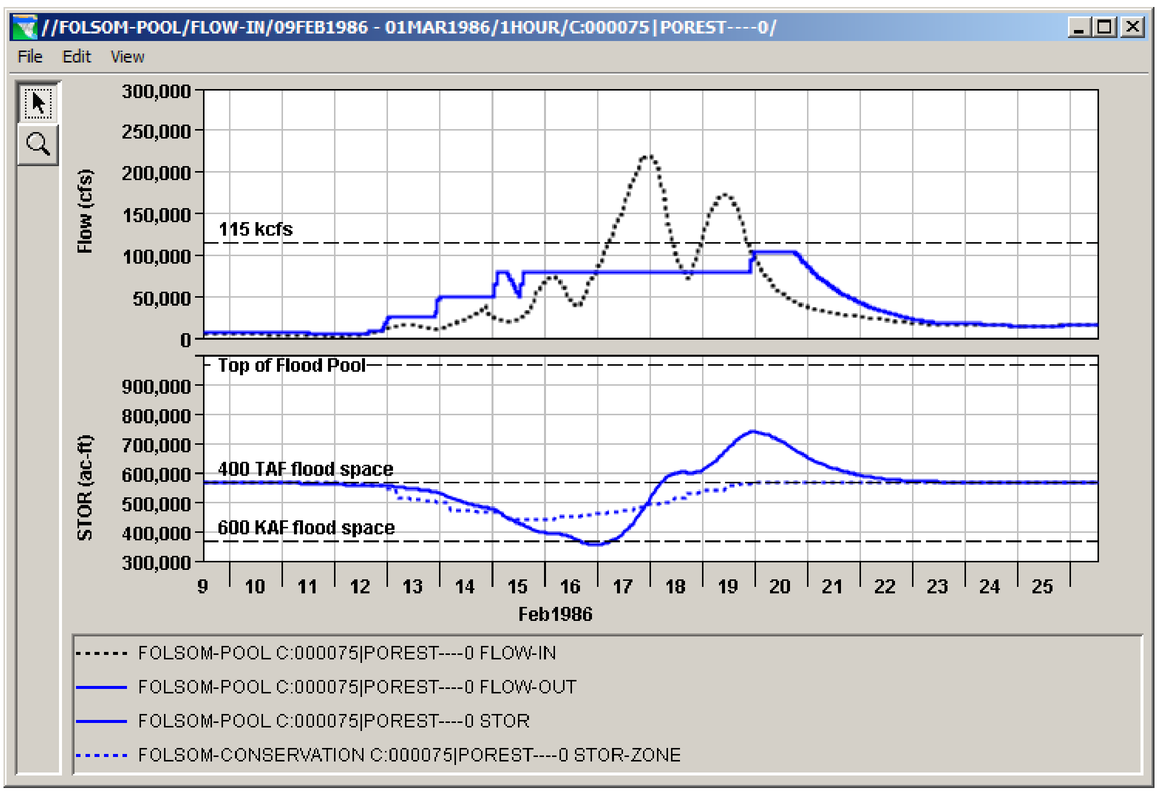


Figure 6-8: 1986 Event – Forecast-based Operation with 75 Percent NEP Volumes

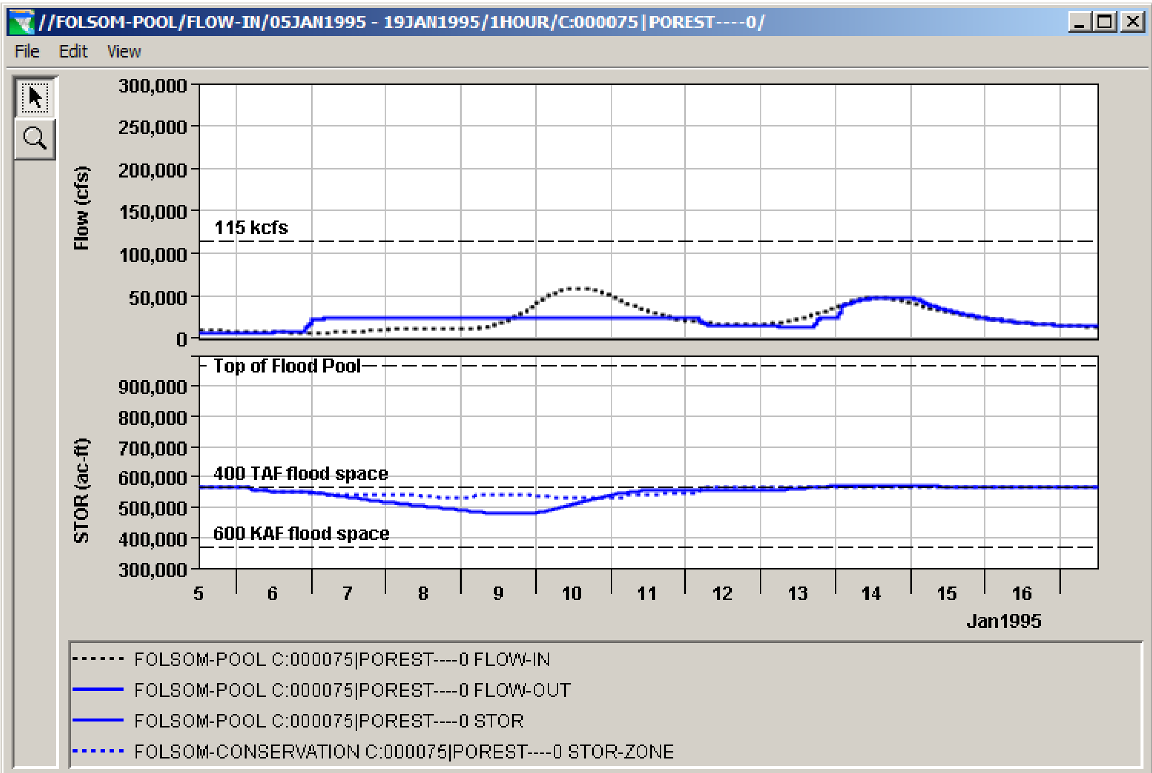


Figure 6-9: 1995 Event – Forecast-based Operation with 75 Percent NEP Volumes

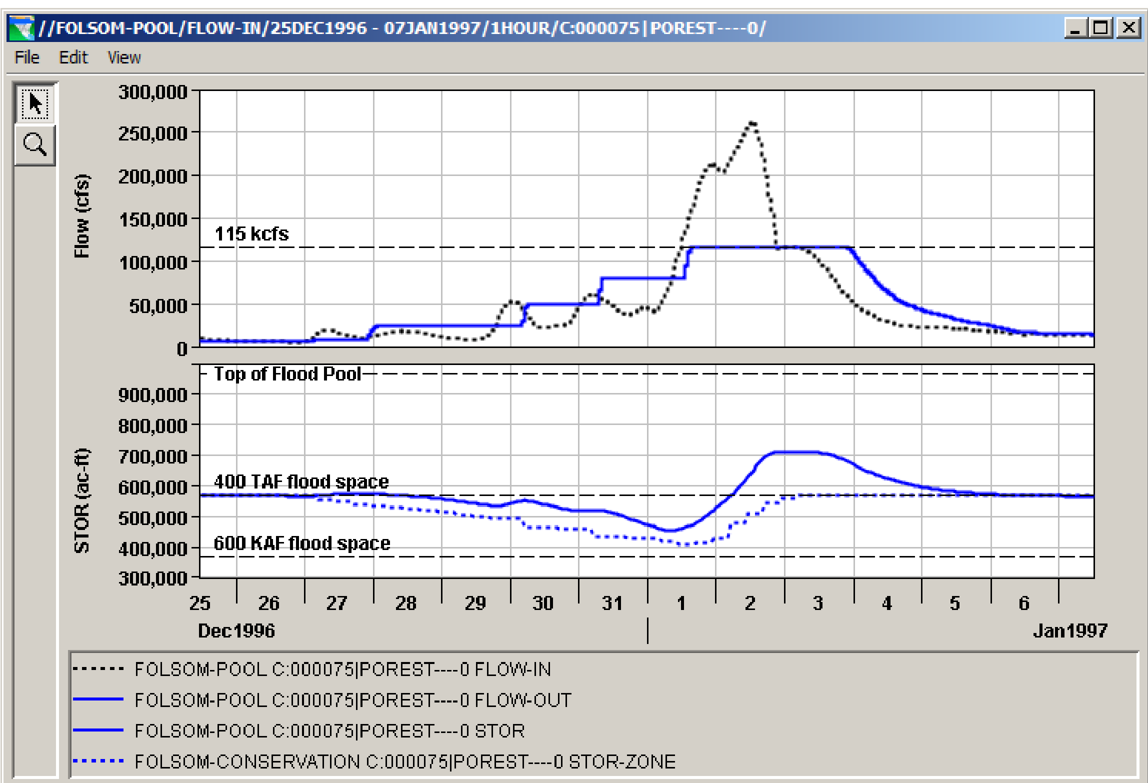


Figure 6-10: 1997 Event – Forecast-based Operation with 75 Percent NEP Volumes

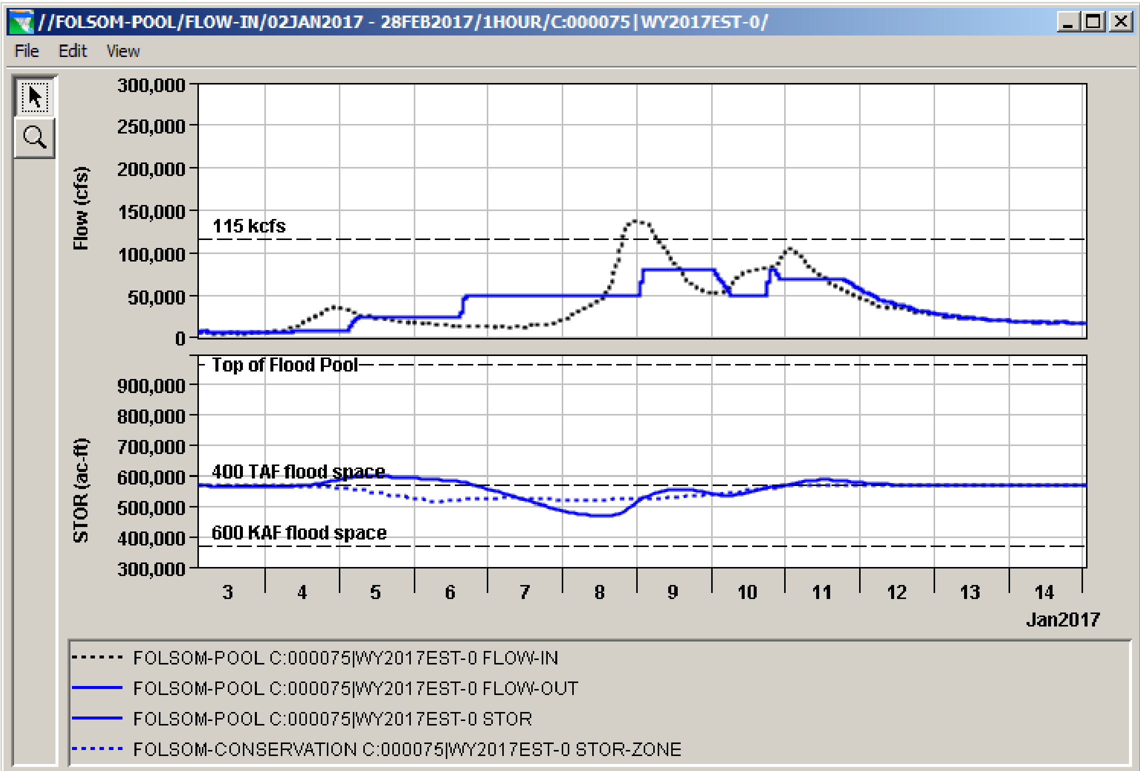


Figure 6-11: 09 Jan. 2017 Event – Forecast-based Operation with 75 Percent NEP Volumes

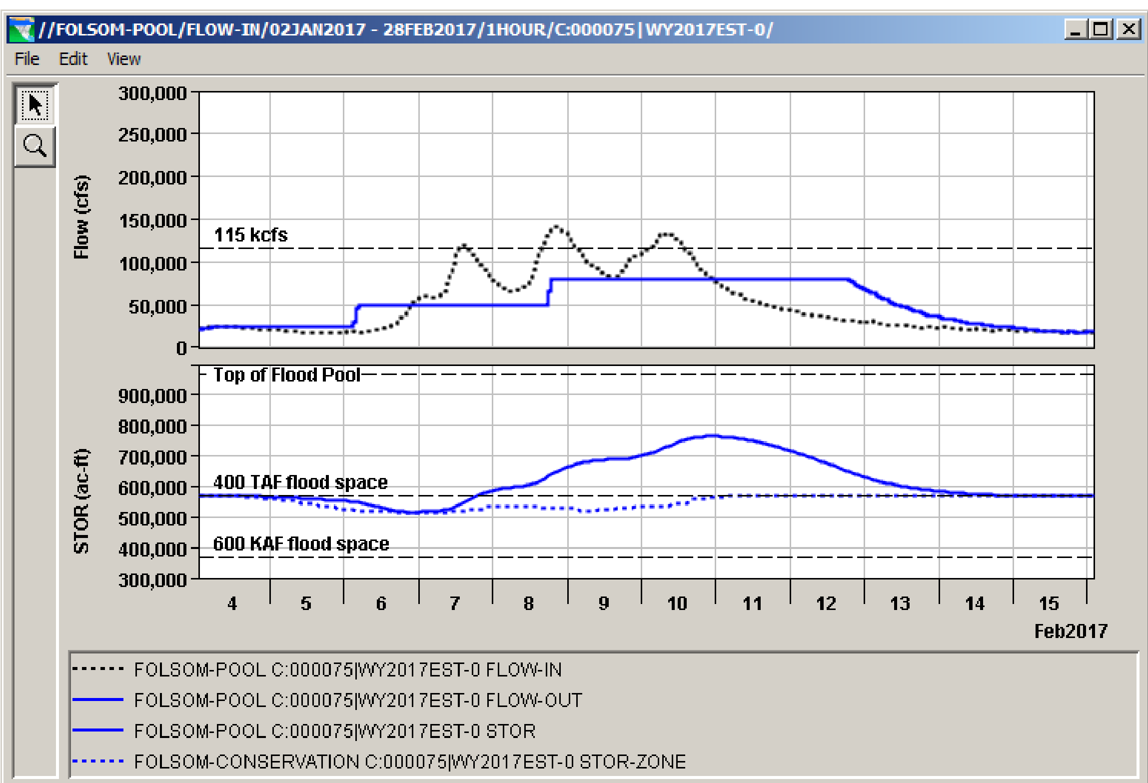


Figure 6-12: 08 Feb. 2017 Event – Forecast-based Operation with 75 Percent NEP Volumes

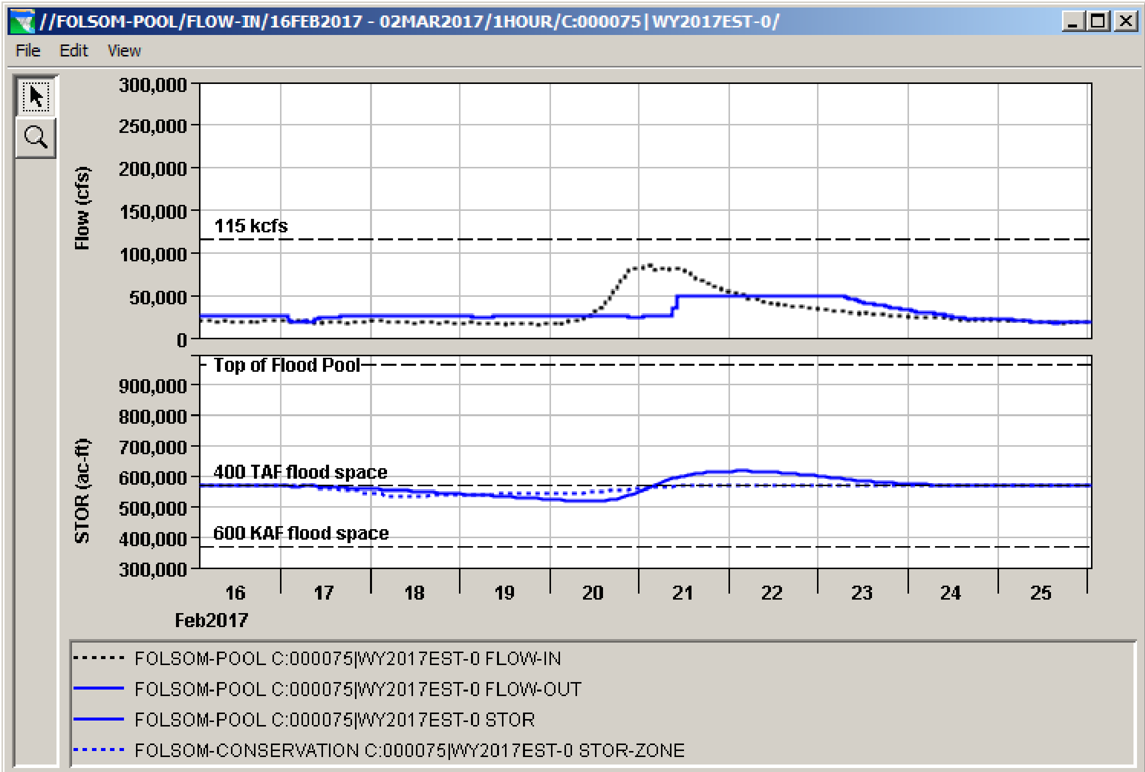


Figure 6-13: 21 Feb. 2017 Event – Forecast-based Operation with 75 Percent NEP Volumes

6.6 Regulated Peak Flow-Frequency Curves

The procedure for developing analytical regulated peak flow-frequency curves is described in Appendix E. That procedure, which develops a single curve from four pattern-specific curves, was applied to develop curves for the Existing Interim, Alternative 1, and Alternative 2 operations. An Existing Corps curve was also developed, but only the 1986 pattern was used in its development. Graphical flow-frequency curves were also developed to better define the left (more frequent) portion of the regulated curve. Graphical curves were constructed by extracting annual maximum peak outflows from the simulated POR for each operation, and assigning probabilities using Weibull plotting positions. A composite curve was then developed for each operation by overlaying the analytical and graphical curves and adopting the graphical curve when lower than the analytical curve. This was done because winter synthetic event starting storage conditions at Folsom Lake and headwater reservoirs reflected by analytical curves are appropriate for rare events but result in overestimates of releases for more common events. The resulting composite curves are displayed in Figure 6-14.

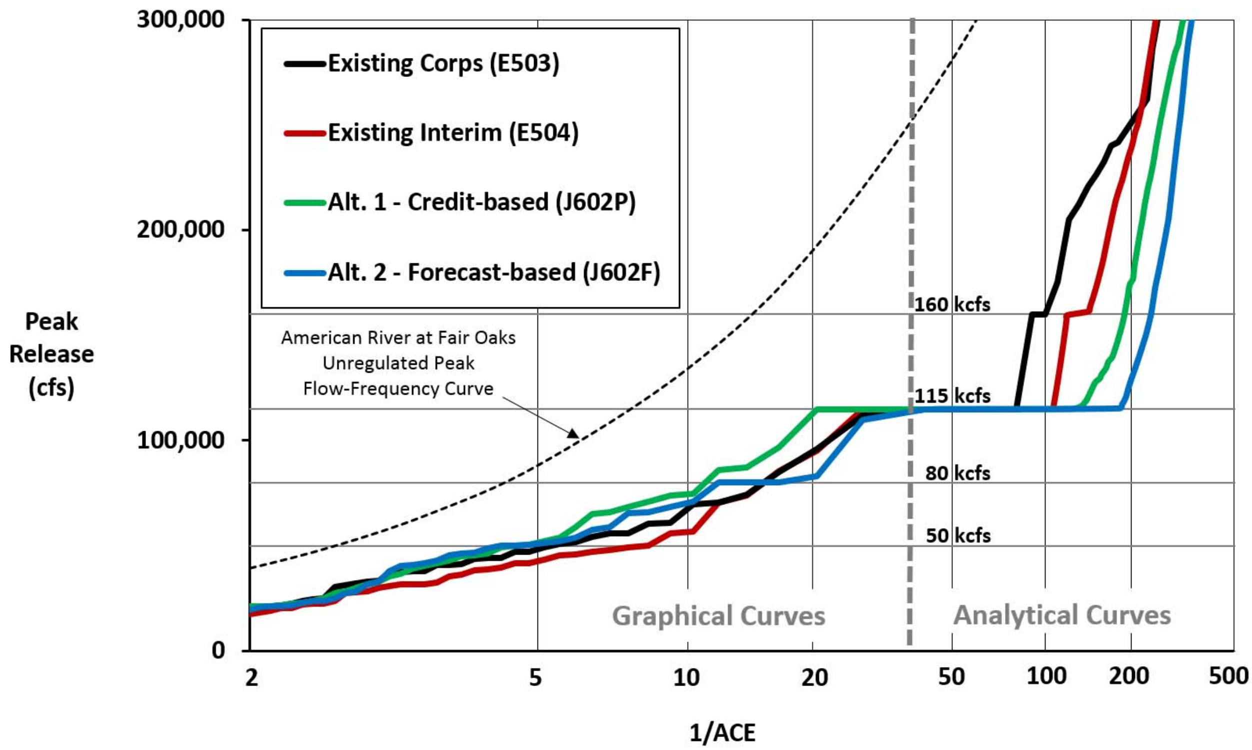


Figure 6-14: Regulated Peak Flow-Frequency Curves

The regulated peak flow-frequency curves in Figure 6-14 were developed for the purpose of comparing FRM operations, and as such reflect operational rules as implemented in the HEC-ResSim models. Sources of uncertainty about the curves should be considered before statements of performance are adopted, or hydrographs adopted based on these curves for use in other studies.

Table 6-23 lists corresponding flow values for selected probabilities, presented with accuracy of three significant figures. Table 6-24 lists return periods of largest events that can be successfully managed without exceeding peak releases of 115 kcfs and 160 kcfs.

Table 6-23: Comparison of Regulated Peak Flow ACE

Annual Chance Exceedence (ACE) (1)	Existing Interim Operation (E504) Flow (kcfs) (2)	Alternative 1 Credit-based Operation (J603P) Flow (kcfs) (3)	Alternative 2 Forecast-based Operation (J602F) Flow (kcfs) (4)
1/10	56.7	74.5	70.4
1/25	109	115	102
1/50	111	115	115
1/100	115	115	115
1/200	238	175	129
1/500	460	406	375

Table 6-24: ACE of Largest Events Passed at 115 kcfs and 160 kcfs

	Operation		
Peak Outflow Threshold (kcfs) (1)	Existing Interim (E504) (2)	Alternative 1 - Credit-based (J603P) (3)	Alternative 2 - Forecast-based (J602F) (4)
115	1/107	1/133	1/182
160	1/140	1/189	1/237

6.7 Alternative 2 – Uncertainty in Regulated Peak Flow-Frequency Curve

This section provides information about uncertainty associated with the peak regulated flow-frequency curve for Alternative 2, the forecast-based operation. Sources of uncertainty considered here are: 1) length of hydrologic record, 2) spatial and temporal distribution (pattern) of event, and 3) forecast information.

6.7.1 Uncertainty due to Hydrologic Record Length

Figure 6-15 shows the median flow frequency curve as a solid line, which is the same as the blue curve in Figure 6-14. Annual maximum values from the period of record (WY 1922-2002) were plotted using median plotting positions to define the graphical portion of the curve. Confidence limits reflecting uncertainty due to record length are shown in Figure 6-15 as dashed and dotted lines. 10 and 90 percent confidence limits were computed for the graphical portion of the regulated curve (probabilities more common than about ACE=1/40). Software program HEC-SSP, version 2.1, was used to implement the ordered statistics method described in ETL 1110-2-

537 to compute the 10 and 90 percent confidence limits from 21 flow-probability pairs interpolated from the graphical curve. The 10 percent confidence curve (upper envelope) was capped at 115 kcfs, as this release was not exceeded during the period of record. 10, 30, 70 and 90 percent confidence limits were computed for the analytical portion of the regulated curve (probabilities more rare than about ACE=1/40). The record length from which annual unregulated maxima flows were extracted to develop unregulated flow statistics was 107 years (WY 1905-2011). Confidence limits for the 48-hour unregulated flow-frequency curve were computed using equations 9-2 through 9-6 in Bulletin 17B. The 48-hour unregulated-regulated transform was then used to obtain the corresponding regulated frequency curve for each confidence level.

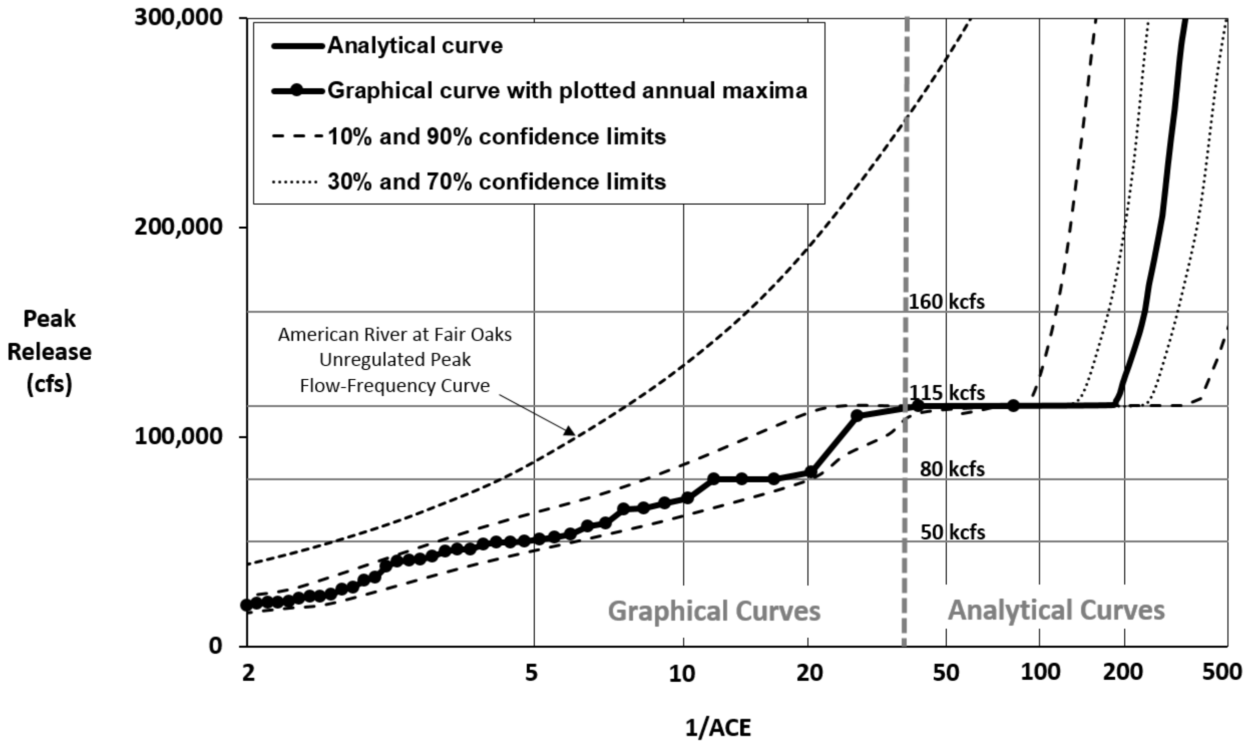


Figure 6-15: Alternative 2 – Uncertainty due to Hydrologic Record Length

6.7.2 Uncertainty due to Event Pattern and Starting Storage

The development of regulated peak-flow frequency curves from synthetic event routings is detailed in Appendix E. An event pattern-specific curve is developed for each of four event patterns, which are then weighted by probability to obtain the final curve. Figure 6-16 shows the four pattern-specific curves and the final weighted curve for the forecast-based operation. The weighted curve (black line) provides the best estimate of peak flow for selected probability. Variations from the black line indicated by the event-specific curves reflect variability in peak release due to event pattern. The graphical curve derived from the period of record annual maxima is shown for reference. The graphical curve is a better indicator of peak release than the synthetic event curves for 1/ACE less than approximately 40. This is because the period of record simulation provides a more realistic representation of starting storage conditions for common events. The synthetic event curve, for 1/ACE less than approximately 40, can be used as an estimate of the upper confidence limit reflecting high starting storage conditions. An equivalent set of synthetic event simulations could be configured using "low" starting storage conditions to obtain a corresponding lower confidence limit.

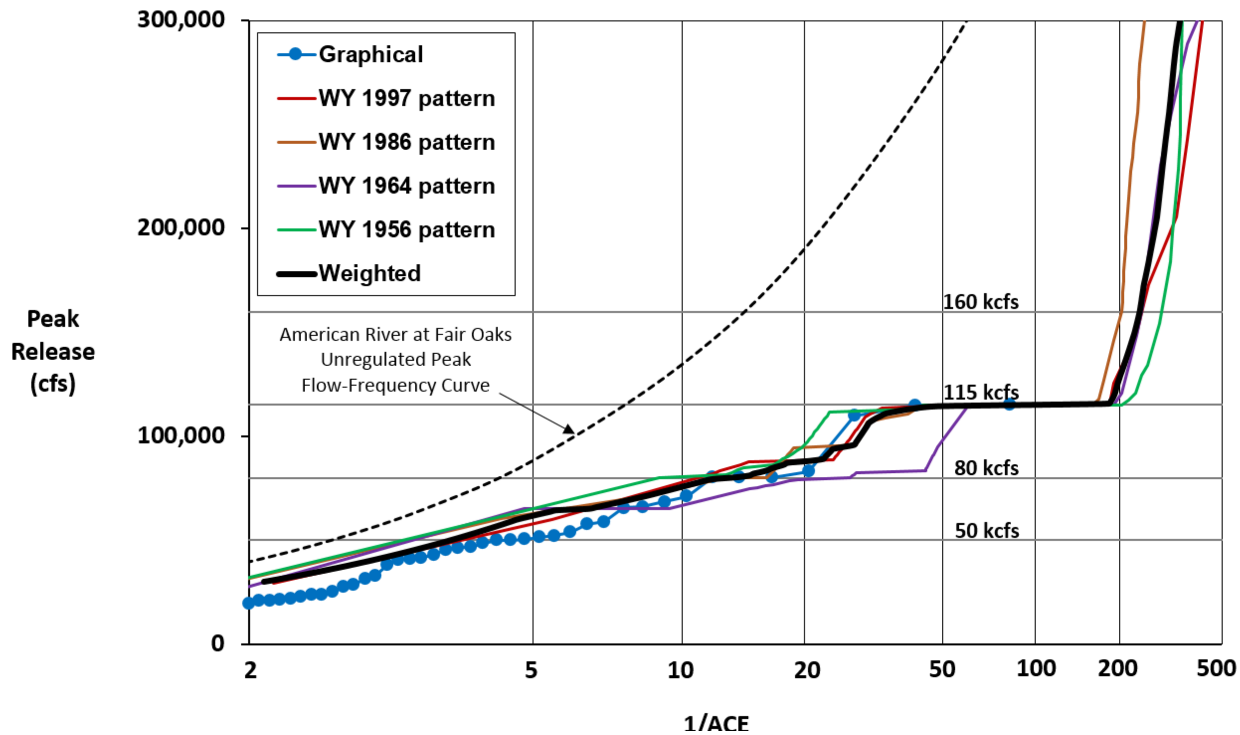


Figure 6-16: Alternative 2 – Uncertainty due to Event Temporal Pattern

6.7.3 Uncertainty due to Inflow Volume Forecast

Figure 6-7 shows for scaled versions of the 1986 and 1997 events and hindcasts, that 115 kcfs peak release will not be exceeded for both ACE=1/100 and 1/200 events when 75 percent NEP inflow volumes are used operationally. Figure 6-7 ACE values are pattern-specific, and do not reflect weighting of four event patterns used to obtain the regulated frequency curve shown in Figure 6-14. From the regulated frequency curve in Figure 6-14, the ACE=1/200 peak release is 129 kcfs, which corresponds to pattern-specific ACE values of 1/204 and 1/191 for the 1986 and 1997 event patterns respectively. Future development of scaled event/hindcast datasets will improve the estimate of uncertainty about the regulated frequency curve due to forecast uncertainty. Until those datasets become available, reasonable estimates of the uncertainty distributions for values of ACE=1/100 and 1/200 are given by Figure 6-5.

6.7.4 Uncertainty due to Climate Change

This topic is discussed in Section 7.8.5 of the following chapter.

7 Description of Study Hydrology

7.1 Overview

This chapter details the development of the hydrology used to support the Manual Update. The hydrological analyses fall into two broad categories: 1) flow frequency curves created for the American River at Fair Oaks using methods recommended in EM 1110-2-1420, and 2) hydrographs created for use in the HEC-ResSim models. Section 7.2 describes how procedures put forth in the Interagency Advisory Committee on Water Data in their Bulletin 17B (USGS, 1982) were applied to annual and seasonal maxima to create volume-duration curves, which establish the probability that a given average flow will be equaled or exceeded. The frequency curves were foundational to creating specific frequency inflow hydrographs representing various seasons of the water year. Section 7.3 describes the development of synthetic frequency hydrographs needed to evaluate, test, and improve the reservoir models. Section 7.4 specifies the development of a daily period of record of historic flows. The period of record flow hydrographs were needed for assessment of alternative operation plan effects on water supply, environmental and socio-economic impacts, and long-term erosion and channel stability. Section 7.5 provides a condensed version of the report which accompanied the revised PMF in 2001 (Corps, 2001).

The largest and most extreme floods that are experienced in the Central Valley region result from an atmospheric river event (AR). The earth's jet stream (westerlies) captures moisture from the surface of the Pacific Ocean and carries it eastward towards California. An AR occurs when a cold air front (normally heading south from the artic region) converges with a stream of moist air carried by the jet stream. AR events create large amounts of precipitation when the moisture laden air is cooled and condensed as it is pushed upward and over the Sierra Mountain Range. At the elevations in the watershed where the air temperature is below freezing, falling precipitation can saturate and ripen the snowpack, thereby inducing significant melt. This combination of rainfall and melting snow produces large inflows, with unregulated peak flows that can approach or exceed several hundred thousand cfs into Folsom Lake. During the spring months, the threat of large floods declines and the water supply pool is increased to capture runoff from the spring rain and the melting of the snowpack. Snowmelt inflows have smaller peaks and are easily controlled by Folsom Dam. Snowpack melt can be predicted well in advance from a combination of snowpack measurements and the use of models. The spring inflow is captured in the water supply pool for beneficial uses. Thunderstorms in the American River watershed tend to consist of smaller isolated cells of rain that only cover a small portion of the watershed; therefore, they do not produce any significant inflow to the dam. The summer months of June through September are typically dry with little to no rainfall. Reservoirs will typically reach their lowest storage in late summer or early fall as inflow is reduced to baseflow.

Rain flood Unregulated Flow Frequency Curves: The rain flood frequency curves are intended to be based on annual maximum flows that are the direct result of rain or rain falling on the snowpack. The snowmelt component of runoff caused by the larger AR events in the Western Sierra Mountains becomes approximately 20 percent of the total runoff hydrograph, although the dominant source of runoff is still the excess rainfall that reaches the ground and is not infiltrated. This combined rainfall-excess and snowmelt is measured as inflow to the reservoirs and is used to derive rain flood frequency curves for high elevation watersheds on the western slope of the Sierra Mountains.

The rainy season in California starts in October and typically lasts through April or May. The historically largest floods occur between December and early March when the largest AR events move westward across the Pacific Ocean. Rain flood events are a different phenomenon than spring snowmelt events. Spring snowmelt runoff occurs over many months (typically late March through July) and is driven by the more gradual and continual melting of the mountain snowpack due to warmer air temperatures. Dr. Leo Beard (one of the original authors of Bulletin 17B) was a proponent of removing snowmelt runoff from rain flood frequency curves for watersheds on the western slopes of the Sierra Mountains due to homogeneity concerns. He spent part of his career working in the Corps Sacramento District. It may not be intuitive, but including spring snowmelt events in annual maximum flow frequency curves for Sierra Mountain watersheds can actually lower the flood quantile values for rare floods when compared to a curve that is derived exclusively from rain or rain-on-snow type events. The spring snowmelt events lift up the lower tail of the frequency curve (usually filling in the drier water years in the period of record), which results in a lower standard deviation for the statistics. A lower standard deviation reduces the slope of the curve, which lowers the size of rare floods.

Runoff that is dominantly the result of warming of the snowpack during the spring months is intentionally removed from the rain flood annual maximums to create a more homogenous dataset. The homogenous dataset is used to produce unregulated frequency curves that correctly estimate the runoff potential from this specific type of flood event, which is dominant on the American River. Rain flood events typically occur during the months of October through March, although they can occur outside of this time window. For this study, the family of unregulated rain flood frequency curves for the American River at Fair Oaks are used to define frequency-based volumes that can be used to create hypothetical flood hydrographs of a specific probability, which are needed for assessing the flood damage reduction capabilities of each operation developed for Folsom Dam modeling. For each hypothetical flood produced, one of the curves (i.e., the one adopted as critical duration, such as 3-day) will be used to balance (i.e., scale) the unregulated hydrograph. These floods represent the best estimate of the unregulated inflow potential during the winter drawdown period, when the maximum amount of flood control space is needed in the American River watershed to protect the downstream community from floods. The maximum drawdown period can vary based on the WCD being utilized, but extends from 1 December to the end of February in the 2004 Reclamation/SAFCA diagram, which is in use currently.

<u>Seasonal Unregulated Flow Frequency Curves</u>: Seasonal frequency curves are used to estimate and develop hypothetical frequency-based floods during specific months of the year that lie outside of the maximum drawdown period. These floods are needed to assess the amount of flood control space needed in the reservoir for: 1) the fall drawdown period when the storm potential is increasing and 2) the spring refill period when storm potential is decreasing, the amount of flood control space is reduced, and the water supply storage is increased to capture spring runoff, including snowmelt, for beneficial uses.

Period of Record Inflows: A period of record of daily unregulated flows was derived for the American River Watershed in order to analyze cumulative impacts from day to day operations (specifically environmental and socio-economic impacts) for each reservoir operation plan analyzed in this study. The period of record flows were routed through reservoir models for the above purpose.

PMF: The PMF is used for design of the reservoir spillway(s). For the American River watershed, the Probable Maximum Precipitation (PMP), that covers the period of December through the end of March is used to define the "all-season PMF." This flood hydrograph was used to design the new Folsom Dam spillway. This hydrograph is also used to test the ESRD of the alternative operation plans in order to ensure the event can be routed through the reservoir while keeping the maximum water surface from exceeding 3 feet below the top of the dam.

Seasonal PMFs: Corps reservoirs with a single ungated spillway are designed to be able to pass the all-season PMF without overtopping the dam. For Corps reservoirs with gated spillways and seasonal rule curves like Folsom Dam, the ESRD should be tested for the PMF potential that exists for each month of the year. A "seasonal PMF" defines the PMF that could potentially occur outside of the all-season PMF period. Guidance on seasonal PMP is defined in HMR 59. For this study, seasonal PMF flood hydrographs for the months of April, May, and June were developed and used to test the ESRD of the alternative operation plans to ensure dam safety during all times of the year.

7.2 Frequency Analysis of Unregulated Flows

As part of the Manual Update, flow frequency curves were derived for the American River at Fair Oaks. Annual flow frequency curves provide an estimate of the probability, or ACE, of flood volumes and peak flows, which can then be used to assess the sufficiency of the space reserved for flood protection. However, since the majority of annual maxima fitted to the Log Pearson Type III (LPIII) distribution come from the months of December through March, the curves overestimate flood volumes during the spring for a given ACE. An analysis of flood flows during the months of the refill (spring) periods provides estimates of target volumes that might be expected when the reservoir has less than the wintertime flood space of 600,000 acre-feet or system-wide equivalent[3] available. Consequently, a seasonal flow frequency analysis for the months of March, April, and May was conducted in addition to the annual flow frequency analysis.

7.2.1 Annual Maximum Flow Frequency Analysis

A flow frequency analysis was performed on 107 years (1905-2011) of continuous unregulated flow data for the American River at Fair Oaks (Corps, 2011). Daily unregulated flows for 1905-1997 were taken from the record developed for the Corps' *Sacramento and San Joaquin River Basins Comprehensive Study* (Comprehensive Study). The record was extended through water year 2011 by calculating unregulated flows from the gaged record.[4]

[3] Storage at Folsom Lake in addition to space available in French Meadows, Hell Hole, and Union Valley reservoirs, along with potential losses to groundwater as indicated by the basin wetness parameter.

[4] Daily unregulated flows were estimated by taking the change in storage at each of five upstream reservoirs (French Meadows, Hell Hole, Loon Lake, Union Valley, and Ice House) and adding them to

Several new approaches were taken for this update to the American River curves:

1. Reexamining the Annual Rain Flood Maxima for Water Years 1905-1997: The period of record was screened for annual maxima that could be identified as rain or rain-on-snow flood events. Events with a significant (> 20 percent) baseflow or snowmelt component, were eliminated. For particularly low-flow years (e.g., 2001), precipitation and temperature records from Blue Canyon Airport were checked to see that flows coincided with measured precipitation rather than snowmelt.

2. Incorporating a Peak Curve Augmented by Maintenance of Variance Extension: A peak frequency curve was developed to act as a bounding curve for the family of volume-duration curves. The peak record was augmented using a linear-regression technique called Maintenance of Variance Extension (MOVE.1; Hirsch, 1982). MOVE.1 parameters were calculated for the period 1905-1986 through a regression of estimated peak flows with daily unregulated annual maxima. The peak series was then extended to water year 2011 (excluding 1997 for which a calculated peak was available).

3. Incorporating the Flood of 1862 in the Peak Curve: Speculated to be the largest event from 1848 to the present day, the flood of 1862 was included as a historical event per Bulletin 17B methodology. A recent estimate of 318 kcfs for the peak flow has been published by the U.S. Geological Survey (USGS) based on a rating curve contemporary with the event (Parrett, personal correspondence). This value was incorporated along with the systematic record, and the historic period adjustment was applied to the peak curve.

4. Incorporating the USGS Peak and Duration Skew Values: Regional peak and duration skew values for use in LPIII frequency analysis came from two studies developed and published by the USGS for the state of California (USGS 2011, 2012). These values supersede those published in Bulletin 17B.

5. Censoring the 1977 Event: The low-outlier test specified in Bulletin 17B identified both 15- and 30-day values as falling below the low outlier threshold. Since the difference between the threshold for all other durations and the 1977 value was slight (Table 7-1), all were censored:

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the next day’s change in storage at Folsom Lake. The summed volume was converted to flow (cfs) and added to the gaged flow for the American River at Fair Oaks (USGS gage 11446500). Negative values were replaced by zeros.

Table 7-1: Results of the Low-Outlier Test for Each Duration and the Observed Lowest Values

Duration	Low Outlier Threshold (ft^3/s)	1977 Maximum Q (ft^3/s)	Difference Q (ft^3/s)
Peak	1,882	2,500	618
1-day	1,489	1,717	228
3-day	1,160	1,548	388
7-day	914	952	38
15-day	778	754	-24
30-day	671	662	-9

6. Adjusting Curve Statistics: As recommended in EM 1110-2-1415, the raw statistics (mean, standard deviation, and skew) were examined for potential adjustment or "smoothing." The skew values for the 1- and 2-day curves were adjusted to fit with trends observed across the durations.
7. Table 7-2 lists the final statistics for the annual curves, which are shown in Figure 7-1.

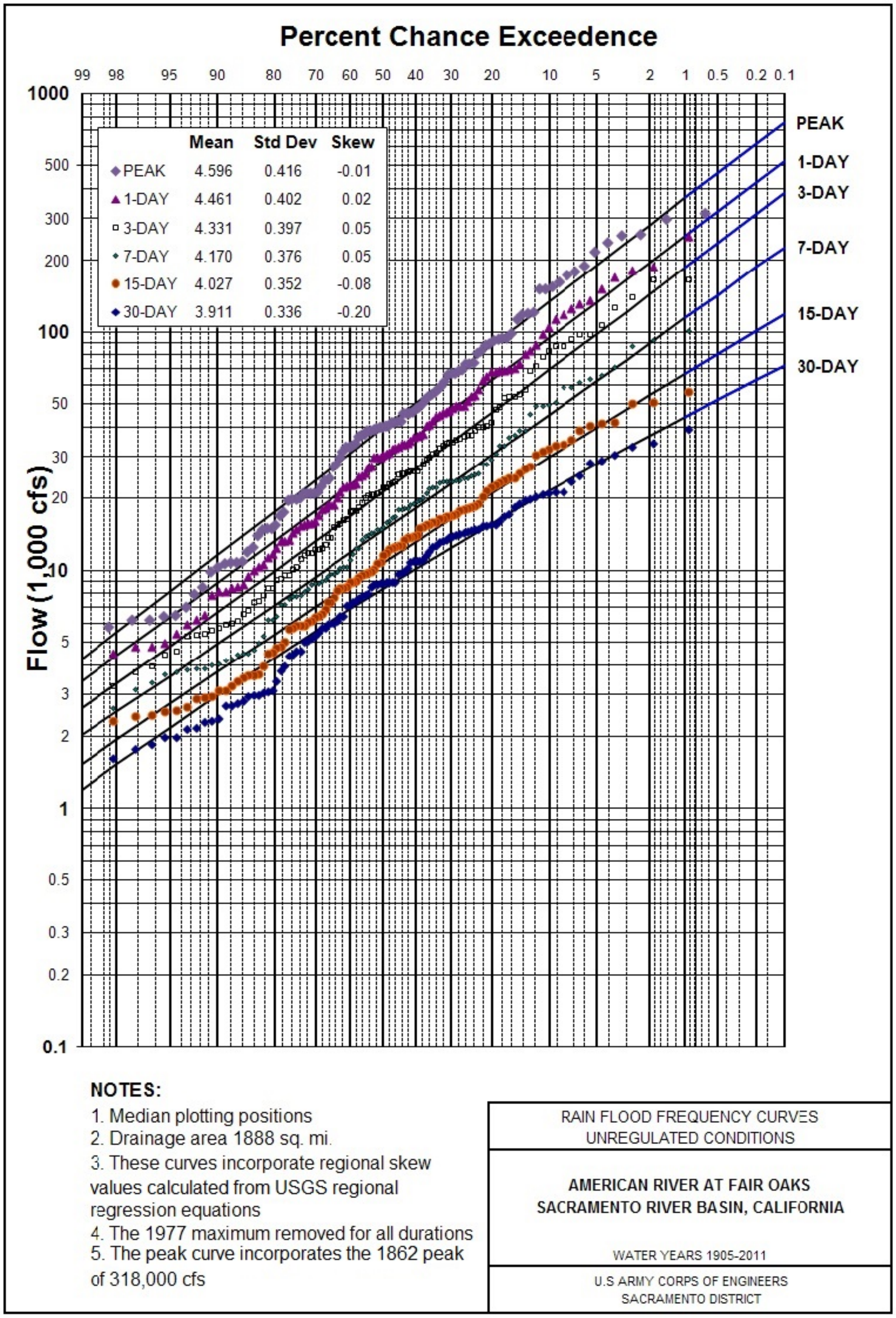

Figure 7-1: The Recommended Frequency Curve, with the 1977 Event Censored for All Durations, the 1862 Estimate Used in the Peak Curve, and the 1-day Skew Smoothed

Table 7-2: Adopted LPIII Statistics for the Family of Curves in Figure 7-1

Duration	Mean	Standard Deviation	Skew
Peak	4.596	0.416	-0.01
1-day	4.461	0.402	0.02
3-day	4.331	0.397	0.05
7-day	4.170	0.376	0.05
15-day	4.027	0.352	-0.08
30-day	3.911	0.336	-0.20

The resulting skew values for all durations are more positive than in previous studies. In frequency studies for basins in the western Sierras, skews tend to become more positive as data is added for analysis, an observation that was made in the 1997 flow frequency update (Corps, 1999). Though non-stationarity of the data has not been deemed statistically significant in a study by the National Research Council (NRC, 1999), the same report pointed out a greater number of large events for the latter part of the twentieth century. As the number of large events above the mean increases, the skew would naturally tend to become more positive.

7.2.2 Seasonal Frequency Analysis

Seasonal frequency curves were developed using the methodology detailed in Bulletin 17B for annual frequency analysis. Unlike the data used in the annual curves, the flow record for the seasonal curves was divided into sub-periods, for each of which an annual maximum series was created. Regarding subdividing the annual record, Bulletin 17B states the following:

“Separation by calendar periods in lieu of separation by events [e.g., rain and snowmelt] is not considered hydrologically reasonable unless the events in the separate periods are clearly caused by different hydrometeorologic conditions.”

However, since the goal is to evaluate the gradual increase/decrease of flood potential during periods in which the hydrometeorologic conditions are also gradually changing (i.e., from dry to wet in the fall and from rain flood to snowmelt in the spring), the division of the water year into month-based windows was necessary.

7.2.2.1 Use of Frequency Curves in Reservoir Analysis

Probabilistic analysis of large stream flows is generally focused on the development of annual maximum flow frequency curves. These curves capture the probability of the largest streamflow in any future year exceeding a given value, not specific to (conditioned on) time of year or watershed state, and thus use the maximum streamflow in each year as the assumed-IID (Independent and Identically Distributed) sample from which to estimate this probability distribution. Per Bulletin 17B Federal guidance, the LPIII distribution is recommended for use with unregulated annual maximum streamflows. In some regions, floods are limited to a certain time of year or flood season, and do not occur during other times of year. In such regions, an annual maximum flow frequency curve is still appropriate for most use (or in the case of the

American River, all annual maximum values caused by rainfloods), but it is understood that the likelihood of flooding estimated by the frequency curve applies to only the flood season, and does not represent the chance of flooding during other times of the year. The California Central Valley is one such region, with a flood season that begins in October, is most intense from December through February, and ends in March or April. Floods are not only more likely to occur in the December through February period than in the earlier and later months, but their magnitudes tend to be greater during that period. Floods are less common during the "tail" months, and those that do occur are smaller.

For most purposes in evaluating reservoir performance for FRM, these unconditional annual maximum flow frequency curves are the appropriate tool for analysis. In this case, the term "unconditional annual maximum" means there are no restrictions on the time of year from which the annual maximums are chosen. Studies focus on the function of reservoirs throughout their project life, and so evaluate their performance during all future years of any hydrologic type, and during all seasons of the year. However, when developing appropriate operation strategies for reservoirs in watersheds with specific flood seasons, there are cases for which consideration of conditional streamflow exceedence probabilities is necessary. Actual operation decisions are made with knowledge of the current season and state of the watershed, and so can consider probabilities of experiencing large floods conditioned on that information. Examining the allowable rate of refill of a reservoir that is used for both FRM and water supply is one such case that requires use of conditional probabilities. The fact that the likelihood of experiencing large streamflows decreases through the spring months allows operators to maintain a consistent likelihood of flood pool exceedence with a decreasing flood pool size through that period.

7.2.2.2 Conditional Probability Distributions

Conditional probability distributions are ones that represent probability only when certain conditions are met. Relevant examples of conditional distributions in flood risk are the likelihood of flooding in spring months as opposed to winter months, or when a watershed is fully saturated versus when it is still dry. For the Manual Update, the spring refill portion of the guide curve was evaluated with a recognition that flood probabilities decrease through the spring. Because of the decreasing probability, or conversely the decrease in magnitude of a flood with a given probability of exceedence, the flood pool maintained in April (for example) can be smaller than that maintained in February and still be sufficient to manage a flood with the same likelihood. To perform these analyses, conditional flow frequency curves were developed for the unregulated American River flows during the spring months for various durations of average flow. These curves are designed to capture the decreased frequency and magnitude of flooding as the flood season draws to an end in the spring, and allow evaluation of the reduction in the size of the flood pool during that time period.

7.2.2.3 Development and Challenges for Conditional Flow Frequency Curves

The typical method of estimating conditional probabilities is to partition the available dataset to create a sample that meets the defined condition. For example, a frequency curve for a saturated watershed would be based on floods that occurred while the watershed was saturated, and a frequency curve for the month of April would be based on floods which occurred in April. Given a dataset of annual maximum flows, these conditional curves would capture the annual likelihood of flooding during those conditions, and could be re-combined to unconditional annual probabilities given the likelihood of experiencing those conditions by using the total probability theorem. The difficulty with such conditional probability studies is that the datasets from which we evaluate annual flow frequency are limited in length, and therefore do not allow adequate sample size in partitioned datasets. An option in developing conditional distributions for operations planning is to change the focus from annual probabilities to probabilities that are limited to a given season. For example, one can evaluate the likelihood of a given magnitude of flood being exceeded during the month of April. Such an analysis would create a new data sample of the largest flow only in the month of April of each year, and so have a value in every year. The resulting probabilities would be clearly and more accurately defined. However, these probabilities would suffer from two complications. The first complication is that even given some target likelihood for exceedence of the flood pool, such as 1 percent annually, it is not clear what likelihood should be allowed in a given month, because the reservoir is in fact operated through many months, each with their own likelihood of flooding (when using this monthly approach). Therefore, allowing a likelihood of X percent in April, as well as the same likelihood in March and May, would allow a total likelihood of almost 3X percent during that three month period. A second complication is that the spring months are not so dramatically different from one another that an event that occurred in March could not have occurred in April. Therefore, the estimation of flood likelihood in April should more completely consider the events that occurred in March as part of the data sample. The chosen approach in the study addressed these complications.

7.2.2.4 Folsom Spring Frequency Curves

The analysis to develop seasonal flow frequency curves for this study can be found in reference (Corps, 2015). The approach to generating conditional spring frequency curves used for this Manual Update balanced the consideration of all appropriate events that could occur in each month with the need to limit the sample to only what is in fact possible in that month. The approach expanded the time period of each curve enough to capture a period of time longer than a single month for each defined probability, which therefore also decreases the effect of the additive nature of probability across the flood season by lengthening the applicable time period. This chosen solution pulled annual maximum flows from moving time windows through the spring season, and also included some conservatism in the specification of what events could potentially occur in a given month (or more precisely, for a given specified date).

As an example, a 3-month window of February through April was defined to estimate flood likelihood on 1 April, using the maximum flow each year that occurred during the February through April window. The resulting frequency curve estimated from this data (IID sample) estimates the probability of a flow of a given magnitude being exceeded during the period between 1 February and 30 April. This particular February through April curve was then used to represent flood probability on 1 April, which includes outcomes from the 2 months before, with greater likelihood of flooding, and 1 month after, with lesser likelihood. Figure 7-2 below shows the moving windows from which data is drawn for each curve, and the date for which the curve developed from that window is applied. Conditional spring frequency curves were developed for 1 March, 1 April, and 1 May for the annual maximum average flow durations of 1-, 2- and 3-days. Figure 7-2 through Figure 7-8 show the plotted data points and resulting estimated 3-month frequency curves separated by duration, showing the family of curves for each month (as well as the annual maximum frequency curve), then separated again by month, showing the family of curves for all durations. The annual maximum frequency curve was considered the appropriate probability estimate for the period of December through February, inclusive. The LPIII probability distribution was used for all frequency curves. The raw estimated distribution parameters were then slightly adjusted to maintain consistency of the families of curves, preventing the curves from crossing within a reasonable range of probability. Distribution statistics are shown in Table 7-3, with adjusted values in italics.

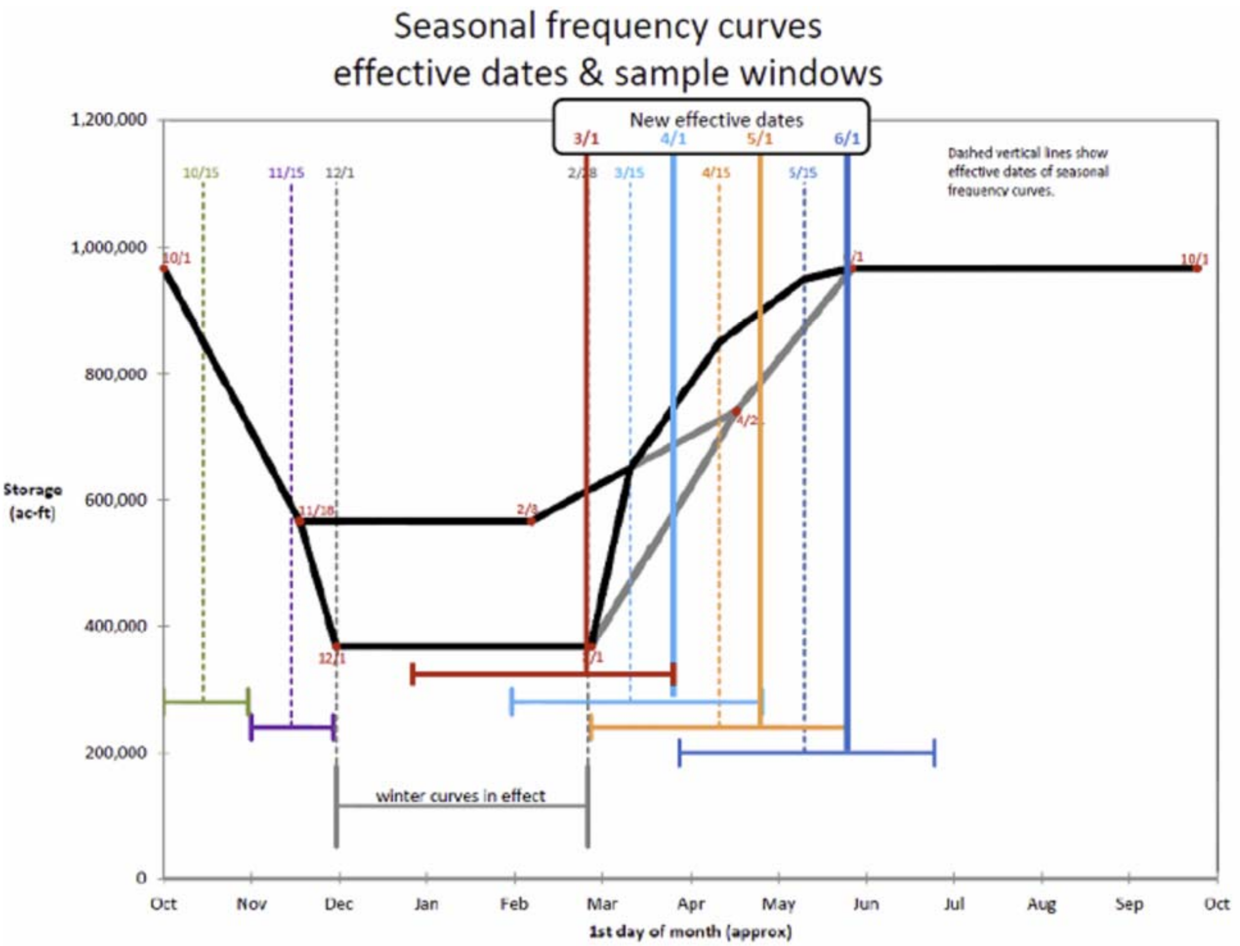

Figure 7-2: Moving Windows to Draw Data for Frequency Analysis, and the Dates for which Resulting Frequency Curves Apply

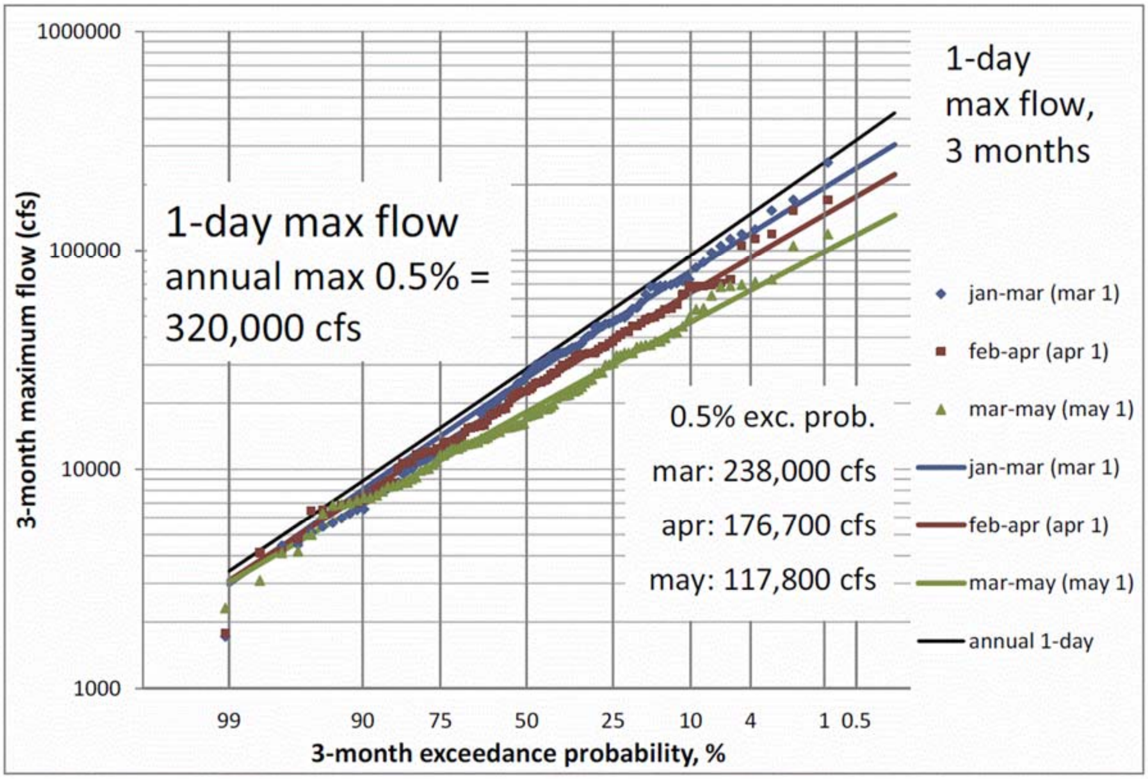

Figure 7-3: 1-day Unregulated Flow Frequency Curves

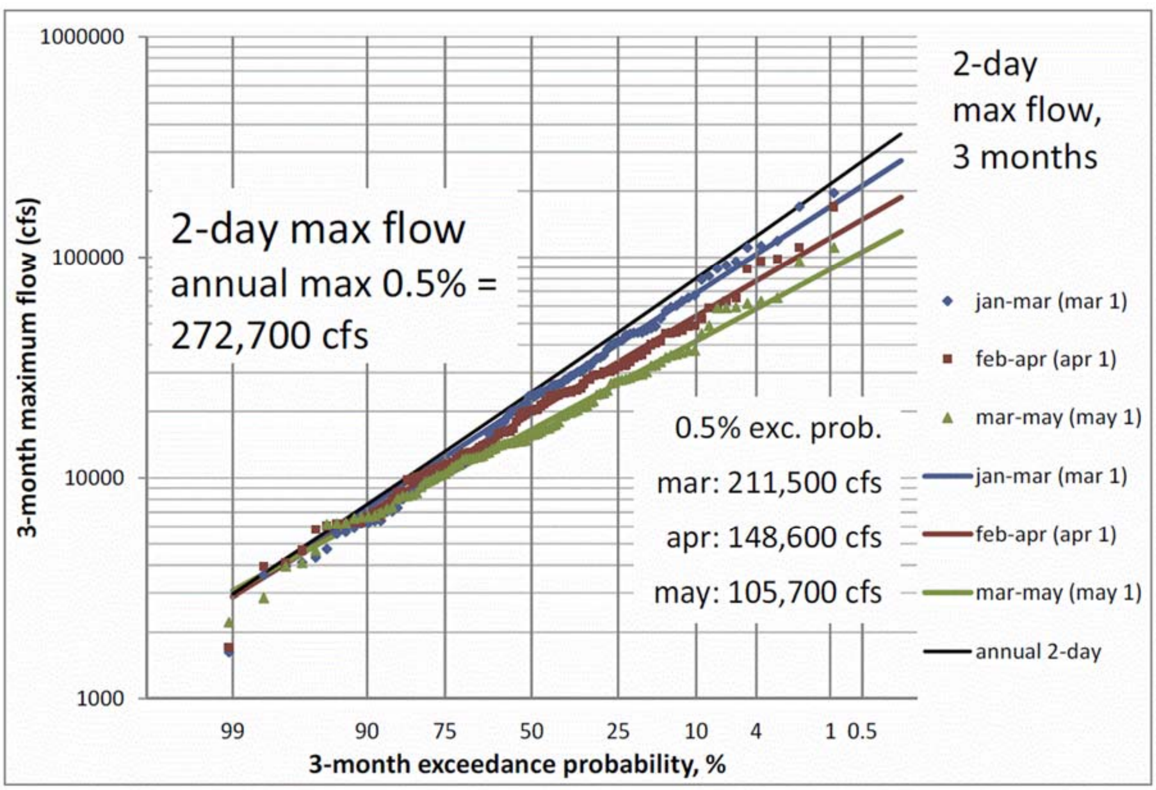

Figure 7-4: 2-day Unregulated Flow Frequency Curves

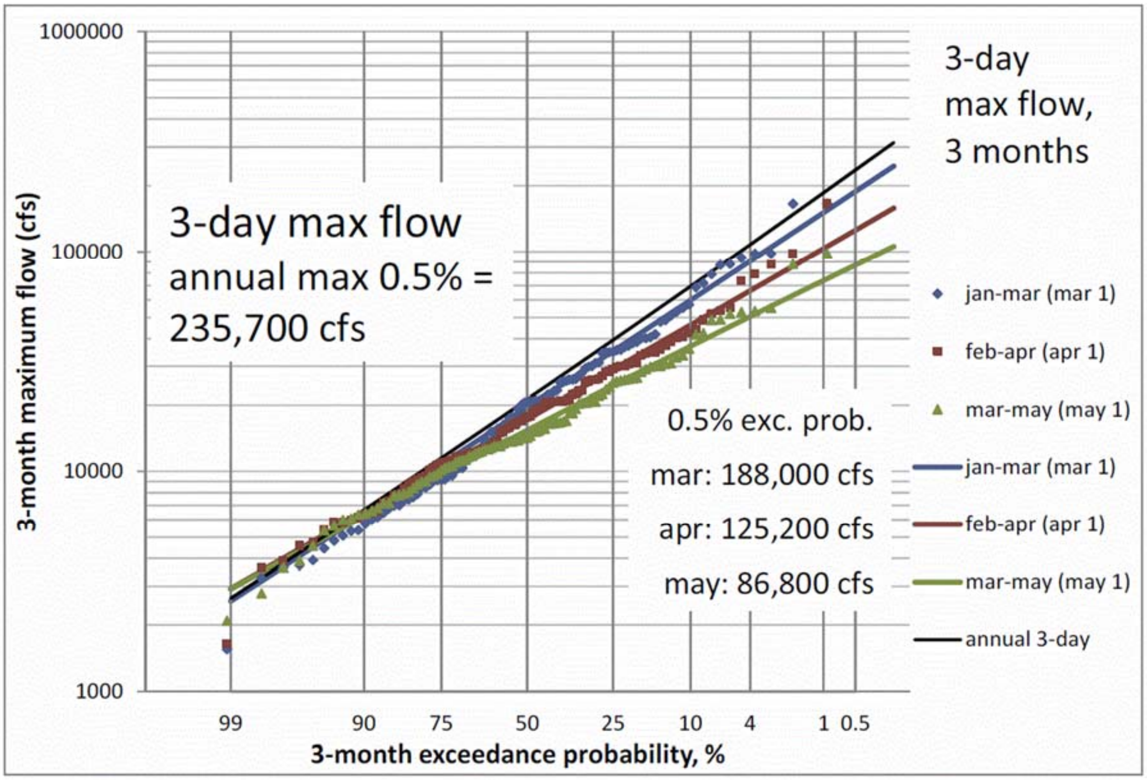

Figure 7-5: 3-day Unregulated Flow Frequency Curves

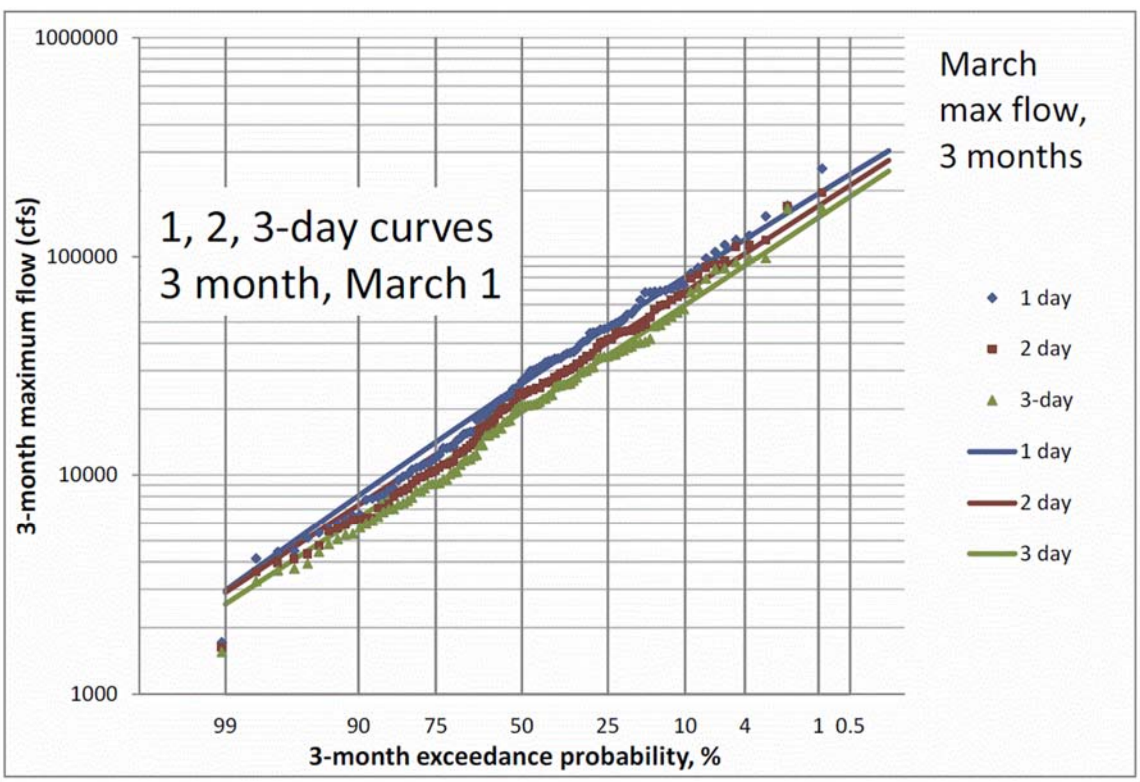

Figure 7-6: Unregulated Family of Curves for March 1st

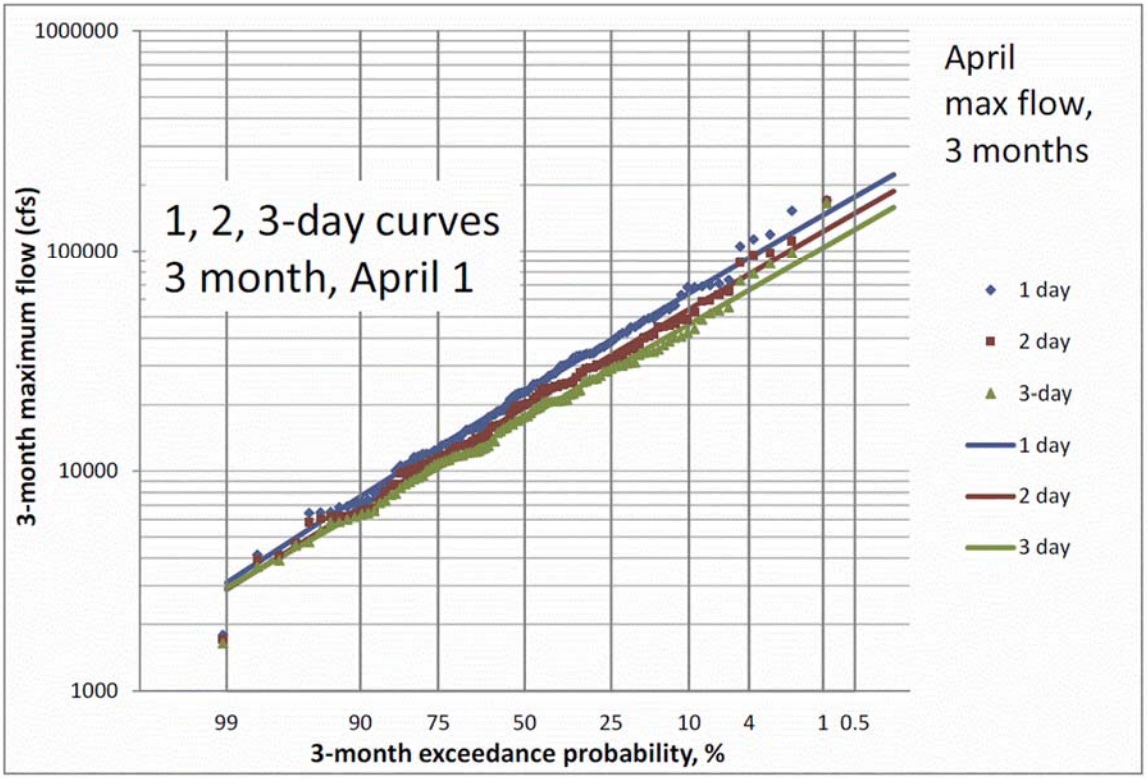

Figure 7-7: Unregulated Family of Curves for April 1st

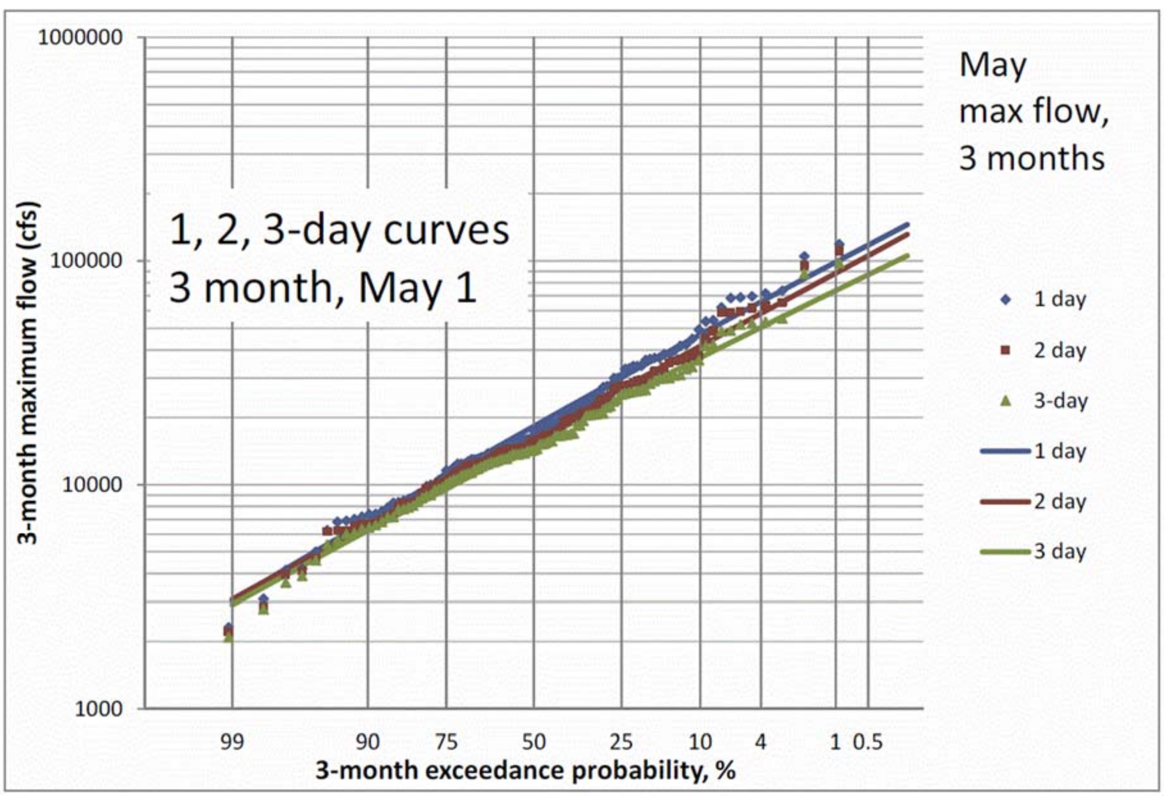

Figure 7-8: Family of Unregulated Flow Frequency Curves for May 1st

Table 7-3: Statistics for Unregulated Flow Frequency Curves

March 1	1-day	2-day	3-day
mean	*4.41*	*4.35*	*4.29*
standard deviation	*0.39*	*0.38*	*0.38*
skew	-0.104	*-0.01*	0.015
April 1			
mean	4.349	4.287	4.238
standard deviation	0.360	*0.35*	0.332
skew	-0.084	*-0.05*	*0.01*
May 1			
mean	4.258	4.218	4.184
standard deviation	0.325	0.313	0.302
skew	*-0.08*	*-0.01*	*-0.08*
Annual Maximum			
mean	4.461	4.391	4.331
standard deviation	0.402	0.400	0.397
skew	0.023	0.038	0.050

7.3 Historic Flood Event Patterns

Inflow datasets were created to evaluate how candidate operations impact all the purposes for which Folsom Dam is used. Flood managers in basins such as the American River, where rain and rain-on-snow events are the source of severe flooding, are concerned with how operational rules handle dramatic increases in inflow volume during the relatively short span (generally 3 or 4 days) of a flood event. Rule sets must be robust enough to deal with a number of potential hydrograph shapes and a wide range of volumes.

Five patterns were selected for evaluating the enhanced operational capabilities of Folsom Dam:

- Four historic floods:[5] December 1955, December 1964, February 1986, and January 1997; and
- One synthetic event: The 2001 revision of the PMF.

The historic floods are the largest four floods since the dam's completion in 1955. Importantly, the four flood hydrographs (Figure 7-9 through Figure 7-12) provide a representative sample of large events that have occurred on the American River. The PMF pattern is shown on Figure 7-13.

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[5] Historic hydrograph shapes for the 1955 and 1964 events were digitized from the Folsom Dam WCM; those for the 1986 and 1997 were based on HEC-1 reproductions created during the revised PMF study.

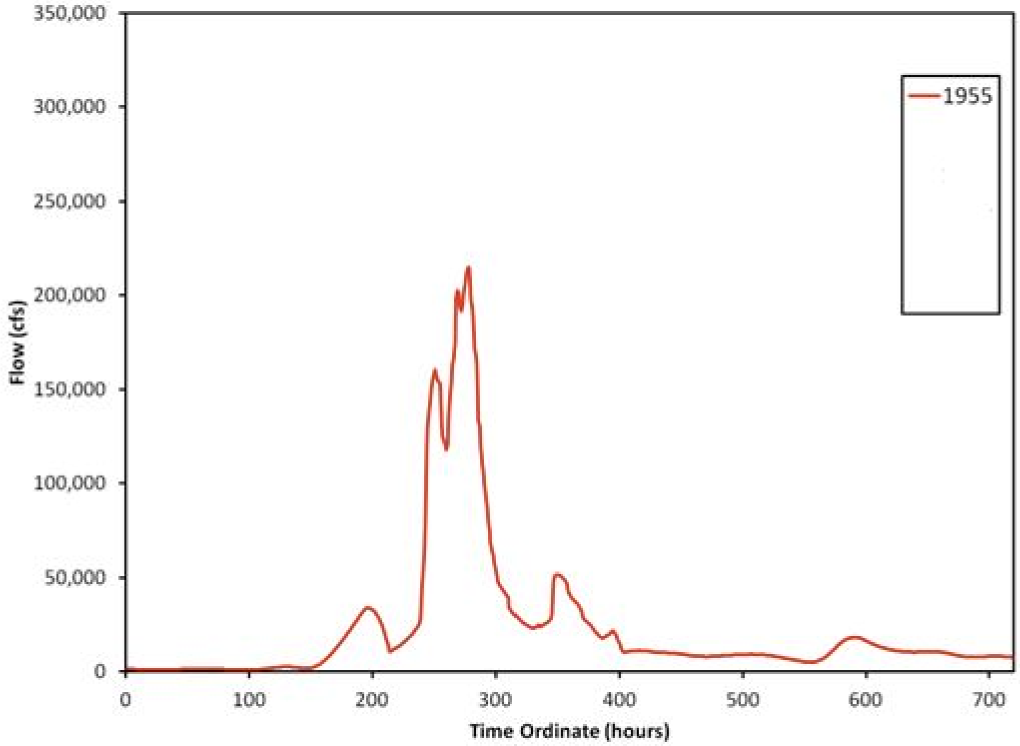

Figure 7-9: The Unregulated Inflow into Folsom Lake due to the 1955 Flood

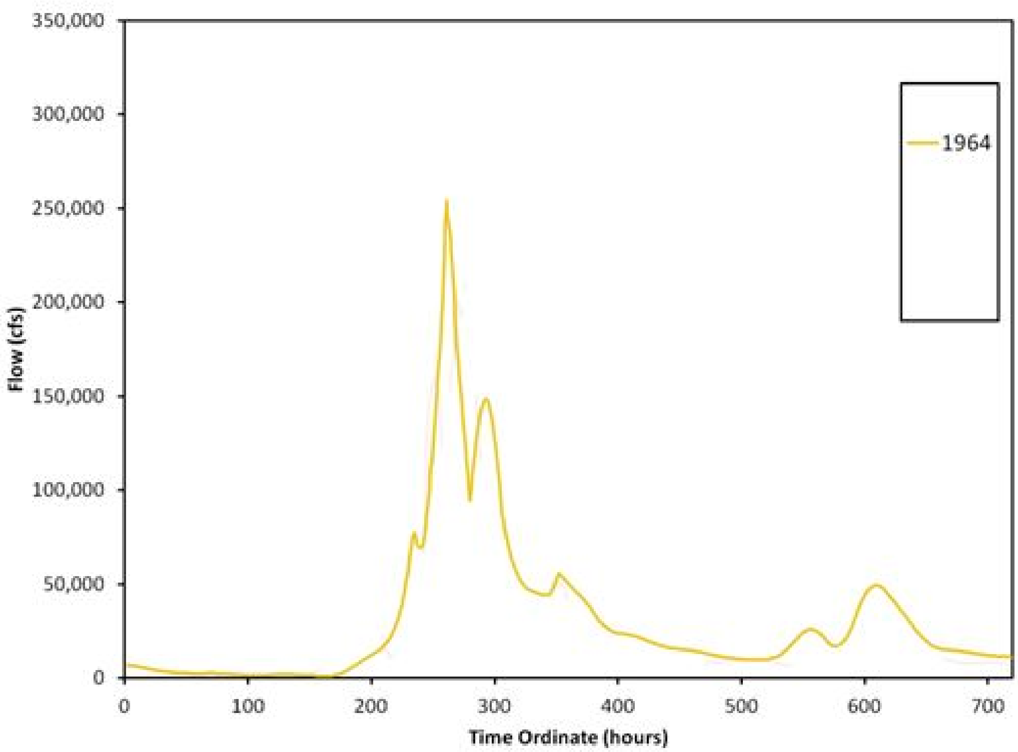

Figure 7-10: The Unregulated Inflow into Folsom Lake due to the 1964 Flood

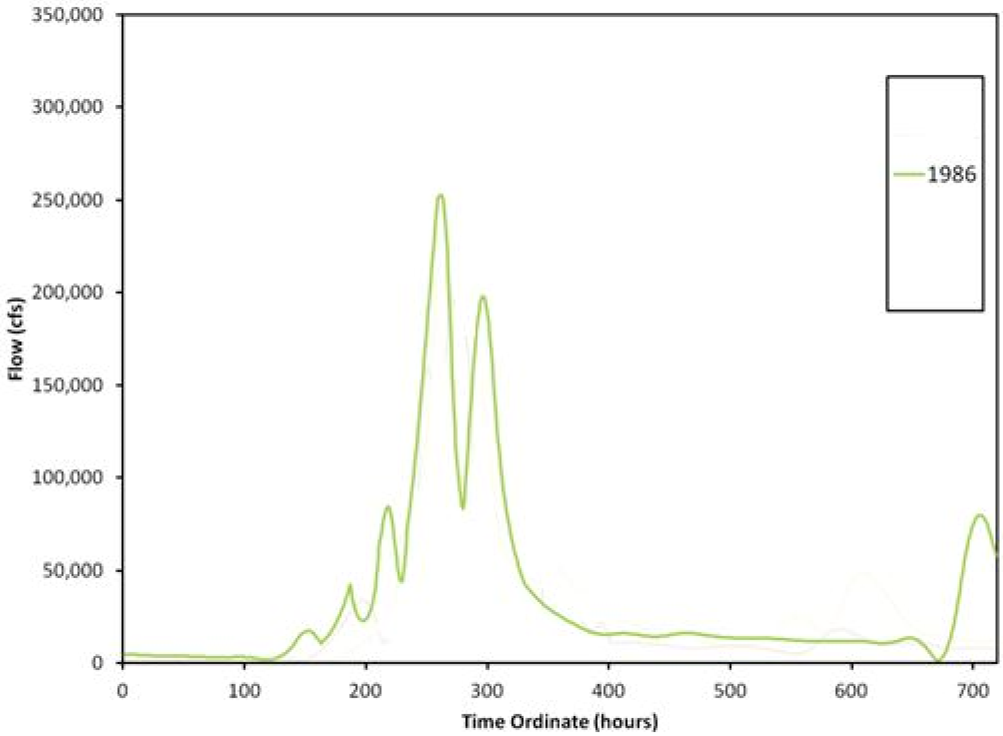

Figure 7-11: The Unregulated Inflow into Folsom Lake due to the 1986 Flood

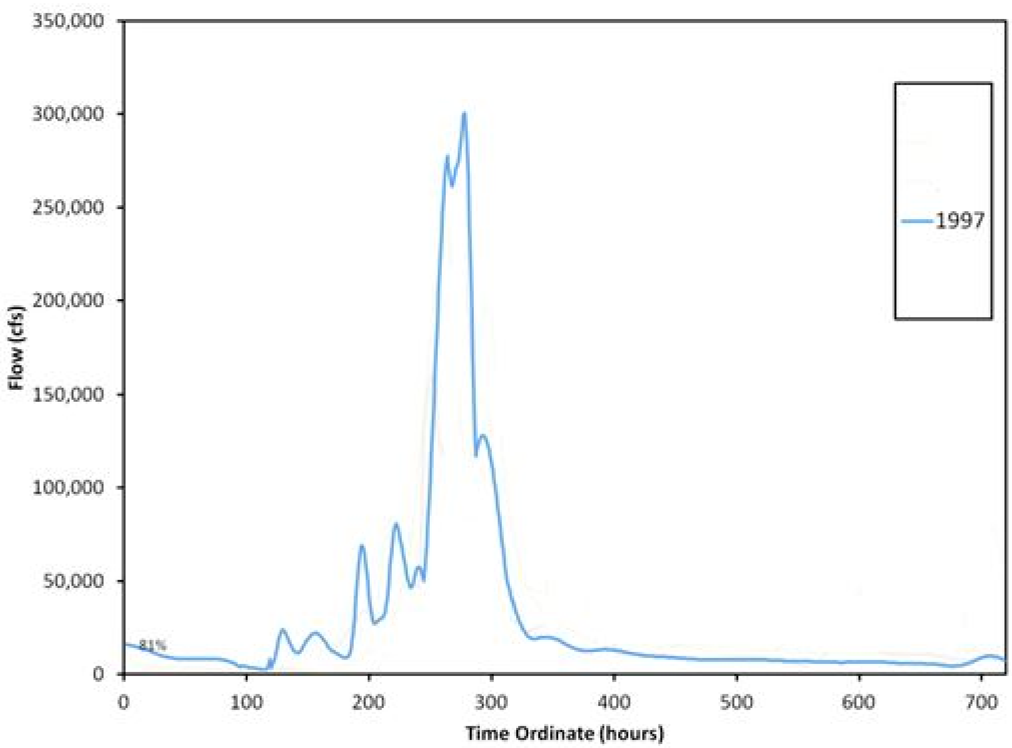

Figure 7-12: The Unregulated Inflow into Folsom Lake due to the 1997 Flood

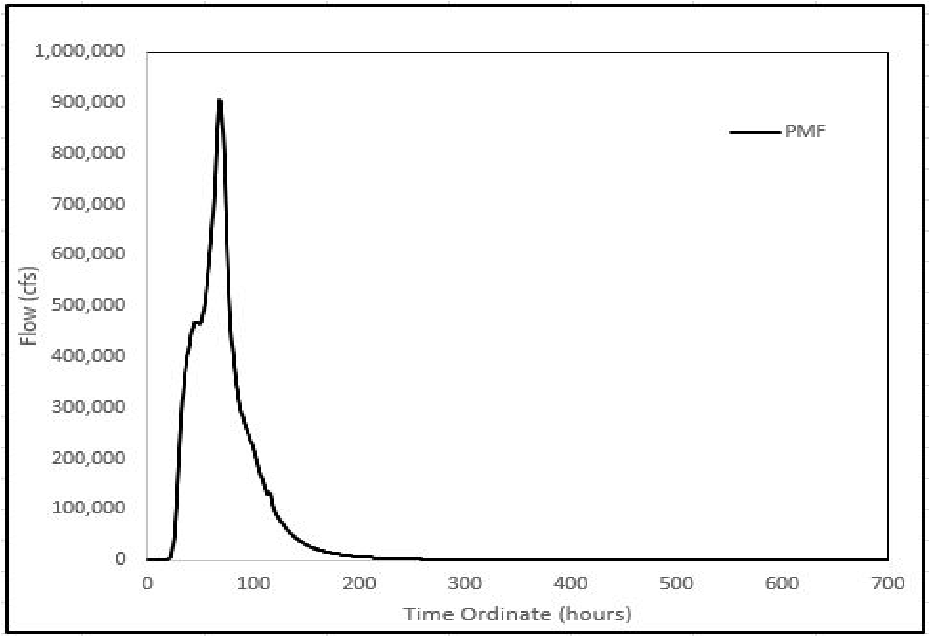

Figure 7-13: The Unregulated Inflow into Folsom Lake due to the PMF Flood

For example, the 1964 (Figure 7-10) and 1986 (Figure 7-11) events have significant second waves, while the 1955 (Figure 7-9) and 1997 (Figure 7-12) events have sharply rising single waves in which the flood volume is concentrated over 1 or 2 days. It is also apparent that the overall ratio of hydrograph height to width does not vary greatly.

For testing the models, two hypothetical events were also selected: the PMF and the SPF. However, these shapes will not be included in the final regulated frequency curve.

Flood-producing runoff occurs during the months of October through April and is most extreme during the months of November through March.[6] The following descriptions of the historic events give a sense of how varied the conditions of rain-on-snow events can be in the American River Basin.

1. December 1955: During the 2-week period beginning 15 December, severe storms and floods occurred throughout an area of approximately 100,000 square miles of northern and central California. In many localities, the floods were the greatest of record. As the result of the great quantities of rainfall, together with an appreciable contribution from snowmelt at high elevations, minor flood peaks occurred on 19-20 December on streams in northwestern California and major peaks on 22-24 December on all streams of the region, except at locations where the runoff was largely controlled by upstream reservoirs. At nearly all places, the uncontrolled peaks exceeded previous maxima (Corps, 1956).

[6] Since snowmelt alone generally does not result in flood-producing flows. No snowmelt events were used for rule operation sets for flood protection.

The flood of December 1955 led to a peak inflow into Folsom Lake of 219 kcfs. Because the dam had just been completed, the initial reservoir level was only 200,000 acre-feet (800,000 acre-feet of flood storage space) and the discharge from the dam was consequently regulated to a maximum of 71 kcfs. If the reservoir only had 400,000 acre-feet of storage available, it was calculated that the discharge would have been regulated to a maximum of 115 kcfs and maintained at that discharge for 2 days. In spite of the fact that Folsom Dam could easily control a flood of this magnitude under existing operating procedures, the December 1955 flood caused concern over the flood protection measures for the LAR. This was because the storm that caused the December 1955 flood was more severe than the December 1937 storm that was used as the design basis for Folsom Dam. Another reason appears to have been the fact that the flood occurred so soon after completion of the dam (Williams, 1973).

2. <u>December 1964</u>: On the weekend of 19-20 December 1964, a combination of a warm mass of moist Pacific air, a flow of cold air from a low pressure trough off the coast, and a strong westerly current created optimum conditions for heavy precipitation. Rainfall in the American River Basin created high stages on most tributaries above Folsom Lake. Hell Hole Dam, a small sloping-core rock-fill structure being built on the Middle Fork, failed under the stress of the flood water. Approximately 30,000 acre-feet from the partially constructed dam was added to the peak inflow of 280 kcfs into Folsom Lake. Storage in Folsom Lake increased 322,000 acre-feet to a maximum of 899,000 acre-feet on 23 December and controlled releases were increased to a peak rate of 115 kcfs and maintained for approximately 50 hours (Corps, 1987a).

3. <u>February 1986</u>: The storms of February 1986 severely affected northern California and northwestern Nevada. The heaviest precipitation occurred 200 miles north to 100 miles south of a line from San Francisco to Sacramento to Lake Tahoe. Over much of this area, the precipitation ranged between 100 and 350 percent of normal February precipitation.

   In the American River Basin, the heavy rains began on 12 February. With continued rains and storm runoff, water levels behind the Auburn cofferdam rose rapidly. On the afternoon of 18 February, the Auburn cofferdam failed. With the failure of the cofferdam, Folsom Lake experienced a peak inflow of 900 kcfs. The releases from Folsom Dam at this time were increased to 125 kcfs. On 19 February, storage in Folsom Lake reached a high of 1,028,000 acre-feet. Releases were increased to a maximum of 130 kcfs. Releases at or above 115 kcfs were maintained for approximately 64 hours during the storm (Corps, 1987a).

4. <u>January 1997</u>: The flood of 1997 on New Year’s Day generated the flood of record for many northern California river basins. Pre-storm conditions prepared the American River Basin for very efficient runoff production. During the first 3 weeks in December, enough precipitation fell to saturate the ground and cover 70 percent of the basin in snow,

ranging from 2 inches at Nevada City (2,700 feet) to more than 45 inches at Lake Spaulding (5,155 feet). Snow depth and water content continued to increase up to the highest elevations (+10,000 feet). Freezing conditions were experienced at most elevations.

Over 70 percent of the rainfall fell in the four-day period of 30 December 1996 to 2 January 1997. The average rainfall depth over the basin during the 10-day period was 17.2 inches, of which 11.8 inches fell during the most intense 4 days. The unregulated maximum 4-day runoff (due to rainfall and snowmelt) was 11.1 inches. The amount of precipitation that occurred during this period for the American River Basin, however, has been equaled or exceeded in the past. The storms that have equaled or exceeded the 1997 storm occurred in water years 1951, 1956, 1963, 1965, and 1986. Four extraordinary factors turned the same precipitation into an unprecedented runoff volume: 1) the extreme saturation of the soil, 2) snow cover, 3) the water content of the snow, and 4) warm temperatures during the heaviest precipitation periods. During the heaviest rainfall period, precipitation fell as rain in the highest elevations, melting snow as it ran off. In addition, areas in the basin may have been frozen because of the cold temperatures of preceding storms. If frozen ground conditions did exist, it would help to explain the extreme runoff experienced during the heaviest precipitation periods.

The resulting flood in the American River Basin above Folsom Lake produced the greatest recorded 1-day volume and peak since the collection of detailed runoff data began in 1905. The 1997 flood duplicated the 3-day volume of the February 1986 event, which had been the largest of record (Corps, 2001). Both events had a 3-day maximum average flow of 166 kcfs (each rounded to the nearest thousand cfs).

5. Discarded Events: Three events (floods of water years 1963, 1980, and 1982) that were routed as part of engineering work for the 1987 WCM were eliminated based on evaluation of the 3-day volumes.

    - The 1963 event was removed as being dissimilar to the other events, in being sharply peaked but of a small overall volume. While the peak remains one of the largest on record at 240 kcfs, the 3-day volume has an ACE of 1/18.
    - Similarly, the relatively common frequency (> 1/25 ACE) of the 1980 and 1982 events' 3-day volumes recommended their removal from the initial set. There were also concerns about the accuracy of the hydrographs found in the WCM for these events, since the plotted peaks were quite different from the published values.

Per Corps EM 1110-2-1415, "it is best not to multiply any one flood by a factor greater than two or three." The concern is that more common floods may have different characteristics than rarer floods. The 1963, 1980 and 1982 3-day average inflows are all less than 100 kcfs, thus requiring factors greater than 3.0 to scale them above the 1/200 ACE volumes. Therefore, these historic events were not adopted for scaling in order to derive rare hypothetical floods.

7.3.1 Adjustment of the Historical Hydrograph Volumes

The hydrograph shapes of the historic events were adjusted so that daily volumes were the same as the unregulated record for the American River at Fair Oaks. Additionally, the peak discharges for the adjusted shapes match the unregulated peak values recorded or estimated for each event.

7.3.2 Inflows for the HEC-ResSim Network

The HEC-ResSim models developed for the Manual Update include 12 nodes for which hydrographs are needed. The node inputs can be classified as one of three types discussed below.

1. Headwater Dams: Five major hydroelectric projects are accounted for in the models. These include the dams of French Meadows, Hell Hole, Loon Lake, Union Valley, and Ice House.

2. Diversions: Four diversions, three within the basin and one outside, were included in the network structure. The intra-basin diversions include those from Duncan Creek to French Meadows, the Buck-Loon diversion, and the Robbs Peak below Union Valley Dam. The Bear River Diversion, which imports water from the Bear River drainage into the American River, contributes roughly 1,000 cfs to Folsom Lake.

3. Local Flows: Gerle Creek, South Fork of the Rubicon River, and Folsom Lake

Headwater and local flows (with the exception of those for Folsom Lake) were created by scaling the total unregulated hydrograph and lagging them to adjust for travel time (roughly 5-6 hours). Scaling factors were calculated as the ratio of the maximum 3-day inflow of each headwater dam or local series to the Folsom 3-day maximum inflow (Table 7-4):

Table 7-4: The Percentage Contribution of Each Headwater Dam to the Total Folsom Lake Inflow

Inflow Location	1955*	1964	1986	1997
French Meadows	2.2	3.8	3.5	4.5
Hell Hole	6.8	9.3	5.5	10.7
Loon Lake	0.5	0.7	0.6	0.8
Union Valley	4.0	5.9	4.6	6.3
Ice House	1.0	1.5	0.7	1.8
Total Headwater	14.5	21.2	14.9	24.1
Total Local Flow	85.5	78.8	85.1	75.9
*Headwater reservoirs were not all in place during the 1955 event. The percentage contribution was estimated based on existing stream gage records for that time period.				

As shown in the table above, the contributions of the headwater inflows range from 76 to 86 percent for these major rain-on-snow events. Since the drainage area above the headwater dams comprises roughly 15 percent of the total basin area (see Figure 7-14: Drainage Basin Area, by Percent, for the Area Above and Below the Headwater Dams), the similar volume contributions for the 1955 and 1986 events are validated. The greater role of headwater inflows during the 1997 event shows the impact that a large antecedent snowpack, saturated ground conditions, and above-freezing temperatures at the highest elevations can have on runoff production.

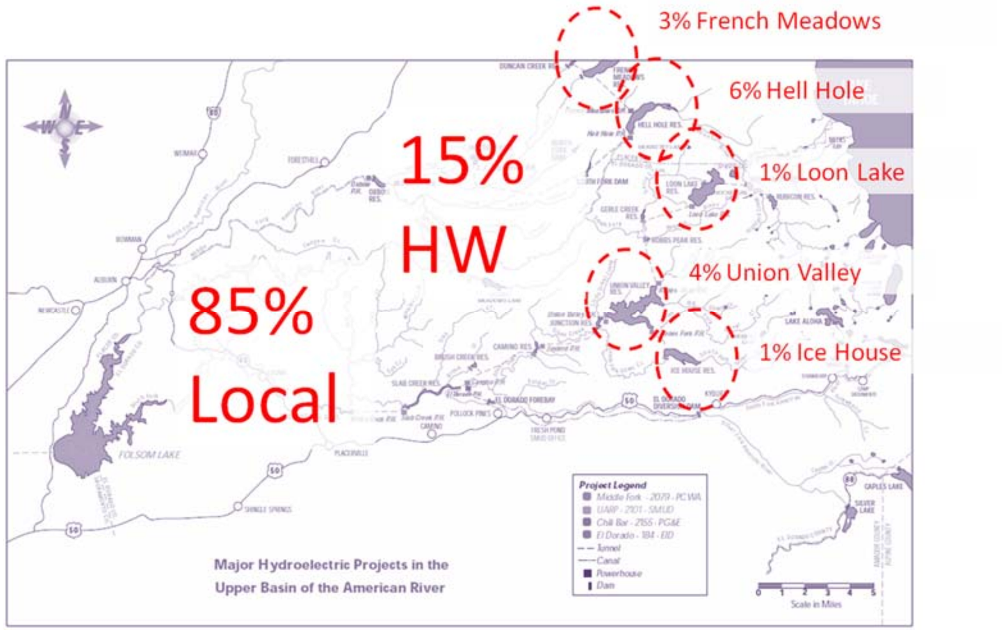

Figure 7-14: Drainage Basin Area, by Percent, for the Area Above and Below the Headwater Dams.

Diversion flows were taken from the record developed for Reclamation by CH2M Hill to support the CalSim II daily water supply model (Reclamation, 2006). These daily records were converted to smoothed hourly time-series to eliminate the step-wise transitions seen in the average daily flow record. However, total daily volumes were preserved in the hourly series. Finally, local flow into Folsom Lake was computed by subtracting all the hourly hydrographs (not including diversions from one headwater reservoir to another) from the Folsom Dam inflow hydrograph.

7.3.3 Scaling the Unregulated Hydrographs

Hydrographs were needed for testing and evaluating reservoir operations in the various models, including the alternatives. Each event hydrograph was scaled by one of 48 factors in order to create events that ranged in magnitude from 1/2 to 1/1000 ACE. For every target ACE volume, the scaling factor was the ratio of either the 2-day or 3-day maximum flow[7] to the average flow for the same duration taken from the frequency curve. Since natural hydrographs tend to have different ACE values for different durations, no attempt was made to scale the historical hydrographs so that maximum average flows for multiple durations all had the same ACE (i.e.,

[7] Based on the preliminary assessment of critical duration for each pattern (see Appendix E), the maximum 2-day average flow was used as the denominator for the scaling values applied to the 1955, 1997, and SPF patterns, while the 3-day average flow was used for the 1964, 1986, and PMF patterns.

the scaled hydrographs are not balanced). Each pattern flood was assigned a critical duration based on the 1/200 ACE event, as simulated in the J602 model. This is an important level of protection that is being targeted for the new operation plan. Once critical duration (i.e., 2- or 3-day) was assigned to a pattern flood, that duration frequency curve was used to assign a frequency to the scaled pattern hydrographs. The critical duration assigned to each pattern is shown in Table 7-5. Critical duration is defined as the volume that is most directly correlated to the peak outflow from the reservoir. The procedure to assess critical duration is outlined in Appendix E.

Table 7-5: Critical Durations

Year	1955	1964	1986	1997
Critical Duration	2 days	3 days	3 days	2 days

The percent contribution from the headwater dams and local flows into Gerle Creek and the South Fork of the Rubicon River did not change with the event size; hence, the same scaling factor used for the total hydrograph was also applied to these. However, the same historical diversion records were used for the whole range of scaled events. This choice was based on the regularity of the flow volume observed in the diversion records regardless of the water year.

7.3.4 Additional Pattern Hydrographs Utilized for Testing

Scaled Hydrographs: As the study progressed, it was determined that additional hydrograph shape sets would be useful for testing the performance of reservoir operation rules. The intention was to provide a more robust set of hydrographs to test the models. For the winter floods, the following additional flood patterns were analyzed for critical duration, and scaled to attain a range of frequencies between 1/2 to 1/500 ACE events: 1) January 1995 flood 2) March 1995 flood, and 3) December 2005 flood. Two synthetic flood patterns based on rainfall runoff modeling (the SPF and PMF) were also utilized.

Balanced Hydrographs: Using balanced hydrographs as inputs to the reservoir models is an alternative method to assess performance of a reservoir operation set. A balanced hydrograph is manipulated so that key durations of the hydrograph have the same frequency. For the main flood season, the pattern floods that were balanced include 1955, 1964, 1986, 1997, January 1995, March 1995, 2005, SPF, and PMF events. The hydrographs were balanced to the 1-, 3-, 7-, and 15-day unregulated frequency curves.

For reservoir rule testing of the spring refill period, the 1986, 1997, and March 1995 pattern floods were balanced to the 1-, 2-, and 3-day unregulated frequency curves for each month of the spring refill period. The balanced hydrographs represented an estimate of specific frequency floods conditional to the time of 1 March, 1 April, and 1 May.

7.3.5 Hydrographs for Deriving the Final Regulated Peak Flow Frequency Curve

The adopted regulated peak flow frequency curve at an index point downstream of the dam is one of the most important metrics to compare the performance of each operation, as it describes an estimate of the level of protection that is provided. In order to assess these curves, the unregulated 1955, 1964, 1986, and 1997 pattern hydrographs were scaled by various ratios and then routed through the reservoir models. The procedure to develop a regulated peak flow frequency curve from four pattern floods is described in Appendix E. The procedure is used to derive a final regulated peak flow frequency curve for the final alternatives, and ultimately, the selected plan and WCM.

7.4 Period of Record Flows

Water supply operations seek to satisfy many goals (e.g., industrial and municipal water supply, environmental mitigation, recreation) over a multi-month or multi-year period, during which a range of inflows naturally occurs. For this purpose, a dataset covering the period 1921-2002 was culled from the record developed for Reclamation for use with the CalSim II water supply model. The same HEC-ResSim models were used for both the period of records and flood event simulations; therefore, hydrographs for the same locations as enumerated for the flood events were developed. In the cases where the CalSim II record did not extend back to 1921, a record was synthesized by inserting year-long hydrographs from the observed record for the same gage. Each year was classified in terms of a scale measuring a range of hydrologic conditions from very dry to very wet, and representative hydrographs for each scale gradation were selected.

Daily records were smoothed into hourly hydrographs, while preserving total daily volume, using an algorithm developed at SPK for the *Central Valley Hydrology Study*. To ensure consistency with the flood event simulations, the hourly values from the four modeled flood events (observed historical hydrographs into Folsom Lake and the scaled hydrographs for headwater and local inflows) were spliced into the smoothed period of record.

The period of record flows were provided to HDR, Inc., which is an A-E firm contracted by Corps to evaluate water supply, environmental, and socioeconomic impacts resulting from the various reservoir alternatives. The use of the period of record inflows (or its modification thereof, if needed) is described in the environmental analysis section of this report.

The period of record inflows were run through the HEC-ResSim models and the resulting flows downstream of the dam were provided to the Corps' Hydraulic Analysis Section for long-term analysis of sediment transport and channel stability. This evaluation is described in the Hydraulic Analysis Section of this report.

7.5 Probable Maximum Flood (PMF)

A revised PMF was developed for Folsom Dam and Lake in 2001 for three reasons: 1) new criteria for computing the Probable Maximum Precipitation (PMP) were developed in 1996 by the Hydrometeorological Branch of the NWS and published in 1999 as Hydrometeorological Report (HMR) No. 59, *Probable Maximum Precipitation for California*; 2) several new studies were under way to evaluate modifications of Folsom Dam's spillway and outlets to reduce the flooding potential for the downstream area; and 3) Corps criteria requires that designs for new dams or those undergoing major modifications ensure the safe passage of the PMF without major damage. The revised PMF supersedes all previous PMF studies. The adopted PMF is from 2001 (Corps, 2001).

The PMF is defined in Corps guidance as that flood discharge that would result from the combination of the most severe meteorological and hydrological conditions considered reasonably possible in a region. It is produced by the combination of the PMP, basin snowmelt (when applicable), and basin runoff characteristics that result in maximum runoff. The conditions of the PMF, such as storm center location and loss rates, generate maximum peak flows.

The volume of rainfall for the PMF is the PMP. The PMP is defined as the greatest depth of precipitation for a given duration that is physically possible over a given storm area at a particular geographical location and time of year. For California, guidance for computing the PMP comes from the Hydrometeorological Design Studies Center of the NWS, and is documented in HMR No. 59. The storm that causes the PMF is based on distributing the PMP aerially and temporally, based on studies of historic major storms. HMR No. 36 was used to determine the temporal distribution of the PMP.

The snowmelt component of the PMF was developed by reproducing historic floods and then determining snowmelt by following methods defined in the EM 1110-2-1406, *Runoff from Snowmelt*, and the Reclamation's Engineering Monograph No. 35, *Effects of Snow Compaction on Runoff from Rain on Snow*. Guidelines for maximum winds, dew points, and temperatures that drive the snowmelt are found in HMR No. 59. Maximum runoff was obtained by combining the experience gained from modeling extreme floods with the guidelines found both in Corps and Reclamation guidance.

The Corps and Reclamation worked together to determine the adequacy of the Corps' 1980 HEC-1 model for producing the new PMF by modeling the February 1986 and the December 1996-January 1997 floods. Simulations of the February 1986 and January 1997 floods helped ascertain whether unit hydrographs, loss rates, or routing parameters needed adjustment. The resulting inflow hydrographs to Folsom Dam and North Fork Dam suggested very few adjustments to the unit hydrographs or the lag times.

The unit hydrographs of previous Corps PMF studies were peaked 20 to 25 percent and routing steps decreased. This produced inflows to Folsom Lake that peaked a few hours sooner and within 1 percent of the non-peaked unit hydrographs. The new PMF used the 1980 HEC-1 model without peaked unit hydrographs. Constant loss rates and antecedent snow cover were based on the historic 1997 flood event.

7.6 Seasonal Probable Maximum Floods

The Folsom Dam spillway was designed to handle the all-season PMF, which represents the maximum possible inflow to the dam during the months of December through February. Per HMR No. 59, the March PMP is to be considered the same as the December through February values. These months represent the heart of the flood season, when the available flood control storage space at Folsom Dam is at or near its maximum capacity. During the spring refill period however, the flood control space is reduced due to the change in meteorological conditions, such that the TOC is equal to gross pool by June. Since Folsom Dam has a gated spillway, restrictions on gate release changes (ramping rates) increase the length of time needed to make large outflows through the spillway. These conditions mandate that the reservoir operation rules be tested to ensure a spring-time PMF will not cause the pool elevation to exceed an elevation equal to 3.0 feet below the top of the dam. As such, off-season PMFs representing April, May, and June were utilized for this effort. The all-season PMF was used to test the reservoir in March. October and November seasonal PMFs were also developed and these were made available to the study team. The development of the seasonal PMF hydrographs is described in the report *Folsom Dam Off-Season Probable Maximum Flood*, dated 2015. Seasonal PMP was developed from HMR No. 59, while seasonally appropriate watershed conditions and assumptions were utilized in an HEC-1 rainfall runoff model.

7.7 Revised Standard Project Flood (SPF)

The study team desired to have a hydrograph representing the SPF to test the performance of the recommended alternative. The standard project flood used to be a basis of design of Corps flood control projects in the nation. It is defined as the runoff from the most severe storm that is considered "reasonably characteristic" of a region. Procedures to develop a Standard Project Storm (SPS) for the watershed of interest were based on studies of the meteorology of the region, including rare historic storms. Procedures were in place to derive an SPS which was then input into a rainfall runoff model to produce the design hydrograph called the SPF. Per the Corps Civil Engineer Bulletin No. 52-8, *Standard Project Flood Determinations*, dated March 1965, the standard project flood can be estimated as a percentage of the probable maximum flood. This bulletin recommended using the ratio of 50 percent of the PMF based on detailed studies in the nation which indicated the ratio typically fell between 40 to 60 percent (page 12). SPK published a report titled *Standard Project Criteria for General and Local Storms, Sacramento-San Joaquin Valley, California*, dated April 1971. In this study, the SPS depths were found to range between 40 and 80 percent of the probable maximum precipitation depths in the Sierra Mountains, and between 40 and 60 percent for the Coastal Ranges. In 1987, Reclamation and DWR requested the Corps to update the hydrology of the American River watershed (Corps, 1987b). In this study, SPK decided to use a ratio of 60 percent of the last updated PMF (generated in 1980 using HMR 36) based on past studies comparing SPF/PMF comparisons in the region. For the purposes of the Manual Update, a new SPF flood hydrograph will be computed using 60 percent of the maximum 72-hour volume in the latest PMF hydrograph for the LAR. The latest PMF was developed in 2001 using HMR 59 criteria. One representative SPF hydrograph was computed. The 1964 unregulated pattern hydrograph was scaled to have the 72-hour volume determined above. The 3-day volume (283,000 cfs) has a 1 in 355 ACE. A figure showing the flood routing of this event is provided in the Water Control Manual.

7.8 Climate Change Impacts

7.8.1 Overview

ECB No. 2016-25 requires Corps planning studies to provide a qualitative description of climate change impacts to inland hydrology. The purpose of this section is to meet the requirements as set forth in the ECB to enhance climate preparedness and resilience by incorporating relevant information on the impacts of climate change to inland hydrology in designs and projects (USACE, 2016). This section will describe how climate change could impact the hydrologic runoff processes in the watersheds in the Sacramento area (see Figure 7-15). It will also provide one quantitative estimate of the potential change in unregulated runoff volume in this region, based on a recent analysis by DWR.

7.8.2 Literature Synthesis

Up to the present time, Corps projects and operations have generally proven to be robust in the face of natural climate variability over their operating life spans. However, recent scientific evidence shows that in some geographic locations and for some impacts relevant to Corps operations, climate change is shifting the climatological baseline about which natural climate variability occurs and the range of the variability may be changing as well. More extreme seasonal conditions of flooding or drought may become more prevalent in some regions, especially in the southwest (USACE, 2016; USACE, 2015; USGCRP, 2014).

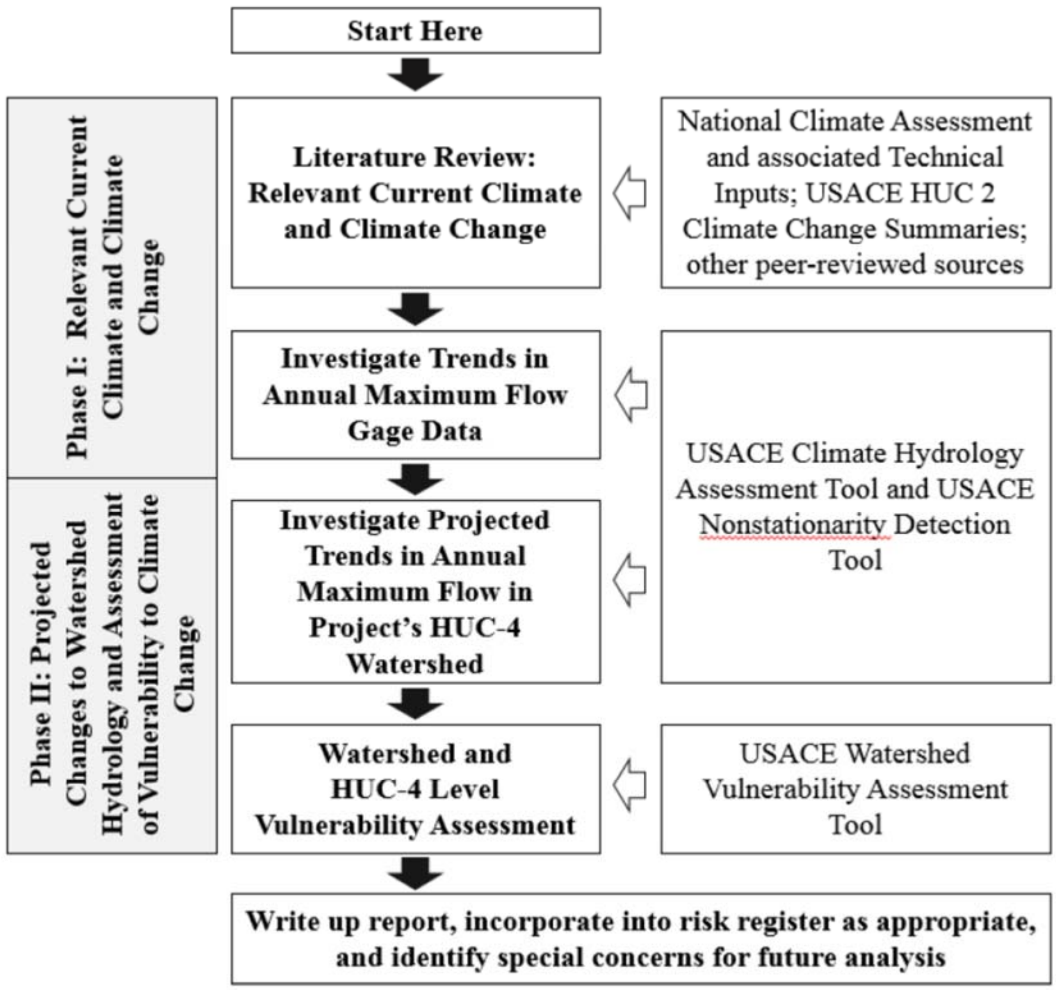

Figure 7-15: Flow Chart Describing the Qualitative Climate Change Assessment to be used in Hydrology Studies for Corps Projects (from ECB 2016-25, Attachment B)

Simulations with global climatic models are mostly consistent in predicting that future climate change will cause a general increase in air temperatures in California, including during the critical months when most precipitation falls. It has been projected that air temperatures will increase by over 3 degrees Fahrenheit by the middle of the current century. November through March is the period when the most significant and damaging storms hit this region. The American River, which flows through Folsom, has many high elevation mountains with peaks ranging from 5,000 to 11,000 feet above sea level. Significant portions of these watersheds are covered in snowpack during the winter months. As temperatures warm during the century, it is expected that the snowpack line (demarcation between bare ground and snowpack-covered ground) will recede to higher elevations, and a greater percentage of the drainage area of individual watersheds will incur rainfall, as opposed to snowfall. This trend is expected to cause significant increases in runoff volume in the high elevation watersheds for large storms. Another impact of warmer air temperatures is that the spring snowpack will melt earlier, thus increasing reservoir inflows at a time when spring storms still threaten the region and empty space is still required to attenuate flood inflows. In other words, flood control operations at reservoirs could become more difficult in the spring months. The snowpack typically begins to melt in late March or early April. With the projected increase in temperatures during the coming decades, the snowpack will begin to melt earlier in the year (i.e., early to mid-March or sooner). This will overlap the time in which large atmospheric river storms normally hit the region. This overlap could potentially increase the size of spring rain-on-snow events. The trend towards earlier spring snowmelt has already been observed in the Sierra Nevada Mountains over the last century.

With less certainty than above, some global climate models indicate that future conditions may increase the amount of moisture in the storms, since warmer air holds more moisture than cold air. When air cools, condensation occurs, which causes precipitation. It is possible that due to increasing temperatures, atmospheric rivers will have higher precipitation depths in the future, and this will lead to an increase in the size of runoff peaks and volumes. The largest storms that typically impact the west coast of the United States are termed “pineapple express” or more recently “atmospheric rivers” by meteorologists. This type of event occurs when a long plume of saturated air moves northeastward from the low-latitudes of the Pacific Ocean and mixes with cold dense air moving southward from the arctic. The mixing of cold and warm air causes a storm front. As these very moist storms move eastward over the Sierra Mountain Range, the air is pushed to higher elevations where more cooling occurs, thus increasing condensation and precipitation. Historically, the largest and most damaging floods in the Central Valley of California are caused by atmospheric rivers. In summary, it is possible that atmospheric rivers will have higher precipitation depths in the future, which will increase runoff peaks and volumes.

7.8.3 Phase I Current Climate Observations

Recent surface observations of temperature and precipitation in the southwest United States including the Central Valley of California indicate a significant warming trend starting about 1970 (NOAA, 2013). This recent warming trend is especially noticeable in the minimum temperatures during the interval from 1990 to about 2005. This warming is in addition to more general warming trends from about 1890 to the present. The reasons cited among scientists include natural multi-decadal oscillations, increased greenhouse gases in the atmosphere, land use changes, and urban heat island effects (NOAA, 2013; Levi, 2008; Barnett et al. 2008; Das et al., 2011).

The Corps Climate Hydrology Assessment Tool (Corps, 2106c) was used to examine observed streamflow trends at a gage upstream but in the vicinity of the Folsom Dam. At this time, the annual maximum 1- and 3-day flows for the USGS Gage (11433300) MF American River near Foresthill CA were studied. The tool only has capability to run first order statistics on the one day and three day flows and the Foresthill Gage was chosen because Flow is only partially controlled by upstream reservoirs, which are used mainly for hydropower with some supplemental flood control space. There is not a gaged location along the American River where the flow is completely uncontrolled so for this analysis, the Foresthill gage was chosen because it appears to have less area affected by upstream regulation than any other available location along the American River and related watersheds. The hydrologic time series for the 1-day and 3-day annual maximum flow at the Foresthill gage are shown in Figure 7-16 and Figure 7-17. The gage exhibits declining trends in stream flow for both the 1-day and 3-day time series. P values of 0.2336 and 0.2820 indicate that these observed trends are not very significant and that there has been little change in the flood risk as measured by the observed record over the last 55 years in the vicinity of this gage.

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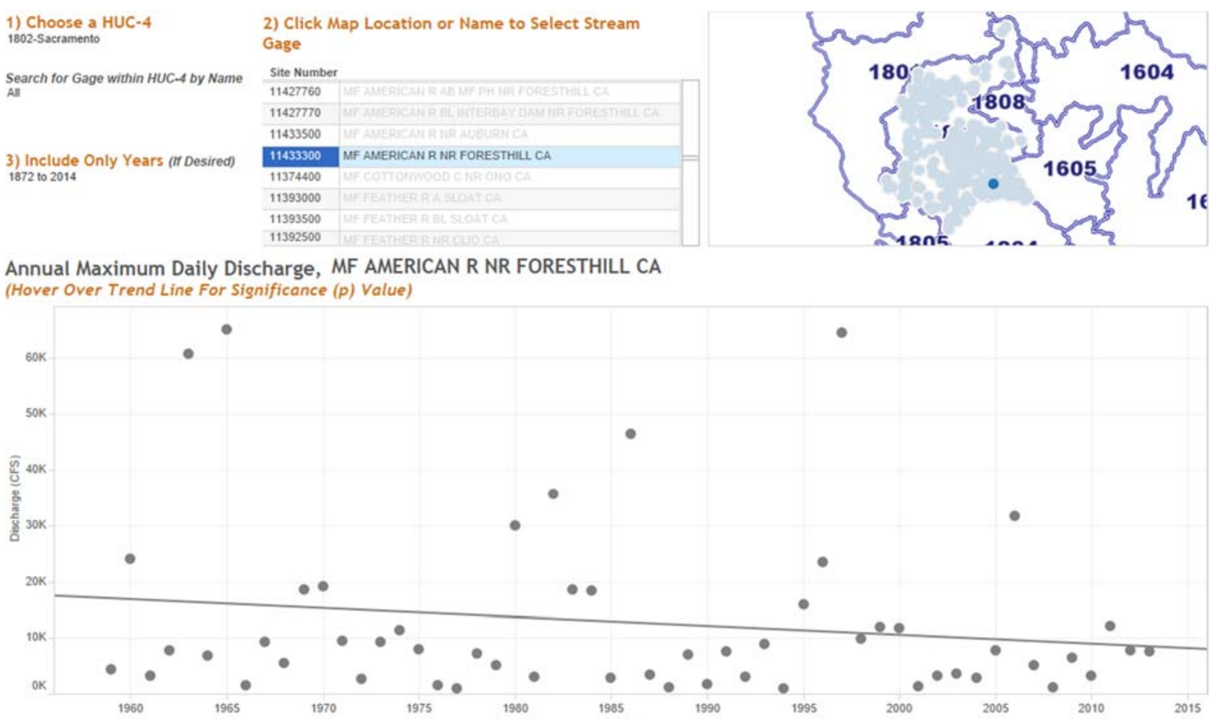


Figure 7-16: Annual Maximum Daily Discharge at Middle Fork of the American River near Foresthill Gage

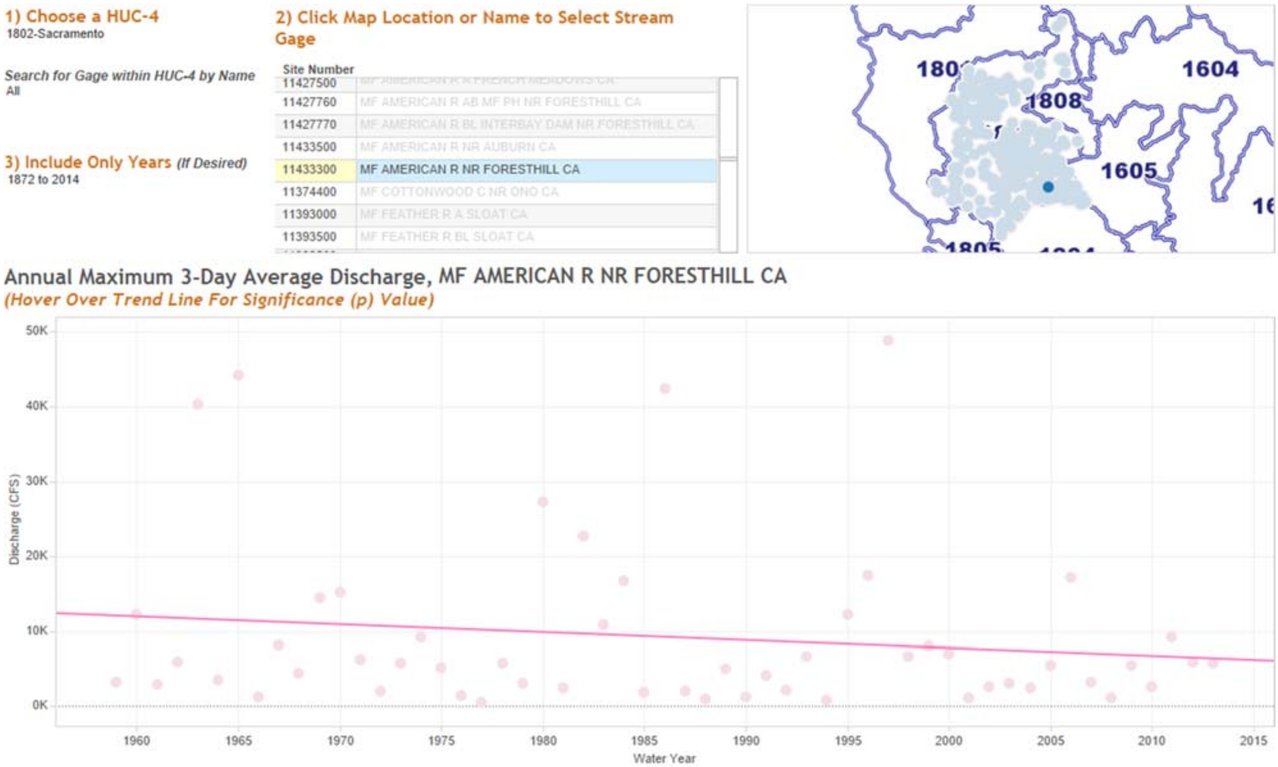


Figure 7-17: Annual Maximum 3-day Flow at the Middle Fork of the American River near Foresthill Gage

The nonstationarity detection tool (http://corpsmapu.usace.army.mil/cm_apex/f?p=257:10:0::NO) was used to examine the annual maximum annual peak flow time series data at the Middle Fork of the American River at Foresthill gage and the American River at Fair Oaks gage (Figure 7-18). Nonstationarities were not detected, further confirming that there has been no change in the flood risk for the area in the vicinity of the Foresthill gage. A monotonic trend analysis using the Mann-Kendall and Spearman Rank Order tests with 0.05 level of significance and no trends were detected (see Figure 7-19). The nonstationarity tool was also utilized to test for nonstationarities at American River at Fair Oaks gage and ,as expected, a prominent nonstationarity occurs at the time of the installation of the Folsom Dam project in 1957, as shown in Figure 7-20.

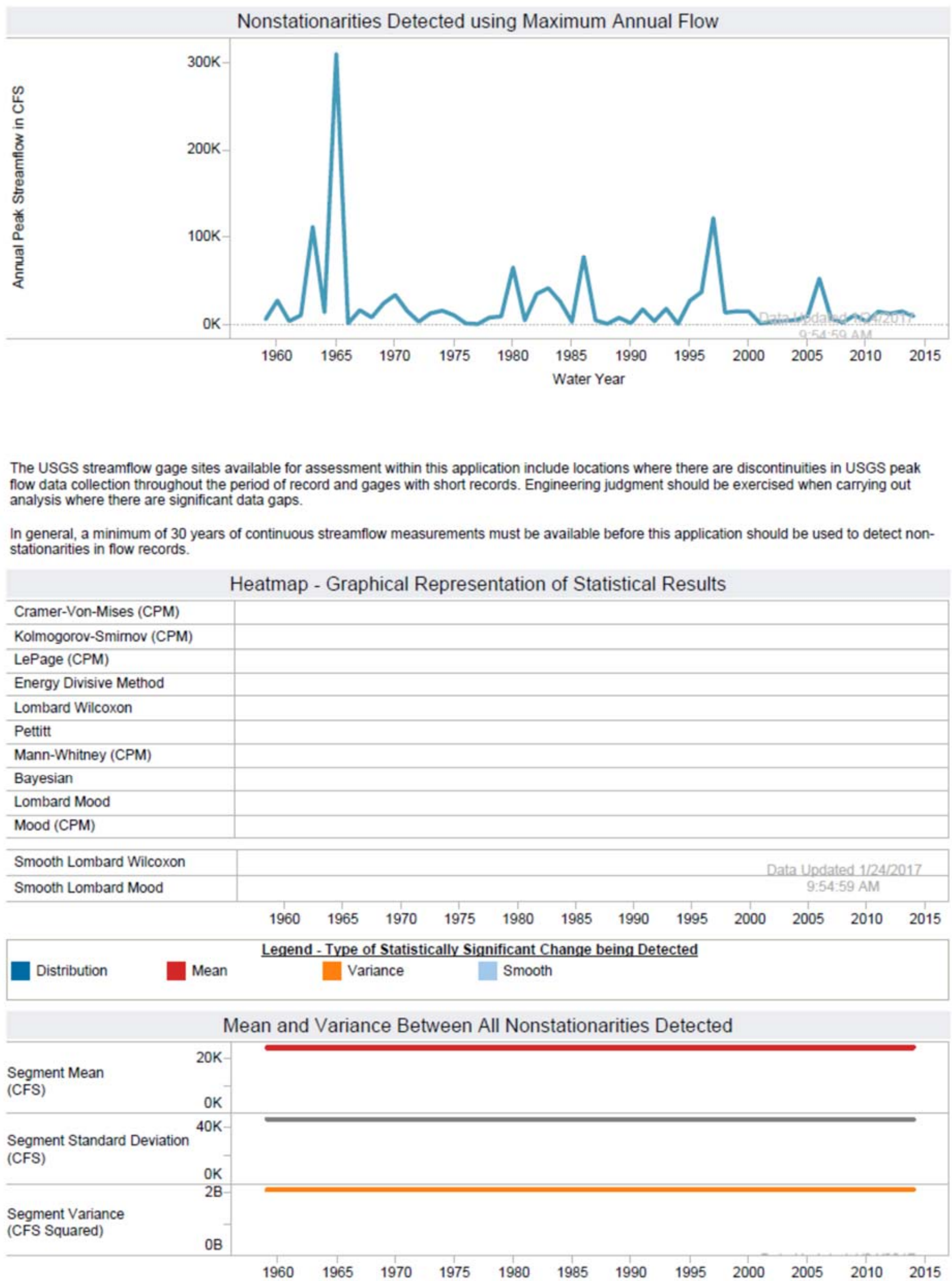
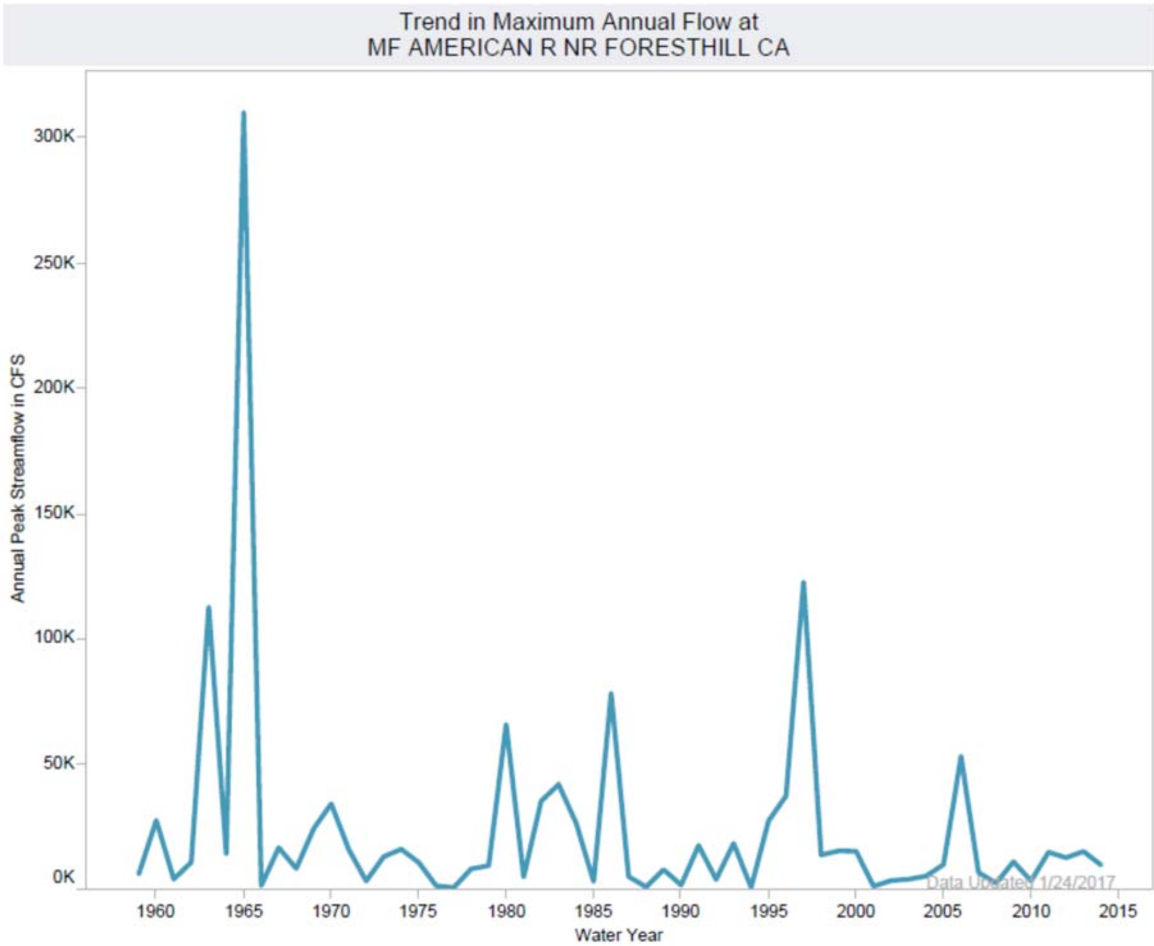


Figure 7-18: MF American River at Foresthill (USGS#1143300)
Nonstationarity determination using maximum annual flow, period of record, 1958 to 2015.
Nonstationarities were not detected.



Monotonic Trend Analysis

Is there a statistically significant trend?
No, using the Mann-Kendall Test at the .05 level of significance.
No, using the Spearman Rank Order Test at the .05 level of significance.

What type of trend was detected?
Using parametric statistical methods, **no trend** was detected.
Using robust parametric statistical methods (Sen's Slope), **no trend** was detected.

Figure 7-19: Trend Analysis of Annual Maximum Flow at MF American River at Foresthill Gage. No trends were detected.

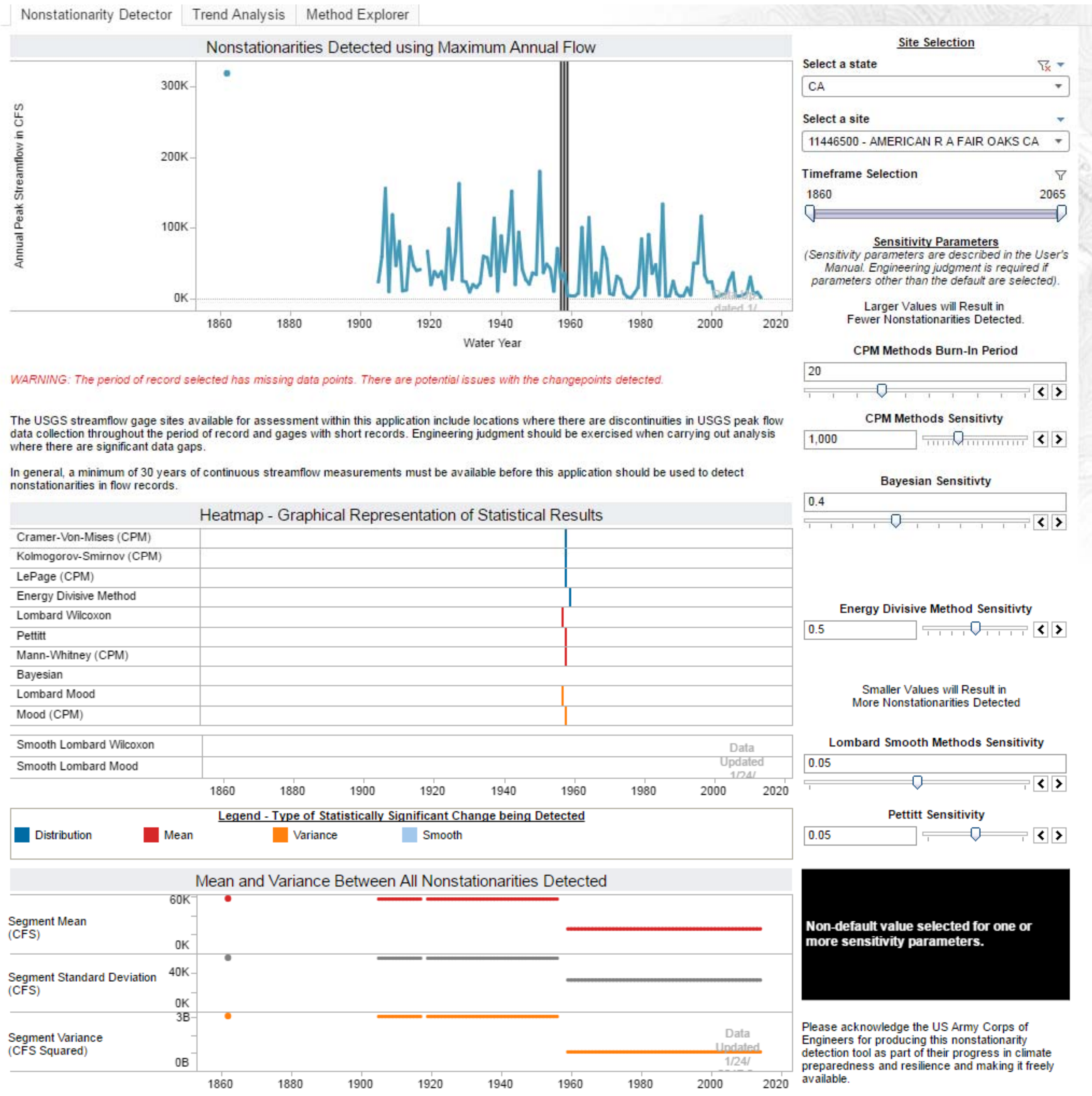


Figure 7-20: Nonstationarities in Flow Record of USGS 11446500, American River at Fair Oaks. The prominent nonstationarity in the year 1957 is due to the installation and operations of Folsom Dam.

7.8.4 Phase II Future Climate Scenarios

CH2M HILL 2014 and NOAA 2013 report that current trends and future climate projections indicate warmer winter temperatures and some changes in precipitation in the Central Valley, and this leads to an increased risk of flooding from large storms. Projected changes in future climate contain significant uncertainties. Uncertainties exist with respect to understanding and modeling of the earth's systems, estimating future development and greenhouse gas emission pathways, and simulating changes at the local scale. Climate models suggest the projected temperature signal is strong and temporally consistent. All projections are consistent in the direction of the temperature change but vary in terms of climate sensitivity. Annual precipitation projections are not directionally consistent. Multi-decadal variability complicates period analysis. Regional trends indicate that it is more likely for the upper Sacramento Valley to experience equal or greater precipitation. However extreme precipitation is likely to increase (Das et al., 2013; NOAA, 2013; CH2M HILL, 2014).

The Corps Climate Hydrology Assessment Tool was used to examine observed and projected trends in watershed hydrology to support the qualitative assessment. As expected, there is considerable and consistent spread in the projected annual maximum monthly flows (Figure 7-21). The overall projected trend in mean projected annual maximum monthly flows (Figure 7-22) increases over time and this trend is statistically significant (p-value <0.0001), suggesting that there may be potential for an increase in flood risk in the future relative to the current time. The result is qualitative only because this tool uses climate data projected by global circulation models translated using a Variable Infiltration Capacity (VIC) model developed for the entire United States. The VIC model is not calibrated to historical values in any particular watershed, thus it does not replicate exact historic streamflow within a high degree of accuracy, and this adds to the uncertainty associated with hydrological error.

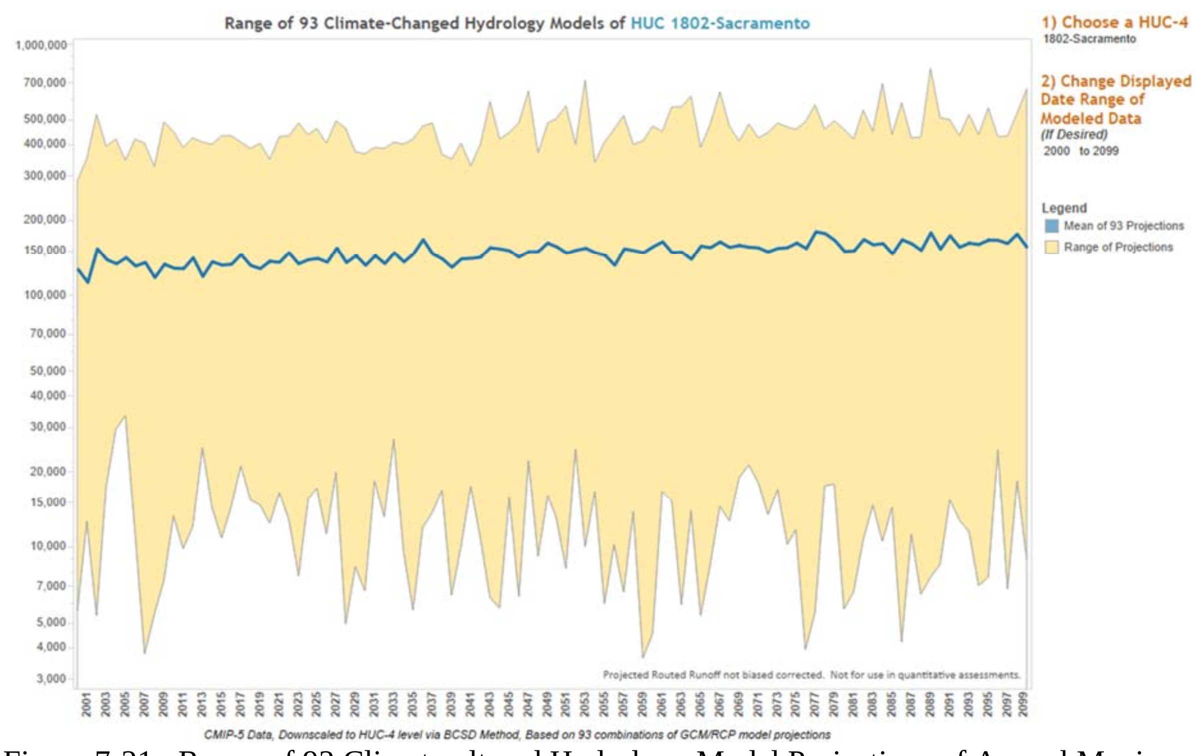


Figure 7-21: Range of 92 Climate-altered Hydrology Model Projections of Annual Maximum Monthly Average Flow in HUC 1802 Sacramento

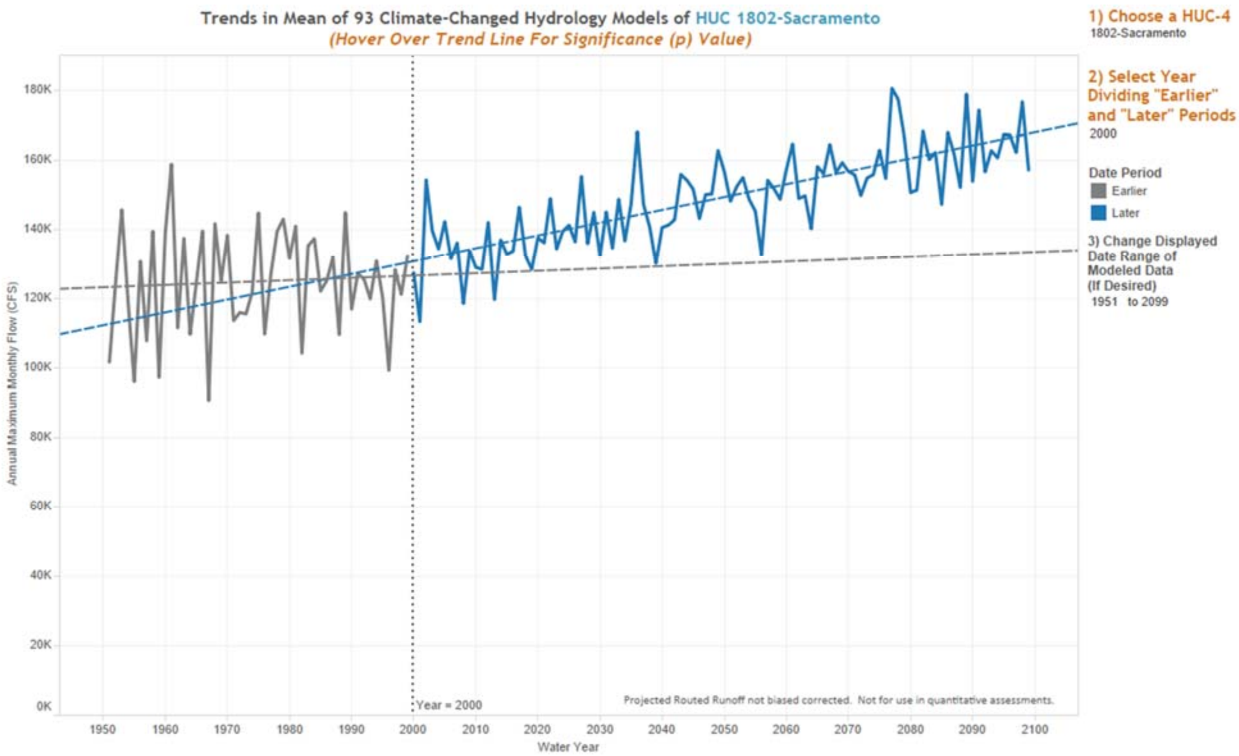


Figure 7-22: Projected Trend in Annual Maximum Flow for HUC-1802 Sacramento.
Dotted line indicates year 2000, gray dashed line indicates present trend from 1950 to 2000, and the blue dashed line indicates projected climate-altered trend in streamflow after 2000 to 2100.

The Corps Watershed Vulnerability Assessment Tool (Corps, 2106e) was used to examine the vulnerability of the project area to future flood risk (Figure 7-23) . Like the Climate Hydrology Assessment Tool, this tool uses climate data projected by GCMs translated into runoff in the VIC model, and the vulnerability assessment for inland hydrology is only qualitative at this time. This vulnerability assessment uses twenty-seven different indicators and eight business lines to develop vulnerability scores specific to each of the 202 HUC-4 watersheds in the United States for each of the business lines. The business lines are the prisms for the evaluation of vulnerability in a given watershed.

The advantages of using the Vulnerability Assessment Tool (VA Tool) are: it allows for the assessment of multiple dimensions of vulnerability; allows for the incorporation of new information; allows for the incorporation of subjective importance of indicators for different business lines; stores all historical settings and analyses such that each user's settings are independent of other users to allow freedom to make changes based on expertise and to conduct new hypothetical analyses; allows for varying risk averseness/tolerance; and that it is web-accessible with a CAC card. The VA tool gives assessments using two scenarios (wet and dry) for two of three epochs, 2035-2064 (centered on 2050) and 2070-2099 (centered on 2085). The remaining epoch (base period) covers the current time and uses recorded data rather than climate model projections. Within each of the future epochs the GCM projections are divided into two equal sized groups. The group with the lower cumulative runoff projections (below the median projection) is used to compute values for the dry scenario and the group with the higher runoff projections (above the median) is used to compute values for the wet scenario. These are all equally likely projections of the future and the dry projection could be wetter than the base epoch. For the Sacramento Watershed (HUC 1802), this tool shows that the area is highly vulnerable to increased flood risk during the twenty-first century for all wet and dry projected scenarios when compared to all other watersheds in the nation. The assessment was carried out using the national standard settings (ORness set to 0.7, all 202 HUC-4 watersheds are considered,. Analysis type is set to "Each" and vulnerability threshold is set at 20 percent). Figure 7-23 shows the breakout of indicators for each scenario and epoch combination. In both the wet and dry scenarios, the increase in the area of the 1/500 ACE, particularly in urban areas, is the dominant risk indicator followed by change in size and timing of flood runoff. This indicates that in the future, floods could increase in magnitude over time and that much of the population and economic activity will be in areas which will be vulnerable to floodwaters (at least the 1/500 ACE year floodplain). Floods could be larger and more damaging than in previous times.

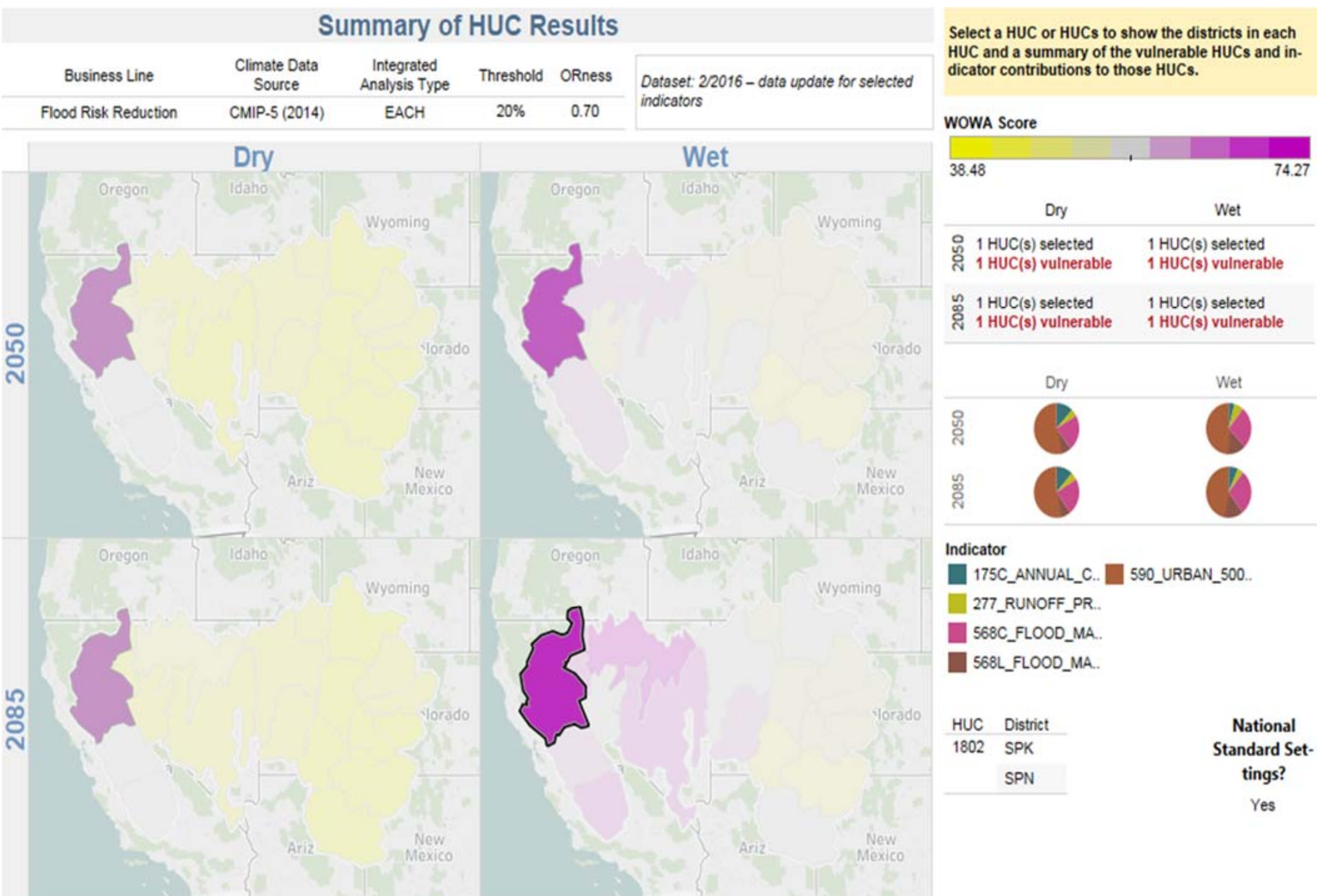


Figure 7-23: Summary of Vulnerability Assessment for HUC 1802 – Sacramento Watershed
Note: This area is vulnerable to increased flood risk due to increases in the area of the 1/500 ACE floodplain and changes in the magnitude of floods as shown in the pie charts on the right of the figure. The Weighted Order Weighted Average (WOWA) scores are in the range of 59-67, which indicates a high overall vulnerability relative to all other HUC-4 watersheds in the United States. WOWA scores (see Table 7-6) can range from 0 to 100. The pie charts display the weight of each indicator in determining the final vulnerability score of the watershed with respect to the business line (see upper left corner) which is Flood Risk Reduction for this figure. The purple shading of the selected HUC (Sacramento River) corresponds to the high vulnerability score.

Table 7-6: WOWA Scores and Contributions for HUC-4 Watershed 1802 Sacramento

Business Line	**Flood Risk Reduction**									
Epoch and Scenario	**Base Period**		**Dry 2050**		**Wet 2050**		**Dry 2085**		**Wet 2085**	
Indicator	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA
590_URBAN_500YRFLOODPLAIN_AREA	20.94	0.39	21.41	0.38	21.43	0.34	21.17	0.37	21.12	0.32
568C_FLOOD_MAGNIFICATION	11.51	0.22	12.50	0.22	16.00	0.26	12.70	0.22	17.89	0.27
568L_FLOOD_MAGNIFICATION	7.94	0.15	8.89	0.16	11.03	0.18	8.81	0.15	12.33	0.19
175C_ANNUAL_COV	7.00	0.13	7.64	0.13	7.43	0.12	7.76	0.14	7.81	0.12
277_RUNOFF_PRECIP	5.90	0.11	6.51	0.11	6.32	0.10	6.73	0.12	6.73	0.10
Total WOWA	53.28	1.00	56.95	1.00	62.22	1.00	57.15	1.00	65.87	1.00

Business Line	**Emergency Management**									
Epoch and Scenario	Base Period		Dry 2050		Wet 2050		Dry 2085		Wet 2085	
Indicator	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA
130_FLOODPLAIN_POPULATION	14.06	0.23	12.35	0.20	12.31	0.19	12.33	0.19	12.28	0.19
175C_ANNUAL_COV	2.97	0.05	3.55	0.06	3.01	0.05	3.52	0.06	3.51	0.05
277_RUNOFF_PRECIP	3.68	0.06	4.06	0.06	4.09	0.06	4.19	0.07	4.19	0.06
443_POVERTY_POPULATION	7.65	0.12	7.79	0.12	7.76	0.12	7.78	0.12	7.29	0.11
447_DISABLED	9.37	0.15	9.39	0.15	9.35	0.15	9.39	0.15	9.35	0.14
448_PAST_EXPERIENCE	3.27	0.05	3.08	0.05	3.26	0.05	3.08	0.05	3.07	0.05
450_FLOOD_INSURANCE_COMMUNITIES	2.78	0.04	2.79	0.04	2.78	0.04	2.79	0.04	2.77	0.04
568C_FLOOD_MAGNIFICATION	4.67	0.07	5.07	0.08	6.87	0.11	5.15	0.08	8.68	0.13
700C_LOW_FLOW_REDUCTION	8.21	0.13	8.72	0.14	8.62	0.13	8.83	0.14	8.16	0.12
700L_LOW_FLOW_REDUCTION	5.66	0.09	6.01	0.10	5.60	0.09	6.09	0.10	5.62	0.09
95_DROUGHT_SEVERITY	0.00	0.00	0.25	0.00	0.29	0.00	0.74	0.01	0.56	0.01
Total WOWA	62.34	1.00	63.06	1.00	63.94	1.00	63.89	1.00	65.49	1.00

Business Line	Ecosystem Restoration									
Epoch and Scenario	Base Period		Dry 2050		Wet 2050		Dry 2085		Wet 2085	
Indicator	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA
156_SEDIMENT	3.86	0.06	3.59	0.06	3.59	0.05	3.59	0.05	3.35	0.05
221C_MONTHLY_COV	11.91	0.19	13.30	0.21	13.28	0.20	13.97	0.21	14.10	0.20
277_RUNOFF_PRECIP	7.91	0.13	8.71	0.14	8.81	0.13	9.01	0.14	9.05	0.13
297_MACROINVERTEBRATE	6.14	0.10	6.14	0.10	5.72	0.09	6.15	0.09	5.73	0.08
568C_FLOOD_MAGNIFICATION	3.56	0.06	4.15	0.06	6.60	0.10	4.22	0.06	7.42	0.11
568L_FLOOD_MAGNIFICATION	2.21	0.04	2.39	0.04	3.06	0.05	2.43	0.04	3.70	0.05
65L_MEAN_ANNUAL_RUNOFF	4.83	0.08	4.89	0.08	4.12	0.06	4.54	0.07	4.11	0.06
700C_LOW_FLOW_REDUCTION	4.25	0.07	4.50	0.07	4.47	0.07	4.91	0.07	4.50	0.07
8_AT_RISK_FRESHWATER_PLANT	16.80	0.27	16.80	0.26	16.81	0.25	16.82	0.26	16.85	0.24
Total WOWA	61.47	1.00	64.47	1.00	66.46	1.00	65.66	1.00	68.80	1.00

Business Line	Navigation									
Epoch and Scenario	Base Period		Dry 2050		Wet 2050		Dry 2085		Wet 2085	
Indicator	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA
156_SEDIMENT	6.97	0.12	6.50	0.10	6.47	0.10	6.48	0.10	6.46	0.09
192_URBAN_SUBURBAN	1.07	0.02	1.19	0.02	1.18	0.02	1.12	0.02	1.11	0.02
221C_MONTHLY_COV	4.49	0.07	5.00	0.08	4.96	0.08	5.97	0.09	5.99	0.09
277_RUNOFF_PRECIP	5.25	0.09	7.01	0.11	7.06	0.11	7.23	0.11	7.22	0.11
441_500YRFLOODPLAIN_AREA	6.34	0.11	5.53	0.09	5.51	0.08	5.17	0.08	5.15	0.08
568C_FLOOD_MAGNIFICATION	8.49	0.14	9.16	0.14	13.30	0.20	9.28	0.14	14.89	0.22
570C_90PERC_EXCEEDANCE	12.31	0.20	12.36	0.20	11.51	0.17	12.36	0.19	11.49	0.17
570L_90PERC_EXCEEDANCE	5.90	0.10	5.94	0.09	5.91	0.09	5.57	0.09	5.53	0.08
700C_LOW_FLOW_REDUCTION	9.50	0.16	9.96	0.16	9.22	0.14	10.05	0.15	9.25	0.14
95_DROUGHT_SEVERITY	0.00	0.00	0.58	0.01	0.66	0.01	1.82	0.03	1.39	0.02
Total WOWA	60.32	1.00	63.23	1.00	65.80	1.00	65.04	1.00	68.47	1.00

Business Line	Recreation									
Epoch and Scenario	Base Period		Dry 2050		Wet 2050		Dry 2085		Wet 2085	
Indicator	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA
156_SEDIMENT	3.22	0.06	3.00	0.05	2.99	0.05	3.01	0.05	3.00	0.05
221C_MONTHLY_COV	9.57	0.17	11.49	0.19	11.44	0.18	13.00	0.21	13.07	0.20
277_RUNOFF_PRECIP	4.55	0.08	5.02	0.08	5.06	0.08	5.20	0.08	4.83	0.07
568C_FLOOD_MAGNIFICATION	5.18	0.09	5.61	0.09	7.70	0.12	5.71	0.09	8.65	0.13
568L_FLOOD_MAGNIFICATION	2.97	0.05	3.47	0.06	4.42	0.07	3.53	0.06	5.35	0.08
570L_90PERC_EXCEEDANCE	12.37	0.22	12.53	0.21	12.47	0.20	11.71	0.19	11.64	0.18
571C_10PERC_EXCEEDANCE	7.44	0.13	7.50	0.13	7.10	0.11	7.53	0.12	7.19	0.11
700C_LOW_FLOW_REDUCTION	10.52	0.19	10.31	0.17	10.21	0.16	10.45	0.17	10.28	0.16
95_DROUGHT_SEVERITY	0.00	0.00	0.87	0.01	1.00	0.02	2.59	0.04	1.98	0.03
Total WOWA	55.83	1.00	59.80	1.00	62.40	1.00	62.72	1.00	66.00	1.00

Business Line	Regulatory									
Epoch and Scenario	Base Period		Dry 2050		Wet 2050		Dry 2085		Wet 2085	
Indicator	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA	Raw WOWA	% WOWA
156_SEDIMENT	2.89	0.05	2.89	0.05	2.73	0.04	2.89	0.04	2.72	0.04
175C_ANNUAL_COV	4.90	0.08	5.85	0.09	4.69	0.07	5.80	0.09	5.47	0.08
221C_MONTHLY_COV	10.07	0.17	11.24	0.18	11.23	0.17	11.81	0.18	11.89	0.18
277_RUNOFF_PRECIP	3.80	0.06	4.44	0.07	4.23	0.06	4.59	0.07	4.33	0.06
297_MACROINVERTEBRATE	4.24	0.07	3.99	0.06	3.76	0.06	3.76	0.06	3.76	0.06
568C_FLOOD_MAGNIFICATION	3.25	0.05	3.74	0.06	6.10	0.09	4.04	0.06	7.26	0.11
568L_FLOOD_MAGNIFICATION	1.98	0.03	2.15	0.03	2.92	0.04	2.18	0.03	3.47	0.05
65C_MEAN_ANNUAL_RUNOFF	5.50	0.09	5.22	0.08	5.13	0.08	5.22	0.08	4.81	0.07
65L_MEAN_ANNUAL_RUNOFF	3.54	0.06	3.37	0.05	3.28	0.05	3.36	0.05	3.07	0.05
700C_LOW_FLOW_REDUCTION	6.78	0.11	7.19	0.11	7.14	0.11	7.28	0.11	6.75	0.10
8_AT_RISK_FRESHWATER_PLANT	14.04	0.23	14.03	0.22	14.05	0.22	14.04	0.22	14.04	0.21
Total WOWA	61.00	1.00	64.12	1.00	65.26	1.00	64.97	1.00	67.59	1.00

Notes: 1). Results from the Corps, CRRL, Watershed Vulnerability Assessment Tool on 10 Mar 2017. 2). Total WOWA scores can range from 0 to 100 and scores are relative to the other HUC-4 Watersheds in the U.S.

7.8.5 Climate Change Research by Department of Water Resources

DWR has invested millions of dollars to study climate change impacts on the flood control system in the Central Valley. Results were recently published in the *Draft 2017 CVFPP Update – Climate Change Analysis Technical Memorandum* dated March 2017. The results are based on downscaled outputs from a subset of the Coupled Model Intercomparison Project – Phase 5 (CMIP5) global climatic models, which DWR has determined are most suitable for modeling climate change on the west coast of California. The downscaled results are fed into a VIC rainfall runoff model of the Sacramento and San Joaquin River watersheds. ECB 2016-25 provides website tools that utilize downscaled results from a larger group of CMIP5 models. The DWR analysis relies upon existing, available climate projections and hydrologic modeling to represent a range of potential future changes to unregulated flow volumes due to climate change. The draft results provided by DWR has projections of volume change for 1-day and 3-day durations at many index points throughout the Sacramento River, including the American River Watershed. This section examines changes in the 3-day unregulated flow volume at the American River index point AMR 14 and translates them into projected regulated peak outflows by use of an unregulated to regulated flow transform curve.

DWR reports that current trends and future climate projections indicate warmer winter temperatures and some changes in precipitation in the Central Valley, and this leads to an increased risk of flooding from large storms.

In general, temperature change projections are more robust (and stable) than changes in precipitation. In order to be able to distinguish the effects of precipitation and temperature separately and to characterize changes over time, the following scenarios were developed:

1. Warming Only Scenarios (no precipitation changes)

    a. Near-Term: Projected warming of about +1.8º F

    b. Mid Century: Projected warming of about +3.6º F, and

    c. Late Century: Projected warming of about +4.5º F to +5.4º F

2. Combined Warming and Precipitation Change Scenarios:

    a. Near-Term: Projected precipitation and temperature changes

    b. Mid Century: Projected precipitation and temperature changes, and

    c. Late Century: Projected precipitation and temperature changes

DWR examined the above scenarios and determined a “most likely” estimate for years 2070 to 2099 (2085 midpoint) while the baseline period is considered 1971-2000 (1985 midpoint).

Table 7-7 provides a summary of the projected increase in the 1- through 30-day quantile values for the baseline condition compared to with-climate change conditions for 2085.

Table 7-7: Climate Change Impact on Regulated Outflow Frequency

Ratio to Apply to Baseline Quantiles to Reach Projected 2085 Future Conditions							
AEP	**Return Period**	**1Day**	**2Day**	**3Day**	**7Day**	**15Day**	**30Day**
0.001	**1000.0**	**1.03**	**1.09**	**1.14**	**1.34**	**1.41**	**1.36**
0.002	**500.0**	**1.03**	**1.09**	**1.14**	**1.34**	**1.41**	**1.36**
0.005	**200.0**	**1.03**	**1.09**	**1.14**	**1.34**	**1.41**	**1.36**
0.01	**100.0**	**1.12**	**1.17**	**1.22**	**1.38**	**1.43**	**1.38**
0.02	**50.0**	**1.22**	**1.26**	**1.30**	**1.42**	**1.45**	**1.40**
0.04	**25.0**	**1.32**	**1.35**	**1.38**	**1.46**	**1.46**	**1.41**
0.1	**10.0**	**1.46**	**1.47**	**1.48**	**1.49**	**1.47**	**1.42**
0.2	**5.0**	**1.56**	**1.56**	**1.55**	**1.51**	**1.45**	**1.40**
0.4292	**2.3**	**1.65**	**1.62**	**1.60**	**1.48**	**1.40**	**1.34**
0.5	**2.0**	**1.66**	**1.63**	**1.59**	**1.47**	**1.38**	**1.32**
0.6667	**1.5**	**1.65**	**1.61**	**1.57**	**1.42**	**1.31**	**1.25**
0.8	**1.3**	**1.61**	**1.56**	**1.51**	**1.35**	**1.24**	**1.18**
0.9	**1.1**	**1.52**	**1.47**	**1.42**	**1.27**	**1.15**	**1.08**
0.95	**1.1**	**1.42**	**1.37**	**1.33**	**1.19**	**1.06**	**0.99**
0.99	**1.01**	**1.18**	**1.15**	**1.12**	**1.19**	**1.06**	**0.99**

Table 7-8 provides a comparison of regulated outflow frequency for Folsom Dam forecast-based operation under today's climate and under a future (2085) scenario.

Table 7-8: Comparison of Existing and Future (2085) Climate Regulated Outflow Frequency

ACE	1/ACE	Reg Flow (cfs)	Reg Flow (cfs)
0.1	10	76,000	89,000
0.04	25	94,000	115,000
0.02	50	115,000	115,000
0.01	100	115,000	115,000
0.005	200	129,000	205,000
0.002	500	375,000	400,000

Note: Based on HEC-ResSim model simulations with starting storage at 400,000 acre-feet flood control space available

7.8.6 Conclusions

New climate projections (CMIP5) are now available which are consistent with the most recent Intergovernmental Panel on Climate Change (IPCC) Assessment Report 5 (AR5) (Taylor et al., 2012). Three on-going, DWR-supported research studies were initiated in 2013 and are expected to provide informative and quantitative analyses of potential impacts. These include the Climate Variability Sensitivity Study (completed by the Corps in 2014), which evaluated the effects of increasing temperature only (not precipitation) on flood runoff on selected watersheds. The other two include the Atmospheric River Study (led by Scripps Institute of Oceanography/USGS) investigating indices and future projections of the major flood-producing atmospheric processes, and the Watershed Sensitivity Study (led by UC Davis) investigating the atmospheric and watershed conditions that contribute to the extreme flows on several Central Valley watersheds. Both observations and downscaled climate model outputs indicate that the climate in the Sacramento Valley of California will be warmer and possibly wetter than the present one. The likelihood of large floods will increase due to increases in moisture content of the storms and higher snow levels leading to more precipitation falling as rain and more basin exposure for runoff to occur. In addition to flood risk reduction, Folsom Dam is also used for hydropower, recreation and some municipal water supply operations and ecosystem concerns (fish releases); thus, it is important to consider other effects of a warming climate on the project operations. Droughts are expected to become more extreme or prolonged, causing water supply and hydropower concerns.

These possible changes in the climate of the Sacramento Valley will impact the operation of Folsom reservoir in the following two ways: 1) storms would bring more rain and less snow, thus creating more runoff than before, and 2) the melting of the snowpack will begin sooner in the year, thus causing a major impact on water supply and hydropower operations, especially in dry years. The increase in the amount of precipitation falling as rain in large storms could mean that more flood control space will be required in wet years; therefore, more serious consideration will have to be given to rainfall and runoff forecasts than before.

The team should consider and evaluate whether there are any actions that can be taken in the context of the current study to make the community more resilient to higher future flows. Such actions might include flood-proofing or acquiring structures, developing evacuation plans, land use planning, changes to levees and levee alignment, and adjusting elevation or spacing of mechanical features (e.g., pump stations), among other actions. Per guidance, a discussion of climate change impacts will be included in the Water Control Manual. Climate Change is expected to have a negative impact on both alternatives by reducing the level of protection that is provided. Similarly, more extreme droughts will reduce water supply available for public use under both plans. Of the two alternative operation plans, the forecast-based operation is more resilient to both droughts and floods. Alternative 2 (forecast-based operation) has been shown to provide better water supply benefits from period of record model simulations due to the Variable Flood Control Space that minimizes the flood space needed to 400k acre-feet, unless a large flood (approximately 10-year or larger) is forecasted. Alternative 2 has also been shown to provide improved performance and a higher level of protection, due to its pro-active method of making room for the flood based on forecast technology. Forecast technology is expected to improve over the coming years and decades, which will assist in making pre-emptive flood space available as the storm approaches.

8 Downstream Flood Risk and Erosion Effects

A variety of hydrologic and hydraulic engineering analyses were conducted to assess the recommended plan in terms of its role in the associated flood risk management system, and broader effects required by NEPA and related requirements. For many of the assessments performed, translation of the hydrologic effects of the changed flood operations required hydraulic modeling simulations of the leveed conveyance system downstream. A variety of other hydraulic engineering analyses were also performed to assess erosion and related effects. Much of this work utilized information generated from prior flood risk management study and design efforts. Many of the analyses assume the existing rock is in place for both the future without- and future with-Water Control Plan (WCP). However, the American River Common Features (ARCF) Project is authorized to place rock to protect the levees. Therefore, the future without- and future with-WCP erosion analyses that do not incorporate the ARCF erosion measures over-predict the impacts to levees.

8.1 Downstream Flood Risk

The overarching goal of the WCM Update project is to minimize downstream flood risk. The downstream American River-Sacramento River system features high levees, and the Sacramento Weir and Bypass, which pull high discharges out of the Sacramento River and into the Yolo Bypass, including flows that originate as releases from Folsom Dam. Multiple Corps FRM planning studies and projects and Corps levee risk assessments on this system have been completed, and others are ongoing. These have informed our understanding of the system's flood risk drivers and resulted in significant mitigation of those factors. Additional levee and system improvements have been authorized and are expected to be implemented in the near future. The recommended Water Control Plan (WCP), however, must account for the system's current flood risk context.

Recently, the American River Common Features General Reevaluation Report study (ARCF GRR) recognized the risk of bank/levee erosion-induced levee failure as a significant problem, and recommended extensive bank protection to mitigate it. The ARCF GRR also characterizes the level of flood risk that would exist after implementation of those measures as relatively high, due to residual levee failure possibilities, the chance that the system's capacity can be exceeded, and the extremely high consequences of such an occurrence. Additionally, Corps Levee Safety risk assessments have been conducted on the American River, and they also identified as the driving LSAC I category failure modes levee breach due to overtopping, and levee breach prior to overtopping due to bank and levee erosion. With the passage of the Water Infrastructure Improvements for the Nation (WIIN) Act in 2016, construction of needed bank protection features is expected to occur over the next 3 to 20 years. This work is expected to generally occur in an order that prioritizes the worst cases first. The recommended WCP reduces flood risk in the interim by reasonably balancing both of the downstream system's primary risk drivers.

In an effort to maximize the flood risk reduction it provides, the recommended WCP (Alternative 2) was developed to minimize the probability of downstream levee overtopping during runoff events that exceed the effective flood storage capacity of Folsom Reservoir without making unnecessarily high releases. Such an event would trigger releases in excess of 160,000 cfs in order to assure the safety of the dam, which would exceed the capacity of the downstream system. The effective flood storage capacity is a function of the releases called for by the reservoir's Water Control Plan (WCP), which serves to regulate the usage of the available flood storage as follows: when releases exceed inflows, available storage will be increased; when releases equal inflows, available storage will be maintained; and when releases are less than inflows, available storage will be used and thereby decreased. When routing flood events, a plan that calls for generally greater releases will use less storage than a plan with lesser releases. As a result, the plan with greater releases will result in a reservoir with greater effective capacity. This greater effective capacity translates to an ability to handle a larger inflow event without triggering releases that exceed the downstream system capacity, which in turn translates to a less frequent occurrence of overtopping (e.g., a 1/230 ACE as compared to a 1/200 ACE). The recommended WCP is checked against available information to indicate its net effect on the possibility of an erosion-induced levee failure. The recommended WCP attempts to appropriately weigh and balance the competing storage maximization and release minimization risk factors, which have competing implications on how a WCP should regulate flood storage and releases.

Fundamentally, the Alternative 2 balances these competing storage maximization and release minimization imperatives with its explicit use of forecasted runoff, which provides several advantages over other, more traditional approaches. First, it leverages the best forecast information available to guide release decisions based on the volume and timing of expected runoff. As a result, it more effectively “right-sizes” releases according to the overall magnitude of the event, and allows for releases to be minimized and curtailed in a timely manner, when the recession of runoff is clearly indicated in the forecast. By comparison, the traditional WCP (Alternative 1) approach relies on instantaneous measurements of storage, inflow, and watershed conditions. This can lead to unnecessarily increased releases and/or prolonged high releases, because the releases it requires near the peak of a runoff event can be greater than needed to safely manage the actual remaining runoff volume. Second, by triggering releases greater than inflows (i.e., “advanced releases” or “pre-releases”) as a large runoff event advances, the recommended WCP will allow more of an event, and generally more events, to be regulated by smaller, less-erosive releases.

The regulated flow frequency relationships developed for recommended (Alt. 2 – Forecast-based (J602F), alternative (Alt. 1 – Credit-based (J602P)) and baseline WCP conditions (Existing Interim (E504)) in Figure 6.10 and Table 6.23 illustrate the flood risk reduction and potential erosion risk effects of the recommended WCP. It regulates events as big as the 1/255 annual chance exceedence (ACE) event to the downstream design discharge value of 160 kcfs, as compared to a 1/118 ACE event for the current condition, and a 1/223 ACE event for a more traditional WCP alternative. Similarly, a discharge value of 115 kcfs is maintained for up to a 1/185 ACE event for the recommended WCP, as compared to a 1/106 ACE and 1/143 ACE event for the current condition and more traditional WCP, respectively. The downstream levee overtopping probability is reduced significantly by the recommended WCP. Additionally, these curves show that flows in the range of 80 kcfs to 115 kcfs will occur less frequently under the recommended WCP than under existing conditions and that a release of 115 kcfs would occur about half as often (about 1/40 ACE) as compared to the alternative traditional approach (about 1/20 ACE). On the other hand, the curves also show that flows of up to 80 kcfs will occur more frequently with the recommended WCP as compared to current conditions. This increase is a byproduct of the recommended WCP's ability to regulate larger floods to a release of 160 kcfs, and a reflection of the release capacity that the Joint Federal Project's auxiliary spillway adds to Folsom Dam. The alternative WCP produces flows above 50 kcfs more frequently than the recommended WCP, and doesn't decrease overtopping risk as much as the recommended WCP.

It should be noted that all of the flow frequency relationships developed overstate the probabilities of large flow releases, likely to a modest degree. The flow data used to develop the flow frequency curves was generated by reservoir routing simulations that strictly apply WCP rules as formal and rigid logic in the routing models. The models provide a useful baseline for understanding how respective WCPs operate and compare, but they do not reflect the discretion that Water Managers have to modify releases based on conditions prevailing at the time. For multiple reasons, the normal operational imperative for Water Management is to minimize the occurrence of large release. As such, for each condition modeled there are likely occurrences of 80 kcfs releases that would in actuality be avoided based on more sophisticated analysis of probabilistic runoff forecasts and sound operator judgment.

Since discharges of 80 kcfs and less on the American River can certainly cause erosion to occur, checks and some analysis were performed to assess the significance of the erosive effects of the flow regime expected from the recommended WCP. A check against empirical data suggests that the recommended WCP effectively balances levee overtopping and erosion risks. Discharges of 115 kcfs or more have occurred twice on the American River since 1986. In each case, erosion occurred and threatened levees in a number of locations. However, in neither case did a levee breach as a result of those flows. The resultant and other problematic erosions sites have since been repaired with Corps-approved riprap bank protection. A comparison of the recommended WCP peak flow frequency results against empirical data shows that flows greater than 80 kcfs have occurred and not caused levee failure from erosion, and the historical flow events that have posed significant erosion threats to the system levees will occur less frequently under the recommended WCP.

Another check was performed using ARCF GRR levee performance curves for the American River, where erosion is of greatest concern in the system downstream from Folsom Dam. These curves estimate the probability of levee failure when river stages range from the toe to the crest of the levee they represent, and account for all potential failure modes, including erosion. Though they rely heavily on professional judgment, do not account for flood fighting activities, and were developed for FRM planning study purposes only, these curves were nonetheless applied as a check on the possible levee erosion and performance effects resulting from the recommended WCP. Curves were selected from ARCF GRR index points that best reflect erosion concerns. The curves indicate levee failure during the occurrence of a flow of up to 80 kcfs is extremely unlikely. This is primarily because the water surface for this flow rate at the index points evaluated falls below the levee toe. At locations downstream, this is not the case, and failure modes other than erosion reflect a very low chance of failure in these locations. However, the driving failure modes at these locations are being addressed by ARCF GRR and other authorized levee improvements in the Natomas basin. The curves evaluated suggest that the occurrence of the maximum discharge, resulting from the recommended WCP, that would be an increase compared to current conditions does not increase downstream flood risk from a levee erosion failure. Notably, these curves predict failure probability due to a single occurrence of the respective flow values, and not repeated occurrences of such flows over an indefinite time period. Over the long-term, multiple additional occurrences of 80 kcfs would likely result in an increased possibility of levee failure. However, the ARCF GRR-recommended bank protection work and/or maintenance activities would mitigate that risk.

Channel widening, sediment transport, and bridge pier scour analyses were conducted to further evaluate the potential long-term effects of erosion on the downstream system to inform environmental effects. These analyses do not address the effects of the WCP on single flood events or for discharges above 115 kcfs. They are designed to assess potential long-term trend differences between existing operations and operations under the WCP. The results of these analyses generally indicate increases in erosion for some portions of the American River will occur, with the potential for increased maintenance activity needed over time. Again, the bank protection work authorized via the ARCF GRR would address some of these concerns. The significance of these and other effects were also assessed, and the results are documented in the draft *Environmental Assessment/Environmental Impact Report* for this project.

Unfortunately, even the best tools and methods for estimating erosion amounts and probabilities associated with various discharge levels produce results that are significantly uncertain. This makes it difficult to heavily weigh such results in flood risk management decisions. In such a context, it is helpful when a variety of approaches yield consistent trends and compatible indications and are supported by historical data. In this instance, analytical results and empirical evidence support a conclusion that the recommended WCP has appropriately balanced competing flood risk factors to reasonably minimize downstream flood risk to the extent it is able to, and is very unlikely to have transferred flood risk to downstream levees. However, the recommended WCP will generate flows capable of erosion up to 80 kcfs more frequently, and there is a possibility that an isolated, unknown, and highly erodible material could be present in a bank near a levee and erode rapidly and then fail a levee during such an occurrence. The probability of this happening is considered highly unlikely, and is expected to diminish over time, beginning in the near future.

8.2 Levee Performance Check

This section translates peak flows existing and selected WCP conditions into a probability of levee failure (levee fragility) function for selected locations (Figure 8-1) on the American River. The purpose of the evaluation is to determine how relative changes in peak flow exceedence probability between the existing conditions and selected WCP conditions affect downstream flood risk based on expected performance of downstream levees.

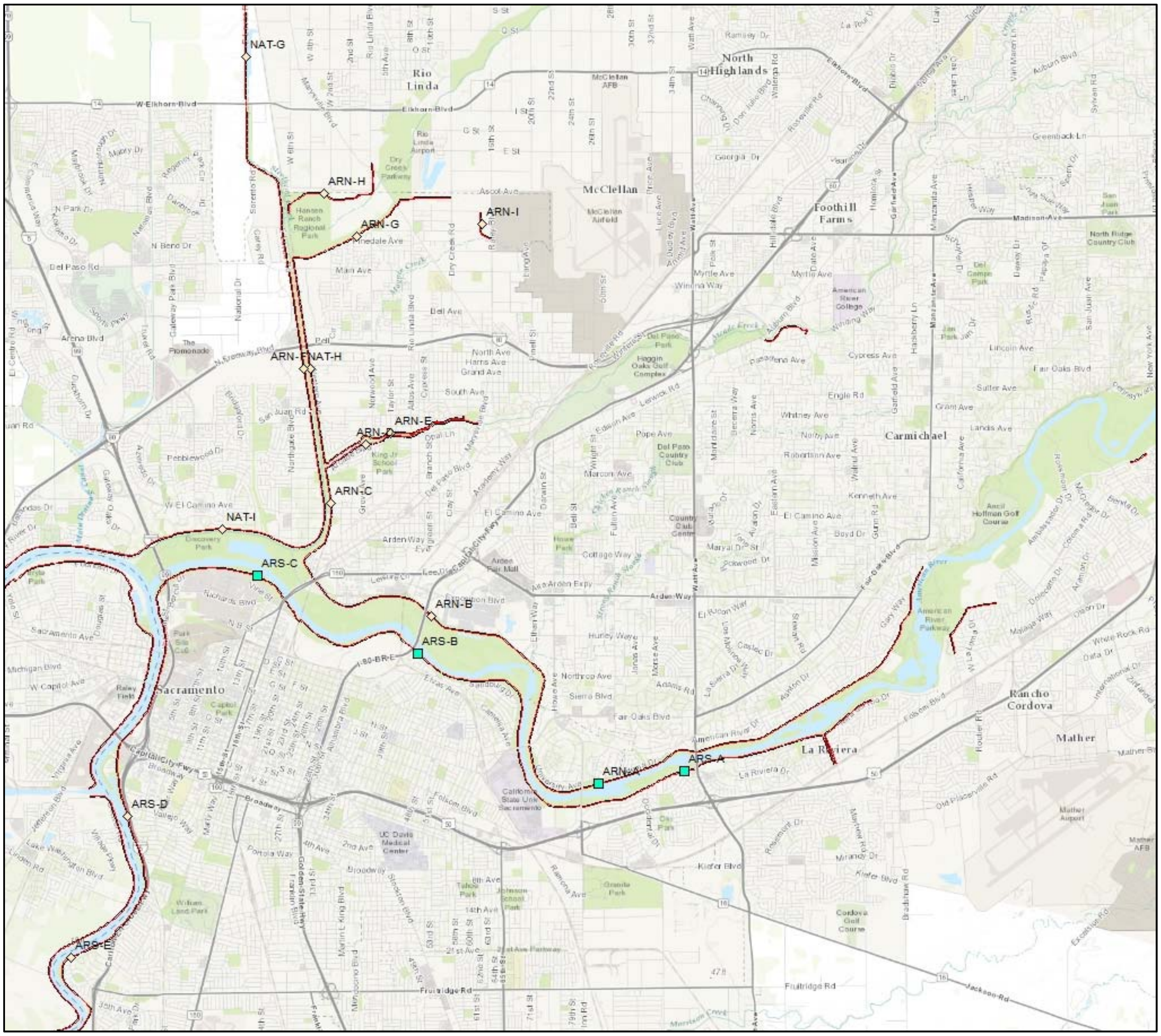


Figure 8-1: Map of Evaluated Index Points (Blue)

The levee fragility-exceedence probability function was developed by inferring each ordinate on the appropriate flow-frequency curve (see Figure 6-10) against a stage-discharge curve and then unto a stage-fragility curve.

8.2.1 Assumptions

1. Analysis doesn't account for volumetric losses such as upstream flanking or levee overtopping nor does it directly factor in effects from high tailwater at the Sacramento River confluence.
2. Analysis only takes into account the relative changes in peak flows between the existing interim and selected WCPs.
3. Fragility curves state that failure doesn't occur at stages below the levee toe, so this assessment does not consider levee failure as a result of bank or foundation failure.
4. Uncertainty with any parameter was not evaluated.
5. All curves are associated using linear Interpolation only.

8.2.2 Stage-Discharge Rating Curves

The stage-discharge rating curves for each index point were developed for the ongoing Dam Raise economic update (FY2017). The rating curves are shown in Figure 8-2.

For each flow ordinate in Figure 6-14: Regulated Peak Flow-Frequency Curves, an associated stage was determined at each index point. It was assumed that there were no volumetric losses between Folsom Dam and the index point, some 25- to 32-miles downstream. But if the peak flow was greater than the maximum flow specified on the stage-discharge rating curve (essentially representing 1/500 ACE) then the maximum stage on the rating curve was used, which does factor in upstream losses and is reflective of the true maximum stage that would be expected regardless of flow rate.

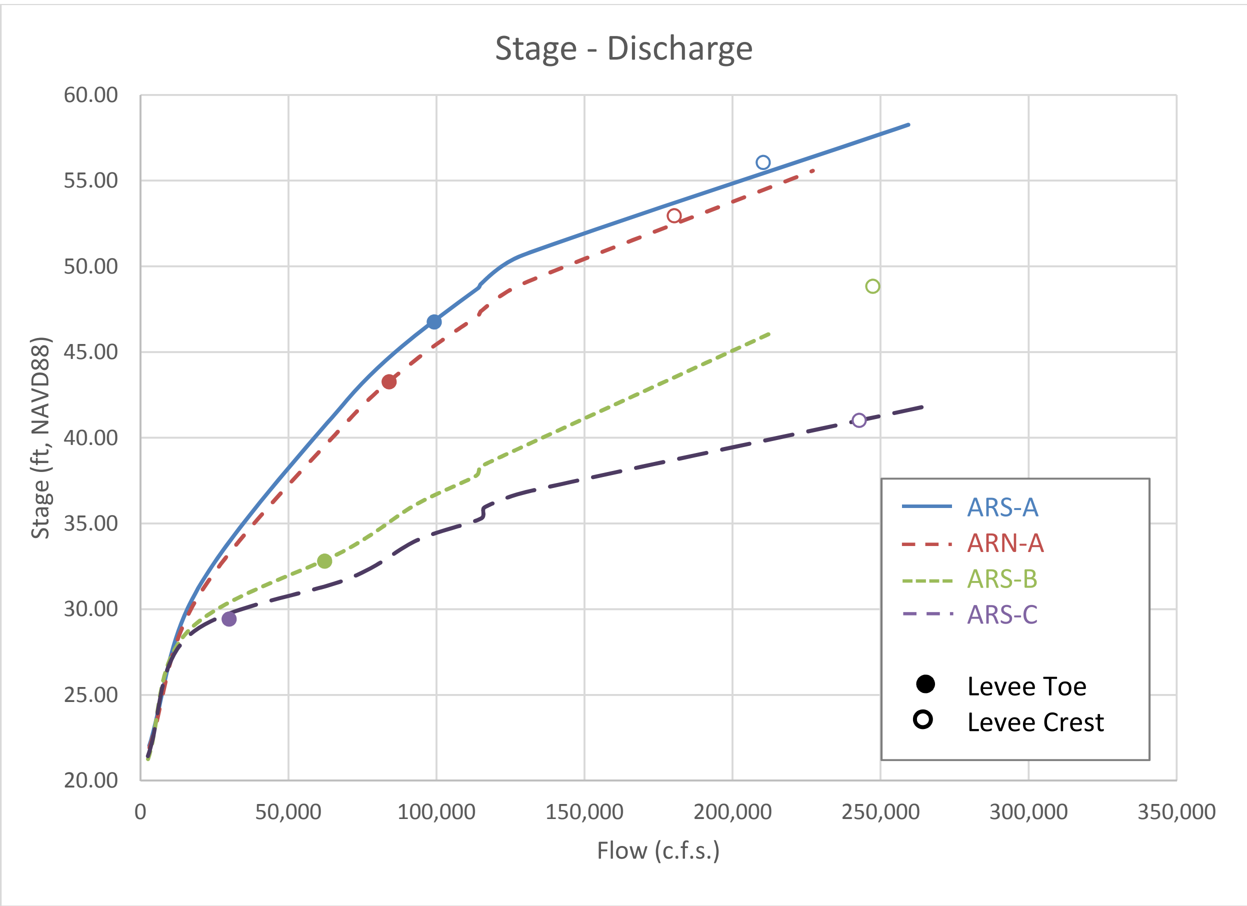


Figure 8-2: Stage-Discharge Rating Curve

8.2.3 Levee Fragility

Each index point has an associated levee fragility which is a function of probability of levee failure based on river stage. The fragility curves were developed by Geotechnical Engineering Branch for the American River Common Features GRR in 2011/2012 and are the most recently developed fragility curves available for these index locations. The fragility curve is defined by five elevations for potential failure: levee toe, toe + 3 feet, half the levee height, crest – 3feet, and levee crest. One additional point was added at 1 foot above the crest where complete failure probability was assumed. The combined probability function factors in underseepage, through-seepage, stability, and engineering judgment, which is composed of other failure modes such as animal burrows and erosion. The levee fragility curves are shown in Figure 8-3.

For each stage ordinate derived from the hydrology function above, an associated probability of failure was then determined. If the stage was below the levee toe, the probability of failure was zero and if the calculated stage was above the maximum stage on the fragility curve, the failure probability was one. Since the original fragility curve doesn't extend beyond the top of the levee, the assumption that total failure occurs when the stage is 1 foot above the levee (or greater) implies that the levees are robust enough to withstand some overtopping flows without failure but that assurance is quickly diminished as the stage continues to rise.

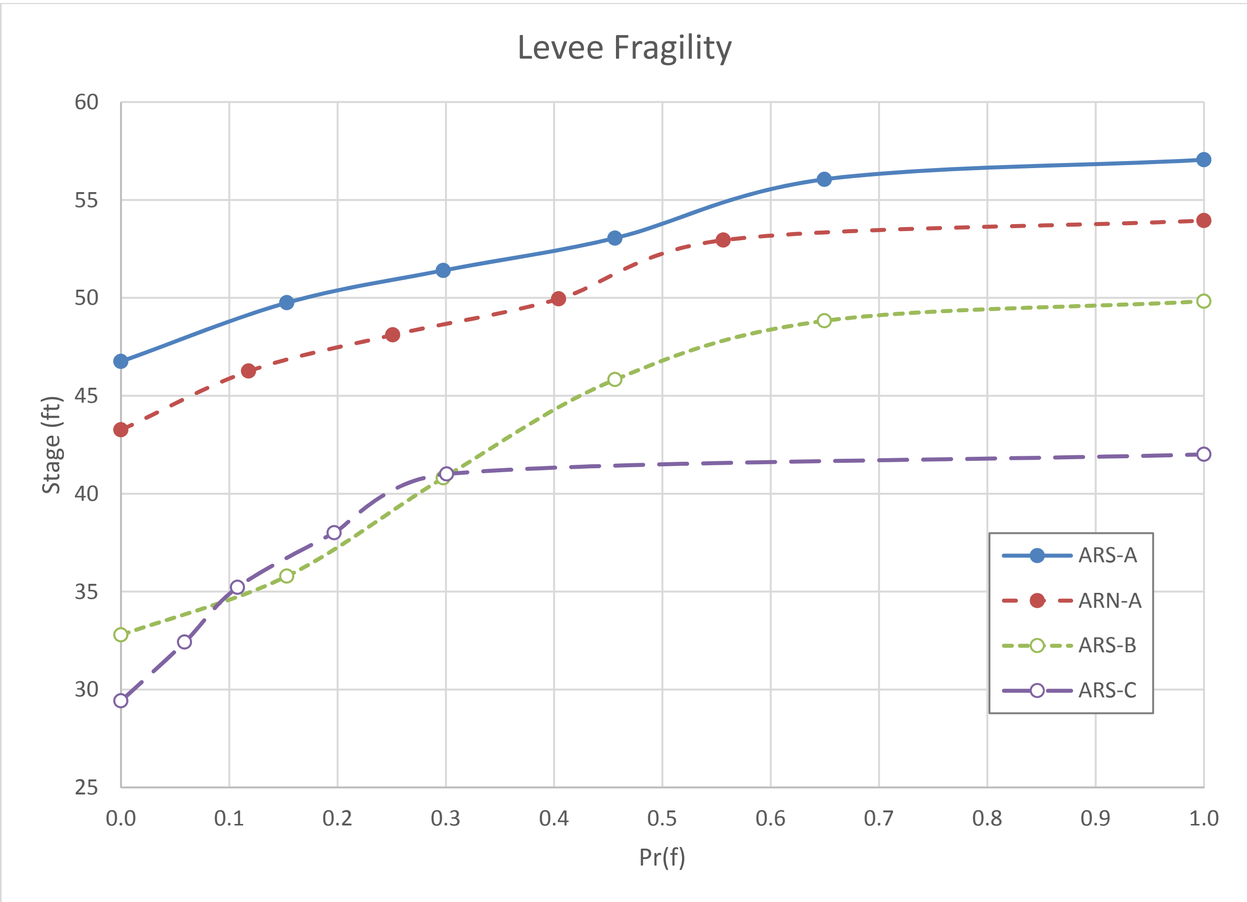


Figure 8-3: Levee Fragility Curve

8.2.4 Levee Failure Exceedence Probability

The compiled curves for the levee failure exceedence based upon probabilistic peak annual chance flows are shown in Figure 8-4 through Figure 8-7. Based upon the failure plots, it appears that the failure potential doesn't occur until flows are greater than the probabilistic 1/5 ACE flows, which is when the levee starts to become loaded. From there, the failure potential increases in step with the flow rate to its maximum failure potential. In general, for a given ACE, the selected WCP appears to have a lower or equal failure potential than the Existing Interim alternative, most significantly around the 1/200 ACE.

It is very important to acknowledge however, that a lot of construction activity on the levees, such as authorized levee improvements and floodway changes, have occurred in the past few years. Therefore, the fragility curves may not precisely represent present day field conditions. Additionally, there are several other factors that would have an impact in determining the true flood risk which were omitted for this exercise. The most significant of these factors is the uncertainty with each of the functions presented in this memorandum. To capture the true risk, a more thorough evaluation of risk and uncertainty should be performed.

8.2.4.1 ARS-A

American River South Index Point A (ARS-A) was evaluated because it is the established index point furthest upstream on the American River south levee and would present the greatest exposure to changes in flow rate. The other index points are further downstream and are influenced by tailwater effects on the Sacramento River (except ARN-A) which would ease channel velocities.

The levee becomes loaded at about 1/20 ACE (Figure 8-4) where the flow rate would be about 95,000 cfs.

The selected WCP has a lower failure potential than the Existing Interim alternative for all ACE instances implying that for this index point, the selected WCP would reduce the levee failure risk.

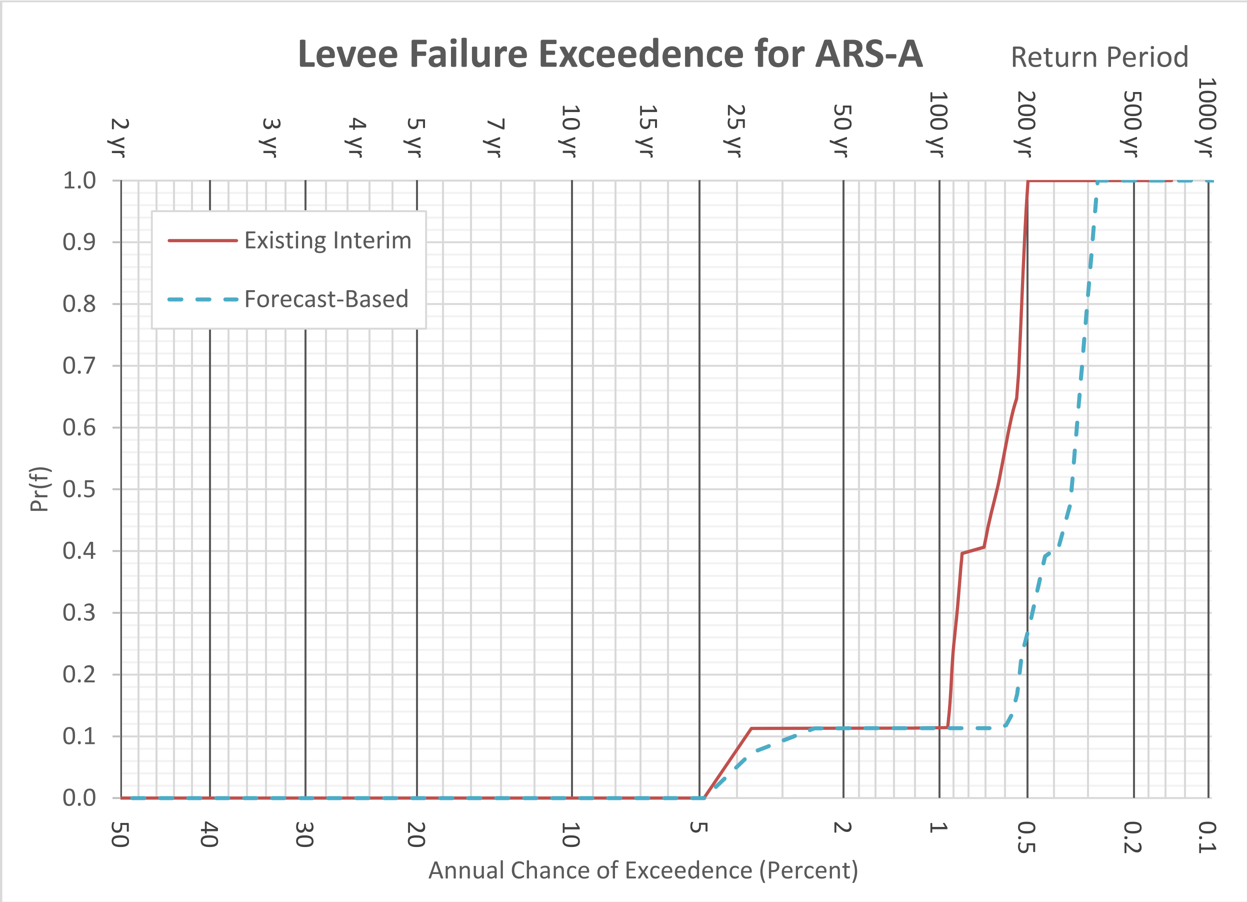


Figure 8-4: Levee Failure Exceedence for ARS-A (River Mile 8.92)

8.2.4.2 ARN-A

American River North Index Point A (ARN-A) was evaluated because it is the established index point furthest upstream on the American River north levee and, similar to ARS-A, would present the greatest exposure to changes in flow rate. ARN-A is 1 mile downstream of ARS-A, but is on the opposite levee. This location is also where the channel begins to reduce in width and begin turning around the bend at Campus Commons, which increases channel velocities.

The levee becomes loaded at about 1/16 ACE (Figure 8-5) where the flow rate would be about 85 kcfs.

The selected WCP has a lower failure potential than the Existing Interim alternative for all ACE instances, implying that for this index point, the selected WCP would reduce the levee failure risk.

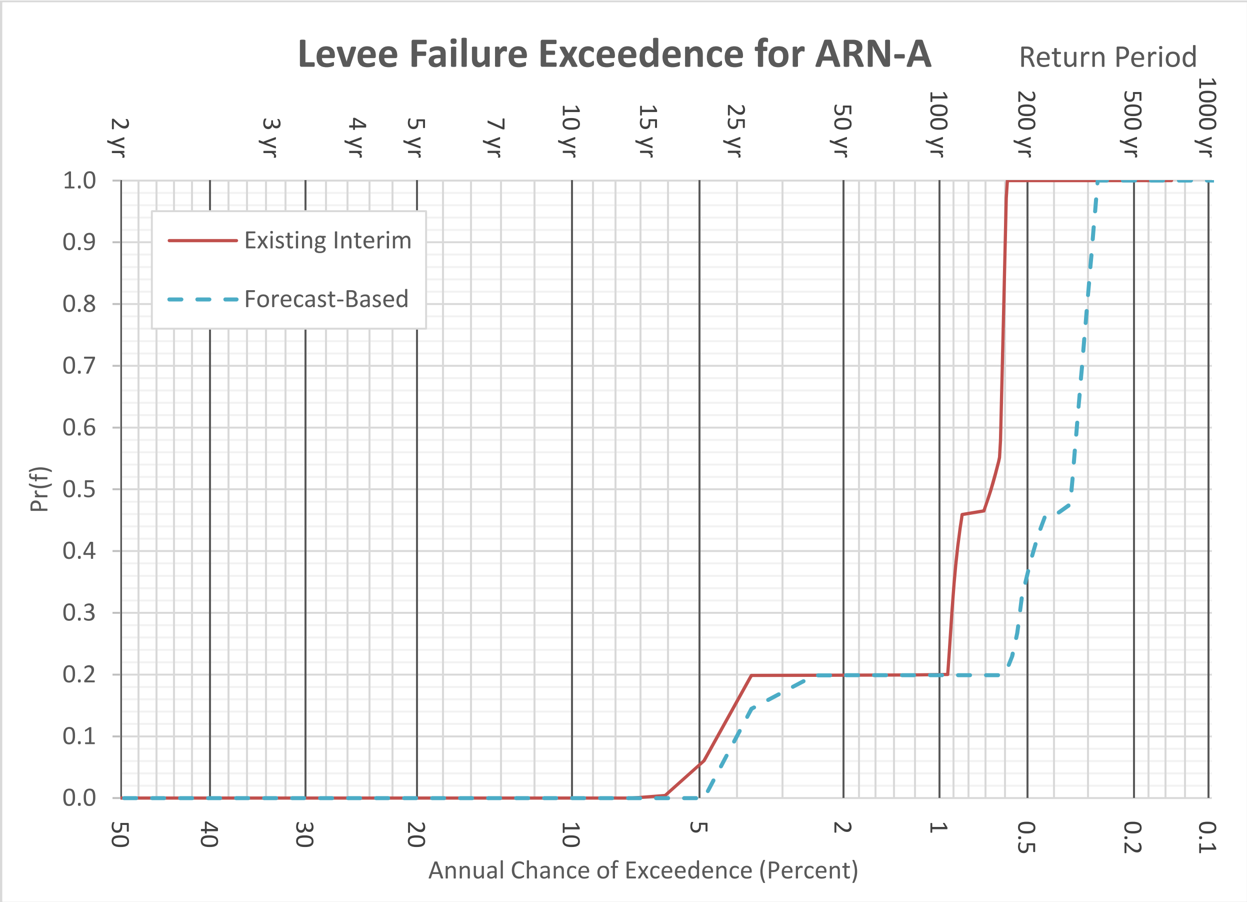


Figure 8-5: Levee Failure Exceedence for ARN-A (River Mile 7.83)

8.2.4.3 ARS-B

American River South Index Point B (ARS-B) was evaluated because it is close to the historic erosion site which severely damaged the riverside levee face during the 1986 flood.

The levee toe becomes loaded when the American River is flowing at about 65 kcfs. Under the Existing Interim alternative this would be about 1/11 ACE (Figure 8-6) and under the selected WCP it would be at about 1/7 ACE. Regardless of flow rate, overtopping at this location is not expected, however, because of upstream losses and because the levee is more elevated above the design water surface than for other index points to account for increased stages from the Sacramento River tailwater.

Although the selected WCP would reduce the failure potential for events smaller (in terms of probability) than 1/15 ACE (flows exceeding 80 kcfs.), it would come at the expense of a slightly greater failure potential for the more common events between 1/7 and 1/15 ACE. Essentially, this is the flow range between 65 kcfs when the levee toe is loaded up to 80 kcfs where JFP regulation begins to show improvement.

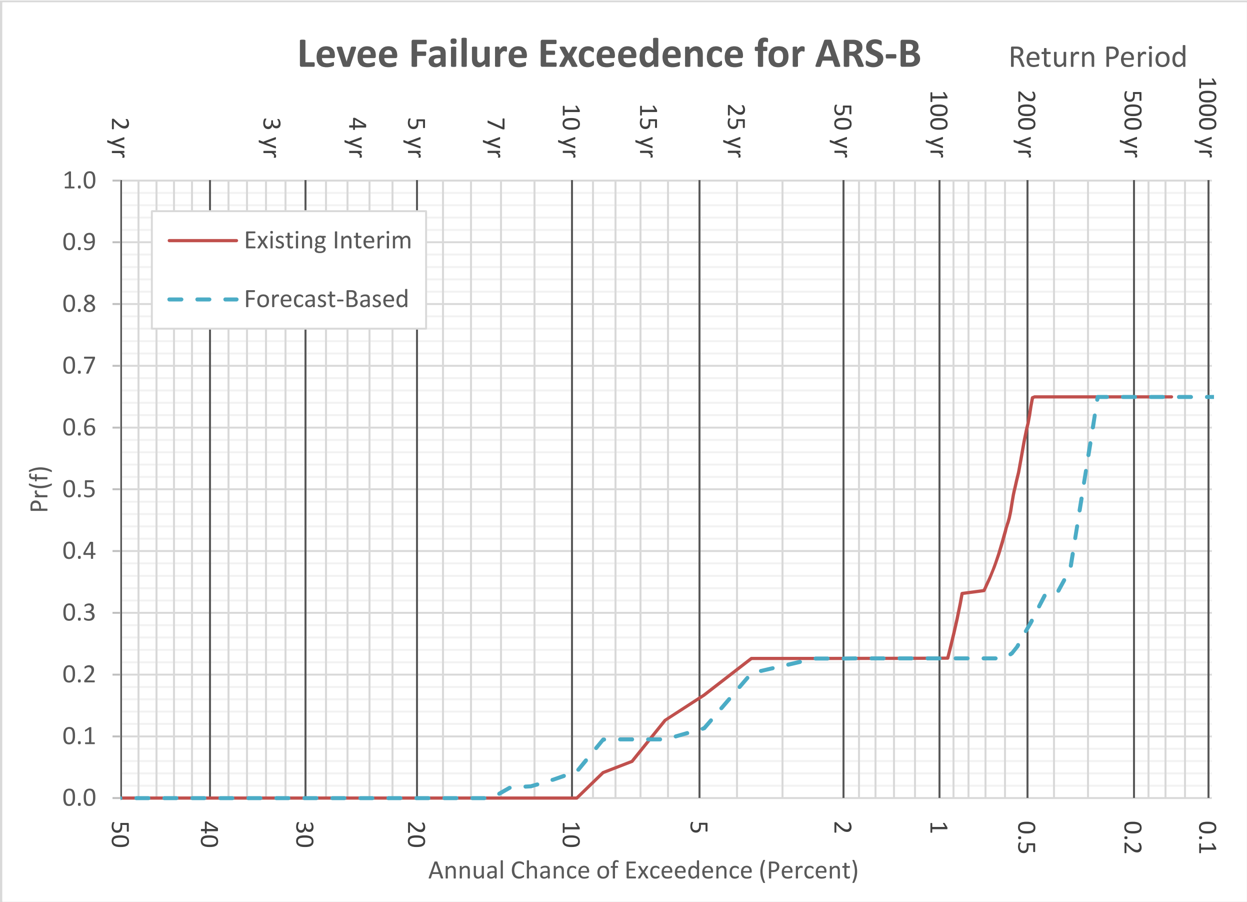


Figure 8-6: Levee Failure Exceedence for ARS-B (River Mile 4)

8.2.4.4 ARS-C

American River South Index Point C (ARS-C) was also evaluated and best considers tailwater impacts to the loading potential.

At this location, the levee toe is loaded most often starting at about 1/3 ACE (Figure 8-7) when the flows are about 31 kcfs. Similar to the reasoning described for ARS-B, the failure potential is reduced under the selected WCP for extreme events where JFP regulation is able to maintain flows of 115 kcfs beyond about 1/106 ACE. Leading up to that exceedence interval, however, there is a very minor difference in levee failure potential between the two alternatives. Furthermore, it may be even more indistinguishable when the Sacramento River has high stages, independent of Folsom Dam operations.

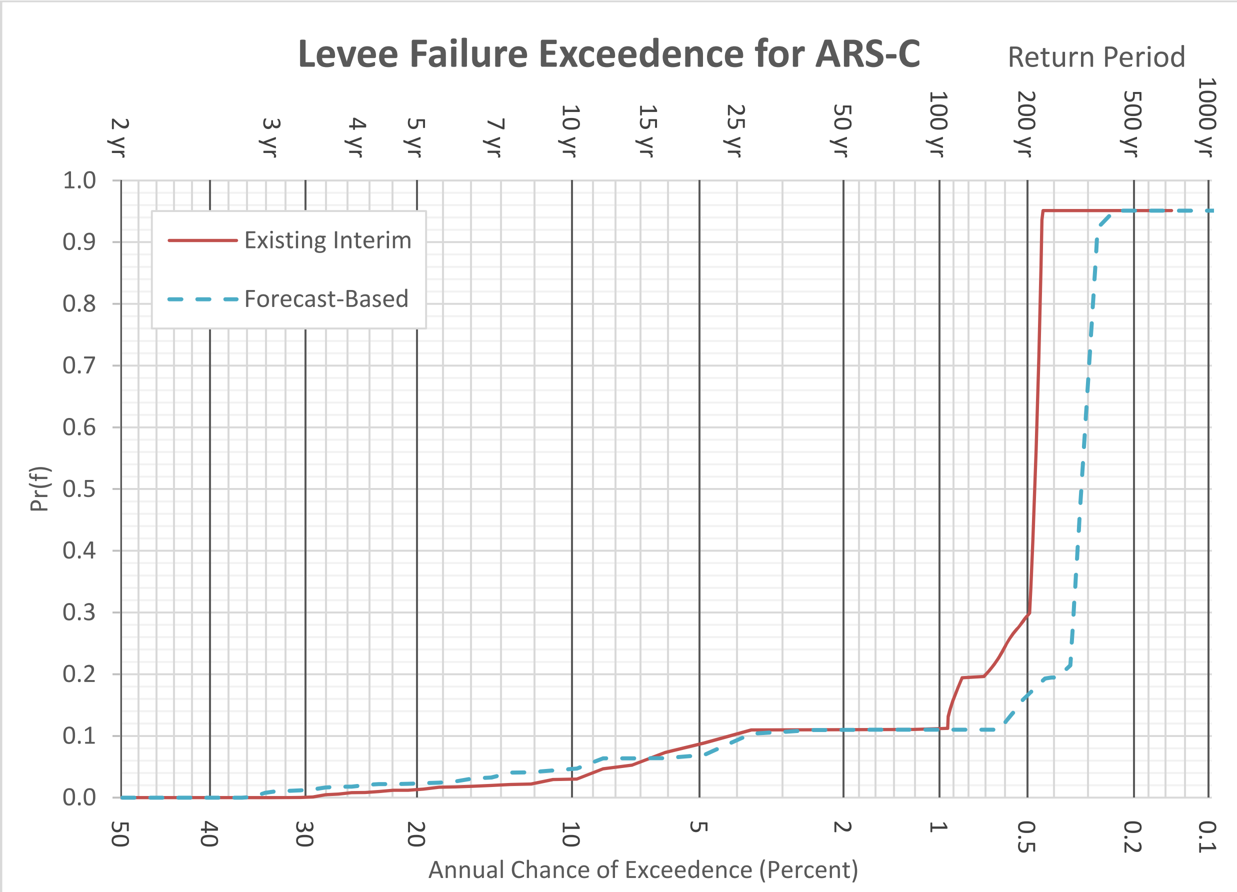


Figure 8-7: Levee Failure Exceedence for ARS-C (River Mile 1.5)

8.2.4.5 NAT-I

Natomas Reach I (NAT-I) was also evaluated, but the levee fragility curve was so robust and the levee is high enough that failure potential was essentially unchanged between alternatives. Under both project alternatives, the levee failure potential didn't exceed 0.0826.

8.3 Downstream Erosion Effects

8.3.1 Erosion Assessment Purpose and Background

The purpose of the Lower American River (LAR) erosion assessment was to assess the relative changes to flood risk and environmental impacts from erosion of the channel bed (i.e., channel incision) and banks (i.e., lateral erosion) between existing and alternative future operation of Folsom Dam. The study area for the erosion assessment is approximately 22 miles of the LAR between Nimbus Dam and the confluence with the Sacramento River (Figure 8-8).

After gold was discovered in the American River in 1849, subsequent hydraulic mining in the American River Watershed in the 1800s caused 15 to 20 feet of aggradation along the LAR, reducing its flood flow capacity and exacerbating frequent flooding in the area. The population along the banks of the LAR grew rapidly and has continued to grow within the greater Sacramento metropolitan area that borders both banks of the LAR. To protect property and lives from floods, levees were constructed, expanded, and strengthened along the lower half of the

LAR between Folsom Dam and the Sacramento River. In addition, dams throughout the region were constructed to regulate flow and provide flood risk management to multiple populations at risk from flooding, including the Sacramento metropolitan area. This included construction of dams on the American River: Folsom Dam and Nimbus Dam. The dams eliminated the supply of upstream sediment, resulting in significant erosion of the hydraulic mining debris within the LAR, as shown in Figure 8-9. This lack of additional sediment from upstream has contributed to armoring of the channel bed and erosion of the banks and levees.
Further erosion along the LAR could impact:

1. Riparian habitat
2. Spawning gravel
3. Levees
4. Soil supporting bridges and other infrastructure

Each of these erosion concerns are assessed as they relate to alternative future operation considered for Folsom Dam. The amount of erosion generally increases with increasing magnitude, duration, and frequency of discharge. Alternative 1 and Alternative 2 operations could potentially result in increased long-term erosion compared to Existing Interim operation.

The overarching goal of the WCM Update project is to minimize downstream flood risk. However, since minimizing downstream flood risk could also degrade habitat, the erosion analysis was developed to address both risk of levee failure (flood risk) and environmental impacts.

To assess downstream flood risk, it is important to 1) balance the risk of levee breach from overtopping flows with the risk of levee breach from erosion and 2) determine whether erosion will lead to increased maintenance rather than result in a levee breach. Chronic erosion that does not result in a levee breach can be repaired but results in higher maintenance costs. A levee breach during a flood event can lead to catastrophic damages and loss of life. In addition, there is a lot of uncertainty with estimating erosion, and the differences between existing operations and alternative operations could well be within the range of the natural uncertainty of the analysis and may not represent a valid statistical difference. The results of the erosion assessment need to be evaluated with all of these issues in mind.

To assess the effects on the environment, the long-term impacts from changing operation of Folsom Dam are evaluated using inflows into Folsom Lake developed from historical hydrology data. The long-term assessment analyzed the relative changes to erosion of riparian habitat, spawning gravel, levees, and bridges and other infrastructure caused by implementing alternative operations. Both the erosion flood risk assessment and the long-term environmental impact assessment are discussed further in the following sections.

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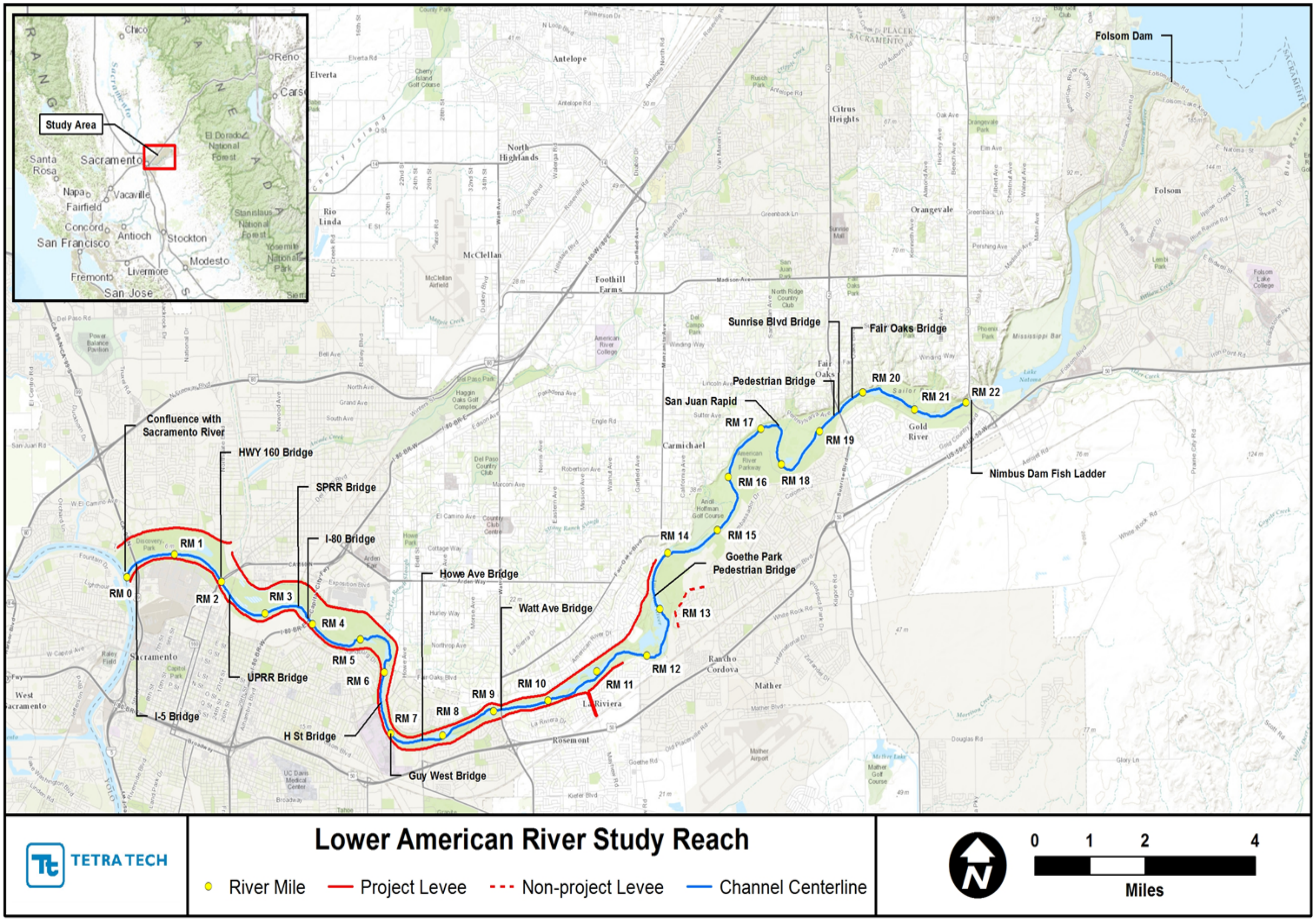


Figure 8-8: LAR Project Location and Area Map (Tetratech, 2015)

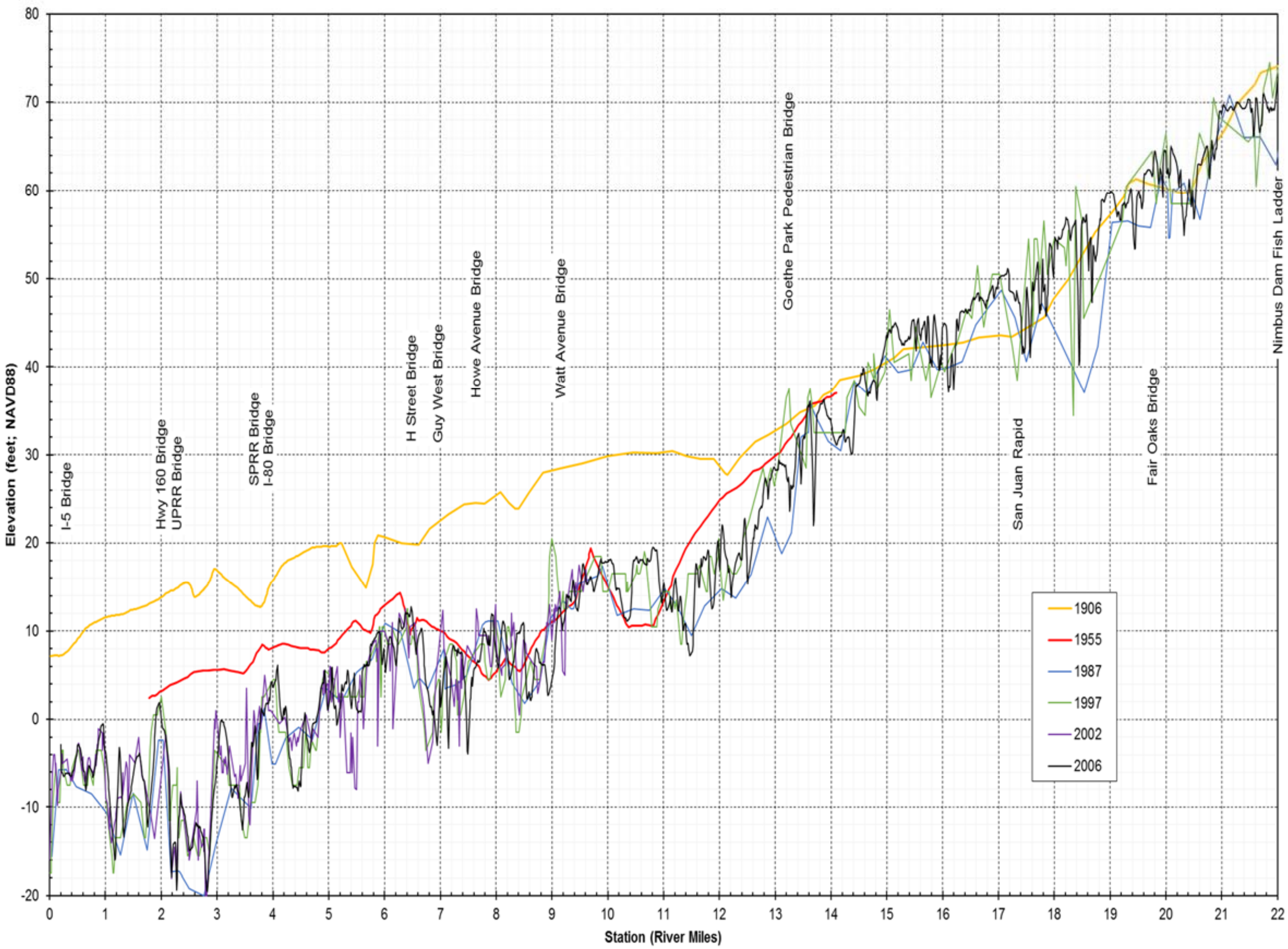


Figure 8-9: Historical Channel Bed Profile of the LAR Showing Significant Degradation since Deposition of the Hydraulic Mining Debris (Tetratech, 2015)

8.3.2 Long-Term Erosion Assessment Methods

The erosion assessment builds on past performance and previous erosion assessments. It compares predicted future erosion due to changes in Folsom Dam operations (Alternative 1 operation and Alternative 2 operation) to predicted future erosion from current Folsom Dam operations (Existing Interim operations). The primary objective of this analysis is to assess the relative difference in the amount of erosion between Existing Interim operations and alternative operations, not to determine absolute erosion volume/distance. The methods used for the analysis include:

1. Reviewing past erosion-related levee performance and erosion assessments
2. Estimating the potential for channel widening
3. Modeling sediment transport using the HEC-6T software
4. Estimating bridge pier scour using FHWA HEC-18
5. Comparing existing and with-project Folsom Dam discharge distributions

8.3.2.1 Reviewing Past Erosion Related Levee Performance and Erosion Assessments

Past levee performance and erosion assessments provide important information and context to assess erosion differences between existing and with-project Folsom Dam operation. For example, past levee erosion in an area can indicate the area is more prone to erosion during future floods. *The American River Watershed Common Features General Reevaluation Report (ARCF GRR) Erosion Protection Report* (USACE, 2014a) provides a concise summary of the past work performed regarding understanding and predicting the stability of the Lower American River with regards to erosion and the capability of the leveed reaches of the LAR to convey/contain flood releases from Folsom Dam. Additionally, this report provides the rationale for the proposed erosion protection features recommended by the ARCF GRR study. This provides important context for the erosion assessment for the Manual Update. For additional information on past performance and erosion assessments see Corps (2014a).

8.3.2.2 Estimating the Potential for Channel Widening

Estimating channel widening provides important information on erosion risks to riparian habitat, levees, and other infrastructure that could be threatened by channel widening. Because the amount of channel widening varies spatially, the LAR was sub-divided into ten geomorphic sub-reaches with similar geomorphic characteristics (see Figure 8-11). The channel widening analysis estimates the rate of channel widening using a sediment-accounting algorithm. The algorithm is dependent on the supply and size of sediment from upstream, the availability of sediment from bank erosion, the erodibility of bank material, and the sediment transport capacity of the channel. Some of these factors could change under alternative conditions. The rate of channel widening is determined by estimating the potential magnitude of widening in each reach by estimating bank erosion rates over an 81-year period of record for the Existing Interim and Alternative 1 operations. A sensitivity analysis on the channel widening computations was conducted by varying the estimated vertical degradation of the channel (i.e., adjusting the longitudinal profile developed into Alternative Profile 1 and Alternative Profile 2 as shown on Figure 8-10), the threshold for incipient motion of the sediment (Shields Parameter), and the downstream stage. Three scenarios were developed which represent the highest reasonable channel widening (scenario 1), the lowest reasonable channel widening (scenario 2), and an intermediate amount of channel widening (scenario 2) as shown in Table 8-1. The results of the channel widening analysis indicate which geomorphic sub-reaches may be at risk of increased channel widening for Alternative 1 operation relative to Existing Interim operation. The results inform the risk from lateral erosion to riparian habitat, levees, and other infrastructure from implementing Alternative 1 relative to Existing Interim operations. For additional details on the channel widening analysis, see Tetratech (2015).

Table 8-1: Summary and Definition of Variables used to Designate the Three Sensitivity Analysis Scenarios used for the Widening Analysis of the Lower American River

Scenario	Channel Bed Profile	Downstream Rating Curve	Shields Parameter
Scenario 1	Existing Profile	Lower Curve	0.03
Scenario 2	Alternate Profile 2	Expected Curve	0.045
Scenario 3	Alternate Profile 1	Higher Curve	0.06

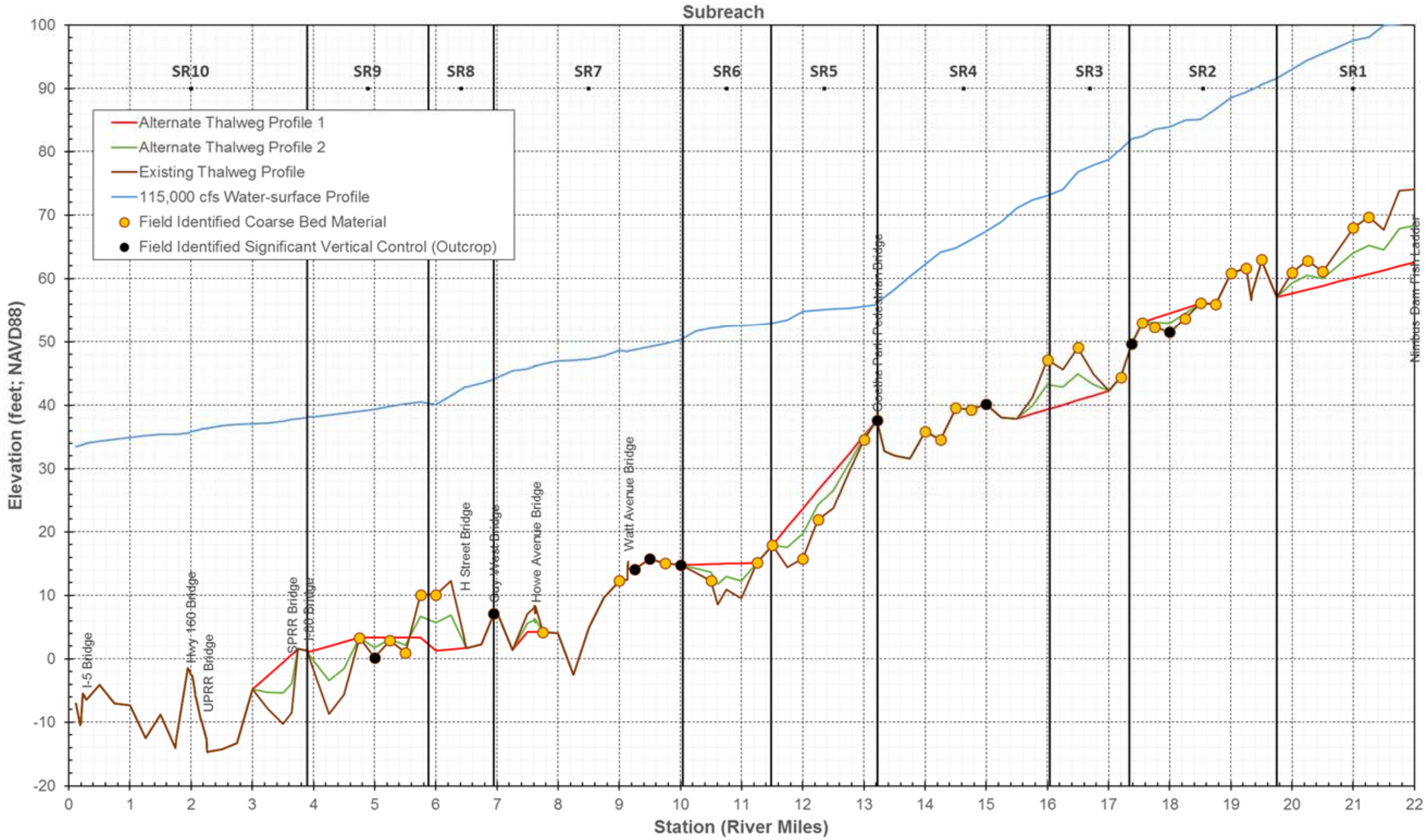


Figure 8-10: Existing Channel Bed Profile of the Lower American River Showing Alternate Channel Bed Profiles to Support the Sensitivity Analysis of Channel-Widening Potential

8.3.2.3 Modeling Sediment Transport Using the HEC-6T Software

Estimating vertical degradation and changes to the gradations of the LAR bed provides important information to assess the change in erosion risk to spawning gravel, riparian habitat, levees, and other infrastructure from Alternative 1 operation relative to Existing Interim operation. Vertical degradation can lead to high steep banks that erode, causing channel widening, which, in turn, could threaten riparian habitat, levees, and other infrastructure. In addition, the vertical degradation can lead to coarsening of the bed, which could negatively impact spawning gravel. The vertical degradation and LAR bed gradation changes were estimated using the HEC-6T sediment transport model for Existing Interim and Alternative 1 operations. The model was developed from an existing HEC-6T model but updated to include new 3D stratigraphic mapping and erosion testing of erosion-resistant material present in portions of the channel. The model was verified by comparing results to past observed changes in the bed. A sensitivity analysis of the model was conducted by widening the channel by 50 and 100 feet. The results of the HEC-6T models indicate areas of increased aggradation, degradation, and loss of spawning gravel. Comparison of results from the Existing Interim operation and Alternative 1 operation inform the erosion risk to riparian habitat, spawning gravel, levees, and other infrastructure from implementing Alternative 1 operation. For additional details on the HEC-6T modeling, see NHC (2015).

8.3.2.4 Estimating Bridge Pier Scour Using FHWA HEC-18

Estimating bridge scour provides important information to assess the change in erosion risk to bridges and similar infrastructure from implementing Alternative 2 operation. A bridge scour analysis was conducted using Federal Highways Administration Hydrologic Engineering Circular 18 (FHWA HEC 18) to estimate changes to bridge pier scour on selected LAR bridges relative to Existing Interim operations due to implementing Alternative 2 operation. The analysis estimates scour depths within a 73-year period of record from 1929 to 2002 on selected bridge piers along the LAR, downstream from Nimbus Dam. The bridges selected include (from upstream to downstream landmarks): the Watt Avenue Bridge, Howe Avenue Bridge, and H Street Bridge (see Figure 8-12). These bridges are selected because they have available erosion test data needed to conduct the computations, the channel in the vicinity of the bridges has exposed cohesive sediment, and the bridges are located in a reach of high velocities. The piers selected for analysis are in the main channel subject to the most erosive flows, to provide a conservative estimate of potential scour depths. The analysis is necessary for computing scour for bridges in cohesive sediment where erosion can occur, but the erosion rate is often low. For bridges in non-cohesive sediment, such as sand and gravel located in the lower reaches, scour estimates are based on a design discharge, which is the same for both Existing Interim operation, Alternative 1 operation, and Alternative 2 operation. For additional details on estimating bridge scour, see Corps (2016f).

8.3.2.5 Comparing Existing Interim, Alternative 1, and Alternative 2 Operations Discharge Distributions

Comparing Existing Interim, Alternative 1, and Alternative 2 operations discharge distributions from Folsom Dam provides information to estimate the change in erosion risk to riparian habitat, spawning gravel, levees, and other infrastructure. This is important for the erosion assessment because the various analyses were developed at different times in the project and used different Existing Interim operation and Alternative 1 operation discharges from Folsom Dam. Alternative

2 was not used in the channel widening and sediment transport modeling as they were not available. The Folsom Dam discharges for Existing Interim operation used for the channel widening analysis is different than for the Existing Interim operation used for the HEC-6T modeling, and both are different than the final Alternative 1 operation Folsom Dam discharges. Alternative 1 operation Folsom Dam discharges are the same for both the channel widening analysis and the HEC-6T modeling, but are not the same as the final Alternative 1 operation Folsom Dam discharges. This makes comparison and application of the analysis results challenging. Comparison of flow distributions between the various operations used in the analysis was used to inform conclusions. Comparison of flow distributions was also utilized to incorporate the erosion protection measures recommended in the ARCF GRR into the erosion assessment.

Over an 81-year period of record, average daily discharges were grouped by roughly 10 kcfs increments to create a discharge frequency distribution for Existing Interim, Alternative 1, and Alternative 2 operations. This was done for the Folsom Dam discharges used in the various analyses as well as the final Folsom Dam discharges. These distributions were compared to show where changes to discharge magnitude, duration, and frequency may reduce or increase erosion for Alternative 2 operation compared to Existing Interim operation.

Erosion occurs when the erosive forces from flowing water are large enough for a long enough duration to overcome the resistive forces of the channel and/or banks. The discharge where erosion is estimated to begin is the critical discharge. Critical discharges for the channel and banks were developed for selected cross-sections based on the soil and bed material grain sizes, testing of the erosion resistance of the soil, and geologic mapping. The change in the total number of days (for the entire period of record) above the critical discharge is used to estimate if a cross-section is potentially impacted by additional erosion for Alternative 2 operation compared to Existing Interim operation. The percent of each geomorphic sub-reach potentially impacted by erosion was estimated. “Potentially impacted” is defined as increased erosion by implementing Alternative 2 operation compared to continuing Existing Interim operation. The percent of the sub-reach potentially impacted by additional erosion was estimated as the percent of the sub-reach with cross-sections that could reasonably be expected to experience increased erosion relative to Existing Interim operation. The analysis was repeated, but this time assumed the erosion protection recommended by the ARCF GRR is constructed. This was conducted by updating the critical discharge for the cross-section where erosion protection is recommended by the ARCF GRR. This comparison shows the positive results of implementing the ARCF GRR recommendations.

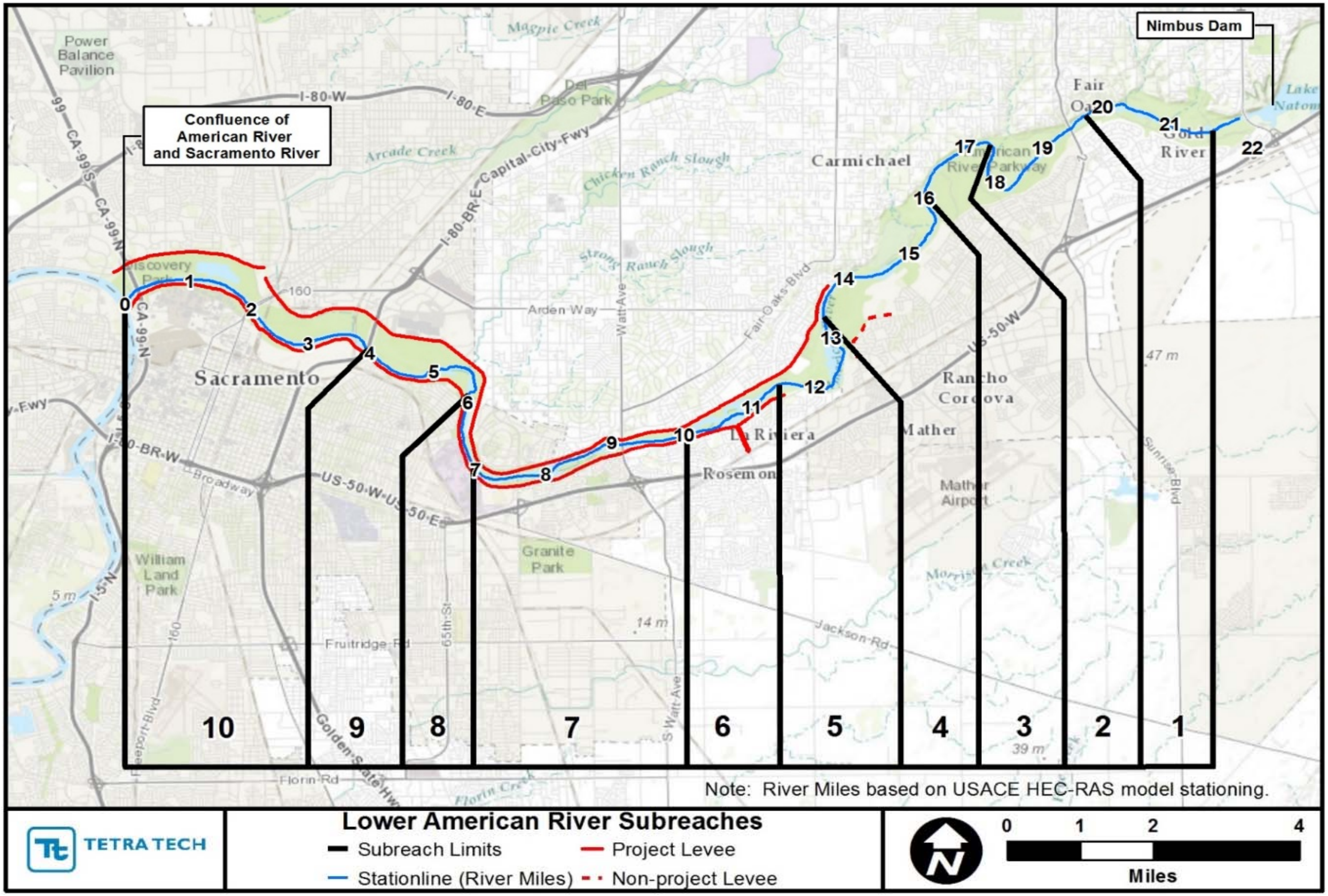


Figure 8-11: Geomorphic Sub-Reaches Used in the Channel Widening Analysis (Tetratech, 2015)

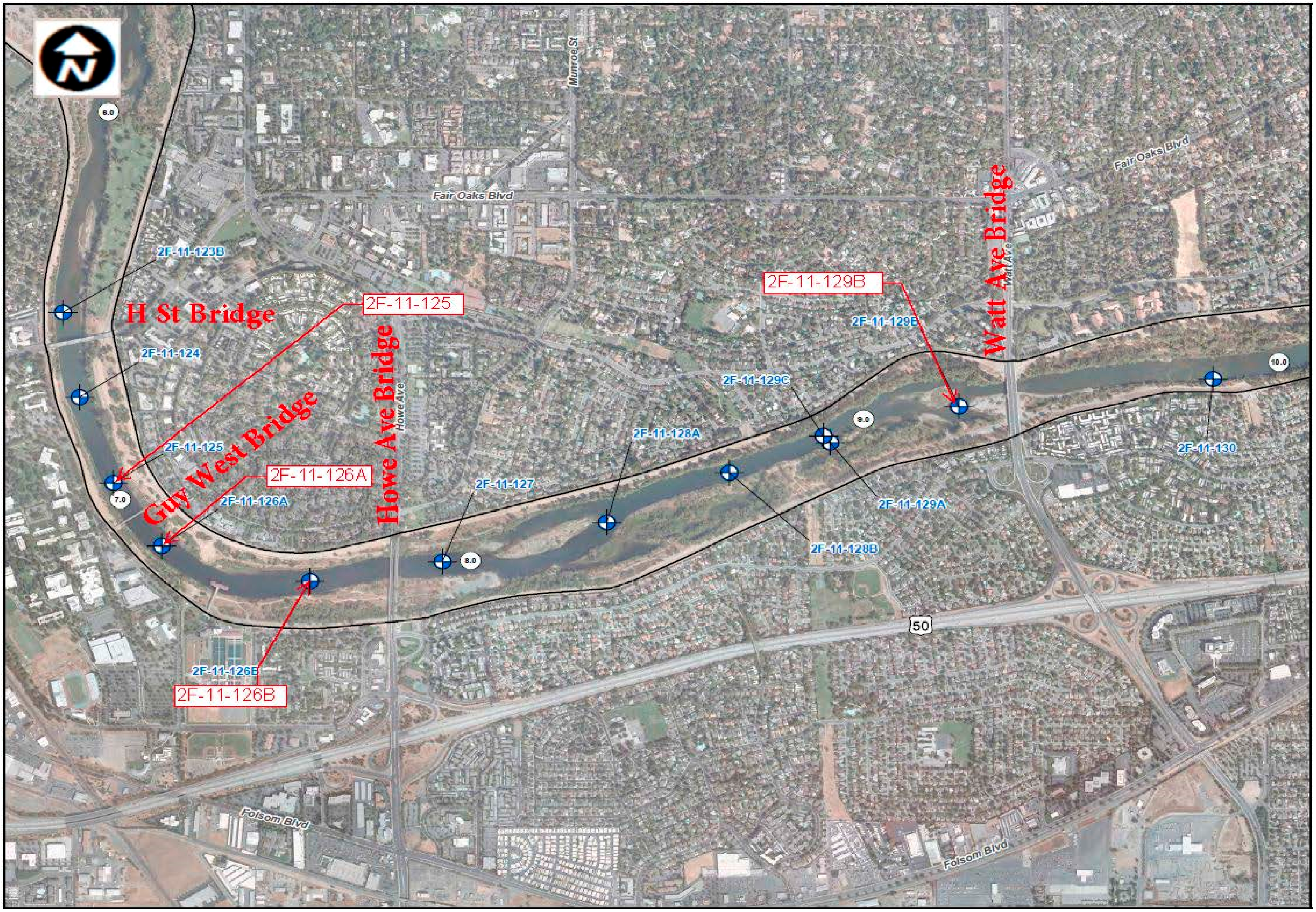


Figure 8-12: Bridge Scour Analysis Area Showing Bridges and Boring Locations (Corps, 2016)

8.3.3 Long-Term Erosion Assessment Results

8.3.3.1 Assessing Past Erosion Related Levee Performance and Assessments

A review of available information on past levee performance and LAR erosion assessments (USACE, 2014a) indicates that past studies have concluded the following:

1. The LAR levees have experienced levee distress from erosion during most of the major flood events in the past.
2. The LAR has experienced near impending levee failure from erosion that was not visible until the water receded (Figure 8-13).
3. Erosion on the LAR has been observed for discharges as low as 7,000 cfs.
4. While portions of the channel bed may have stabilized vertically, the need for bed protection to prevent additional degradation that could threaten the integrity of the levees should be monitored.
5. Failure to implement the recommended erosion protection measures proposed by the ARCF GRR will likely cause levee failure, catastrophic damages, and possibly loss of life.

The assessment of past levee performance and erosion assessments indicates a high risk of flooding from erosion-related failures for Existing Interim operation of Folsom Dam. Since the erosion assessment is comparing Existing Interim operation to alternative operation, the starting point for the comparison is high flood risk from erosion-related failures for Existing Interim operation.

8.3.3.2 Estimating the Potential for Channel Widening

The channel widening results for Existing Interim operation are shown in Figure 8-14 while the results for Alternative 1 operation are shown in Figure 8-15 with the differences plotted in Figure 8-16. These figures include results of a sensitivity analysis that varies input parameters. The Existing Interim operation and Alternative 1 operation Folsom Dam discharges used in the analysis are early versions and not the final versions. Alternative 2 was not developed at the time the analysis was conducted. The channel widening analysis reveals the following:

1. The channel widening analysis is not sensitive to differences in Existing Interim operation and Alternative 1 operation. The analysis is more sensitive to other input parameters, such as selection of the Shields Parameter.
2. Geomorphic sub-reach 8 could be at increased risk for systematic channel widening for Alternative 1 operation compared to Existing Interim operation, although results are inconsistent between scenarios.
3. Geomorphic sub-reaches 1 through 4 and 7 could also experience some systematic channel widening for Alternative 1 operation compared to Existing Interim operation, but results are inconsistent similar to sub-reach 8.
4. Geomorphic sub-reaches 5 through 7 have erosion resistant banks and/or a low-flow channel that is imbedded in the erosion resistant formation. This provides an erosion-resistant bank toe, keeping any channel widening relatively low to imperceptible.
5. Geomorphic sub-reaches 9 through 10 are located in wider portions of the channel with significant overbank flood plains. These wider reaches have lower velocities and relatively low to imperceptible channel widening.

6. Sub-reaches 1 through 4 are bounded by relatively erosion-resistant banks which contribute significantly to the reduced erosion risk in these sub-reaches compared to other reaches.
7. Mid-range discharges (e.g. 20 to 100 kcfs) may contribute to most of the channel widening for some locations along the LAR (see Figure 8-17)

8.3.3.3 Modeling Sediment Transport Using the HEC-6T Software

The HEC-6T modeling results are shown in Figure 8-18, Figure 8-19, Figure 8-20, and Figure 8-21. The Existing Interim operation and Alternative 1 operation Folsom Dam discharges used in the analysis are early versions and not the final versions. The Existing Interim operation Folsom Dam discharges used in this analysis are also different than what is used in the Channel Widening Analysis. However, Alternative 1 Folsom Dam discharges are the same for the channel widening analysis and the HEC-6T modeling. Alternative 2 was not developed at the time the analysis was conducted. The HEC-6T modeling reveals the following:

1. Changes to channel invert profile of the LAR are not sensitive to differences between Existing Interim operation and Alternative 1 operation.
2. Changes to channel invert profile and gradations along the LAR are not sensitive to channel widening of up to 100 feet.
3. The presence of an erosion-resistant hard surface will likely reduce degradation for portions of the channel, such as between river miles (RM) 7 and 11.5.
4. Upstream of RM 13, long-term degradation is expected for both Existing Interim operation and Alternative 1 operation with negligible differences.
5. The furthest downstream reaches experience a gradual aggradational trend for both Existing Interim operation and Alternative 1 operation.
6. The middle reaches may experience very little vertical change for both Existing Interim operation and Alternative 1 operation.
7. Loss of gravel-sized material is expected upstream and in the vicinity of the Goethe Park Pedestrian Bridge around RM 13 for both Existing Interim operation and Alternative 1 operation.
8. The largest most infrequent discharges cause the most degradation for the upstream reaches (at and above RM 13).
9. The long-term aggradational trend in the furthest downstream reaches is not well correlated to the largest most infrequent discharges as is seen in the upstream reaches, and occurs for smaller, more frequent, discharges, too.

8.3.3.4 Estimating Bridge Pier Scour Using FHWA HEC-18

The results of the Bridge Pier Scour analysis using FHWA HEC-18 procedures for cohesive sediment are summarized in Table 8-2 the Existing Interim operation and Alternative 2 operation Folsom Dam discharges are the final versions, as the analysis was conducted after these were available. Table 8-2 reveals that the bridges are not expected to be substantially impacted by changing operation at Folsom Dam from Existing Interim operation to Alternative 2 operation.

Table 8-2: Bridge Scour analysis Results for Selected Bridges on the LAR for Existing Interim Operation and Alternative 2 Operation. From Corps (2016f).

Bridge Name	Existing Interim Operation Erosion (ft)	Alternative 2 Operation Erosion (ft)
Watt Avenue Bridge	7.58	7.12
Howe Avenue Bridge	23.63	23.67
H Street Bridge	4.24	4.85

8.3.3.5 Comparing Existing Interim, Alternative 1, and Alternative 2 Operation Discharge Distributions

The Folsom Dam discharge frequency distribution used for the various erosion analyses are shown in Table 8-3 along with the final Existing Interim operation and Alternative 2 operation Folsom Dam discharge frequency distribution. From observing this table, it is evident that the Existing Interim, Alternative 1, and Alternative 2 Folsom Dam discharges developed from early ResSim models are not the same as final Folsom Dam discharges from final ResSim models. Therefore, professional judgment needs to be used when interpreting the results of the analysis. Comparison of Existing Interim operation and Alternative 2 operation discharge frequency distribution reveals the following:

1. Changes to flows less than 10 kcfs are insignificant (approximately 0 percent change).
2. There is a small increase in discharges in the 10 to 20 kcfs range (approximately a 17 percent increase in total number of days).
3. There is a substantial decrease in the frequency of flows in the 20 to 40 kcfs range (approximately a 40 percent decrease in total number of days).
4. There is a substantial increase in the frequency of flows in the 40 to 90 kcfs range (approximately 70 percent increase in total number of days).
5. There is a substantial decrease in the frequency of flows over 90 kcfs (approximately 40 percent decrease in total number of days), but the total number of days for these infrequent events is quite small.

It is unclear from these results if Alternative 2 operation will increase, decrease, or keep erosion the same relative to Existing Interim operation. However, only discharges above a critical discharge will cause erosion. The critical discharge (discharge at which erosion is estimated to begin) for each geomorphic sub-reach was estimated and results are summarized in Table 8-4. The percent of each sub-reach potentially impacted is shown in Table 8-5. “Potentially impacted” is defined as an increased number of days where average daily discharge is equal to or greater than critical discharge.

The critical discharge is computed for each cross-section in each reach. The number of days where the average daily discharge exceeds critical discharge is computed for the Existing Interim and Alternative 2 operations for each cross-section in each reach. The increased number of days above Existing Interim operations is computed for each cross-section. The total number of cross-sections where there is an increase in the total number of days is computed for each reach. The percent of the total length of each sub-reach where there is an increase in the total number of days is computed. This is based on the length each cross-section represents compared to the entire length of the reach.

This analysis was then updated to include implementation of the erosion protection recommended by the ARCF GRR and the results shown in Table 8-6. This was repeated for the decreased number of days as shown in Table 8-7 and Table 8-8. The difference between the percent of each reach impacted (more erosion) and percent of each reach improved (less erosion) was then computed for the existing rock and the ARCF GRR rock future condition (Table 8-9 and Table 8-10) The results reveal the following:

1. There is a wide range of critical discharges along the entire LAR, which is likely reflective of natural variability along the LAR.
2. Some areas of the LAR will likely not be impacted by Alternative 2 operation relative to Existing Interim operation, and some areas may experience less erosion.
3. Some areas of the LAR will likely be impacted by Alternative 2 operation relative to Existing Interim operation.
4. The left and right banks of sub-reaches, 2, 4, 5, 6, and 9 may experience more areas of increased erosion than areas of decreased erosion without the ARCF GRR erosion protection. Addition of the ARCF GRR erosion protection reduces this to sub-reaches 2, 4, 5 for both banks, and sub-reaches 6 and 9 for the right bank. Sub-reaches 2 and 4 have erosion-resistant banks, and any potential additional bank erosion is likely to result in relatively small amounts of bank retreat, based on historical observation. Sub-reaches 5 6, and 7 are within the ARCF GRR area and any need for additional erosion protection for this reach will be evaluated in detail during the design process and constructed as needed. Sub-reach 9 right bank levees are set back from the main channel (approximately 2,000 feet), and this analysis uses the discharge in the main channel from a 1D hydraulic model. So, while some additional bank erosion may be expected next to the main channel, it is not expected to have any impact on the right bank levee for sub-reach 9.
5. This analysis is based on simplifying assumptions, such as 1D model versus 3D reality, using a single value erosion parameter for very general soil type descriptions, calibrating to estimated values of bank retreat that are on the high side of the reasonable range, and using the main channel flow to compute bank erosion even when the levees are set-back. There is a lot of uncertainty with this analysis
6. Implementation of the erosion protection recommended by ARCF GRR will reduce the risk of erosion-related levee failure (levees occur in sub-reaches 5 through 10).
7. Further erosion analysis is needed during implementation of the ARCF GRR to ensure that all portions of the levees at risk of erosion are adequately protected.

Table 8-3: Summary of Distribution of Average Daily Discharges for Channel Widening Analysis, HEC-6T Modeling, and Final Existing Interim Operation, Alternative 1 Operation, and Alternative 2 Operation

Discharge Range (kcfs)	Existing Interim Operation (Channel Widening) Frequency (# of Days)	Existing Interim Operation (HEC-6T) Frequency (# of days)	Alternative 1 Operation (Channel Widening & HEC-6T) Frequency (# of days)	Existing Interim Operation (Final) Frequency (# of days)	Alternative 2 Operation (Final) Frequency (# of days)
< 10	28,248	28,486	28,475	28,388	28,348
10 to < or = 20	946	750	849	830	967
20 to < or = 30	175	153	134	202	147
30 to < or = 40	112	121	40	109	40
40 to < or = 50	46	33	42	22	39
50 to < or = 60	18	13	10	8	15
60 to < or = 70	12	6	6	6	3
70 to < or = 80	5	7	2	4	11
80 to < or = 90	4	2	7	1	3
90 to < or = 100	3	3	1	2	1
100 to < or = 115	9	4	12	6	4

Table 8-4: Estimated Critical Discharge Summary by Sub-Reach

	Model Location		Left Bank			Channel Bed			Right Bank		
Sub-Reach	Upstream River Station	Downstream River Station	Q Critical Average (cfs)	Q Critical Max (cfs)	Q Critical Min (cfs)	Q Critical Average (cfs)	Q Critical Max (cfs)	Q Critical Min (cfs)	Q Critical Average (cfs)	Q Critical Max (cfs)	Q Critical Min (cfs)
SR1	22	19.753	91,101	>160,000	31,806	45,892	>160,000	9,200	91,101	>160,000	31,806
SR2	19.75	17.38	85,913	>160,000	54,444	29,895	118,000	3,686	85,913	>160,000	54,444
SR3	17.29	16.0833	78,671	158,333	33,056	31,255	43,500	14,400	78,671	158,333	33,056
SR4	16	13.22	105,205	>160,000	44,583	28,426	47,000	16,500	116,079	>160,000	44,583
SR5	13.216	11.5	29,429	>160,000	1,000	60,745	>160,000	2,300	29,429	>160,000	1,000
SR6	11.416	10.0833	77,833	>160,000	13,500	141,667	>160,000	73,000	77,833	>160,000	13,500
SR7	10	6.951	60,600	>160,000	500	76,791	>160,000	500	56,050	>160,000	500
SR8	6.948	5.91666	>160,000	>160,000	>160,000	33,490	51,000	1,625	54,563	>160,000	1,000
SR9	5.833	3.913	118,525	>160,000	13,200	108,563	>160,000	85,000	84,625	>160,000	1,778
SR10	3.894	0.115	94,957	>160,000	21,667	3,294	5,300	500	64,765	>160,000	21,667

Table 8-5: Summary of the Percent of the Total Sub-Reach Length Potentially Impacted by Implementing Alternative 2 Operation Relative to Existing Interim Operation, without ARCF GRR Bank Protection.

	Model Location				
Sub-Reach	Upstream River Station	Downstream River Station	Estimated % of Left Bank Potentially Impacted	Estimated % of Channel Potentially Impacted	Estimated % of Right Bank Potentially Impacted
SR1	22	19.753	28	28	28
SR2	19.75	17.38	45	21	45
SR3	17.29	16.0833	38	62	38
SR4	16	13.22	49	32	41
SR5	13.216	11.5	28	14	28
SR6	11.416	10.0833	60	20	60
SR7	10	6.951	31	62	38
SR8	6.948	5.91666	0	50	0
SR9	5.833	3.913	39	0	61
SR10	3.894	0.115	0	0	0

Table 8-6: Summary of the Percent of the Total Sub-Reach Length Potentially Impacted by Implementing Alternative 2 Operation Relative to Existing Interim Operation, with ARCF GRR Bank Protection

	Model Location				
Sub-Reach	Upstream River Station	Downstream River Station	Estimated % of Left Bank Potentially Impacted	Estimated % of Channel Potentially Impacted	Estimated % of Right Bank Potentially Impacted
SR1	22	19.753	28	28	28
SR2	19.75	17.38	45	21	45
SR3	17.29	16.0833	38	62	38
SR4	16	13.22	49	32	41
SR5	13.216	11.5	28	14	28
SR6	11.416	10.0833	0	20	60
SR7	10	6.951	0	62	8
SR8	6.948	5.91666	0	50	0
SR9	5.833	3.913	0	0	61
SR10	3.894	0.115	0	0	0

Table 8-7: Summary of the Percent of the Total Sub-Reach Length Potentially Improved by Implementing Alternative 2 Operation Relative to Existing Interim Operation, without ARCF GRR Bank Protection.

	Model Location				
Sub-Reach	Upstream River Station	Downstream River Station	Estimated % of Left Bank Potentially Impacted	Estimated % of Channel Potentially Impacted	Estimated % of Right Bank Potentially Impacted
SR1	22	19.753	39	50	39
SR2	19.75	17.38	19	43	19
SR3	17.29	16.0833	41	38	41
SR4	16	13.22	8	68	8
SR5	13.216	11.5	0	14	0
SR6	11.416	10.0833	24	20	24
SR7	10	6.951	35	23	36
SR8	6.948	5.91666	0	25	50
SR9	5.833	3.913	0	87	0
SR10	3.894	0.115	54	0	78

Table 8-8: Summary of the Percent of the Total Sub-Reach Length Potentially Improved by Implementing Alternative 2 Operation Relative to Existing Interim Operation, with ARCF GRR Bank Protection

	Model Location				
Sub-Reach	Upstream River Station	Downstream River Station	Estimated % of Left Bank Potentially Impacted	Estimated % of Channel Potentially Impacted	Estimated % of Right Bank Potentially Impacted
SR1	22	19.753	39	50	39
SR2	19.75	17.38	19	43	19
SR3	17.29	16.0833	41	38	41
SR4	16	13.22	8	68	8
SR5	13.216	11.5	0	14	0
SR6	11.416	10.0833	0	20	24
SR7	10	6.951	0	23	0
SR8	6.948	5.91666	0	25	0
SR9	5.833	3.913	0	87	0
SR10	3.894	0.115	0	0	78

Table 8-9: Summary of the Net Percent Change of the Sub-Reach Length Potentially Impacted by Implementing Alternative 2 Operation Relative to Existing Interim Operation, without ARCF GRR Bank Protection (positive values are increased erosion and negative values are decreased erosion)

	Model Location				
Sub-Reach	Upstream River Station	Downstream River Station	Estimated % of Left Bank Potentially Impacted	Estimated % of Channel Potentially Impacted	Estimated % of Right Bank Potentially Impacted
SR1	22	19.753	-11	-22	-11
SR2	19.75	17.38	26	-22	26
SR3	17.29	16.0833	-3	24	-3
SR4	16	13.22	41	-35	32
SR5	13.216	11.5	28	0	28
SR6	11.416	10.0833	36	0	36
SR7	10	6.951	-4	38	2
SR8	6.948	5.91666	0	25	-50
SR9	5.833	3.913	39	-87	61
SR10	3.894	0.115	-54	0	-78

Table 8-10: Summary of the Net Percent Change of the Sub-Reach Length Potentially Impacted by Implementing Alternative 2 Operation Relative to Existing Interim Operation, with ARCF GRR Bank Protection (positive values are increased erosion and negative values are decreased erosion)

	Model Location				
Sub-Reach	Upstream River Station	Downstream River Station	Estimated % of Left Bank Potentially Impacted	Estimated % of Channel Potentially Impacted	Estimated % of Right Bank Potentially Impacted
SR1	22	19.753	-11	-22	-11
SR2	19.75	17.38	26	-22	26
SR3	17.29	16.0833	-3	24	-3
SR4	16	13.22	41	-35	32
SR5	13.216	11.5	28	0	28
SR6	11.416	10.0833	0	0	36
SR7	10	6.951	0	38	8
SR8	6.948	5.91666	0	25	0
SR9	5.833	3.913	0	-87	61
SR10	3.894	0.115	0	0	-78

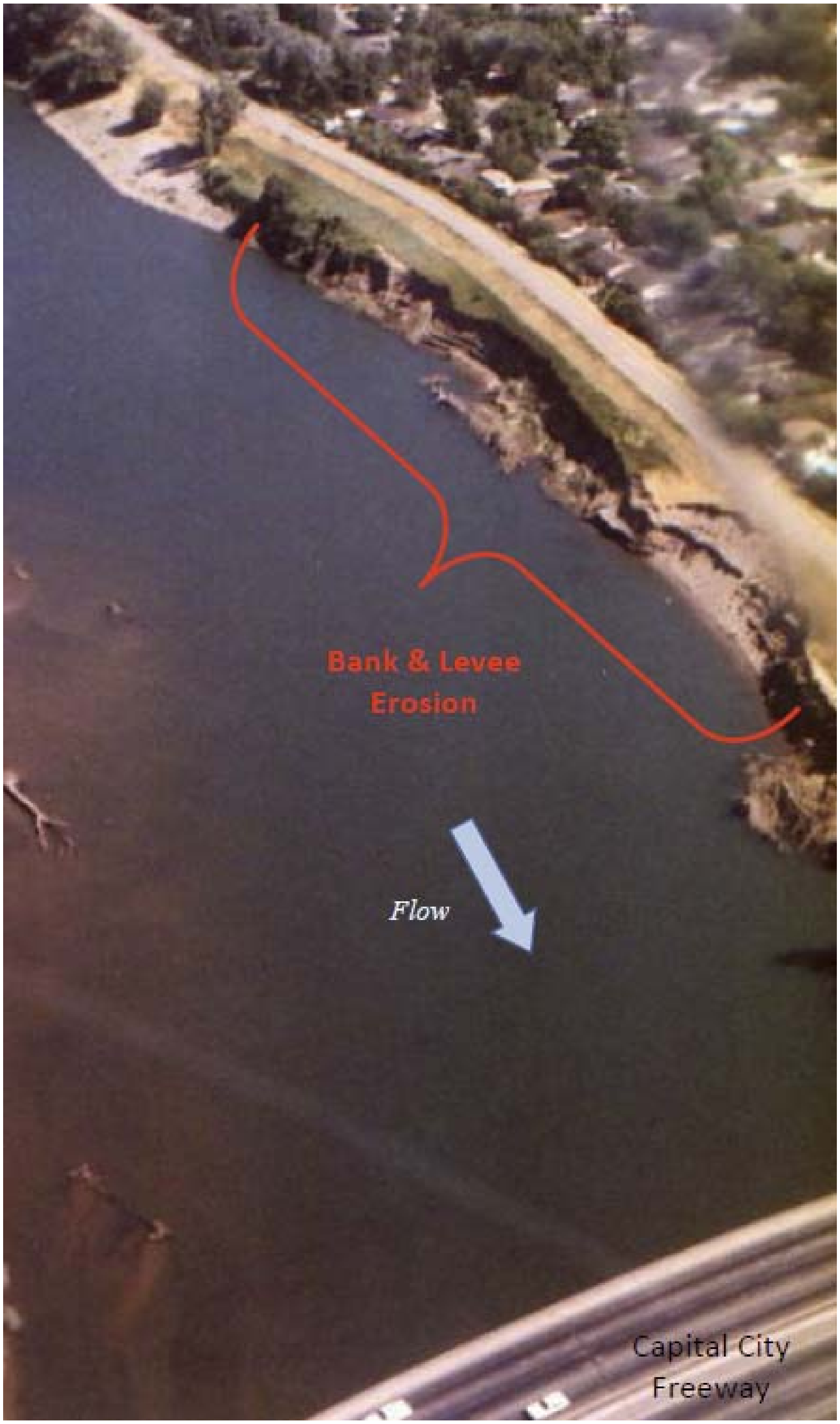


Figure 8-13: Bank and Levee Erosion Upstream of Business I-80 Bridges across the LAR after 1986 Floodwater Receded

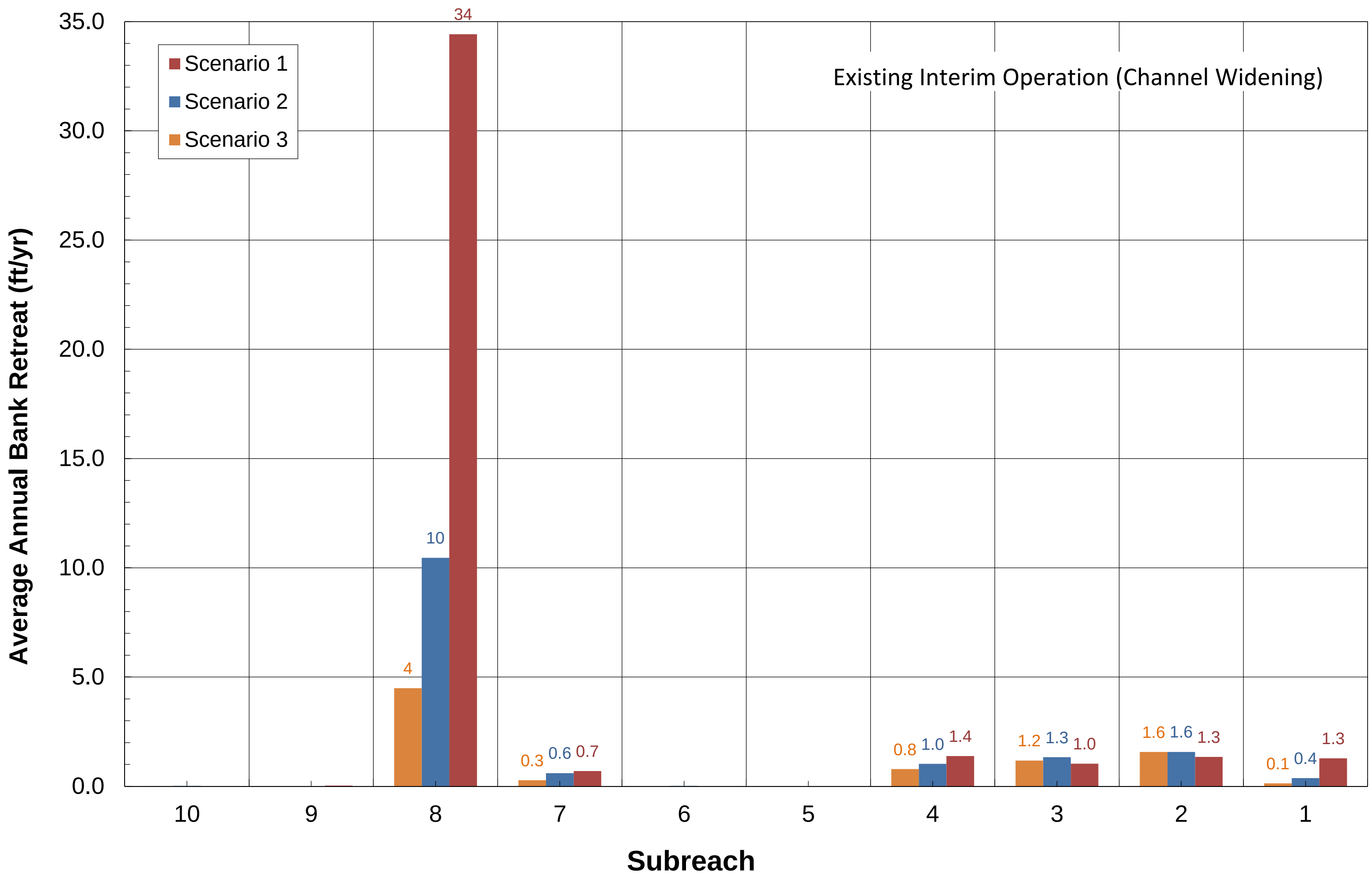


Figure 8-14: Sensitivity Analysis Results for Existing Interim Operation for Three Scenarios

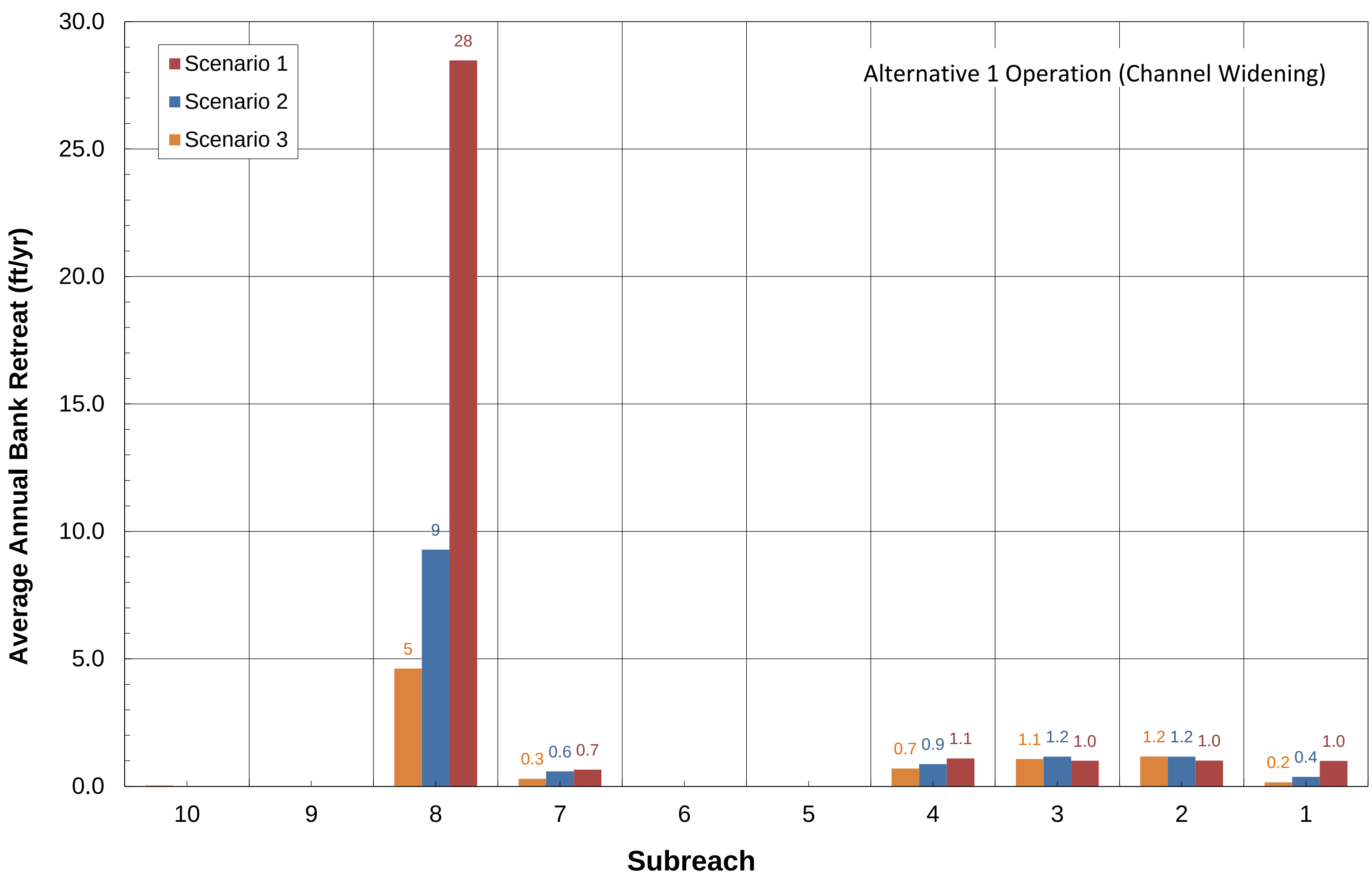


Figure 8-15: Sensitivity Analysis Results for Alternative 1 Operation for Three Scenarios

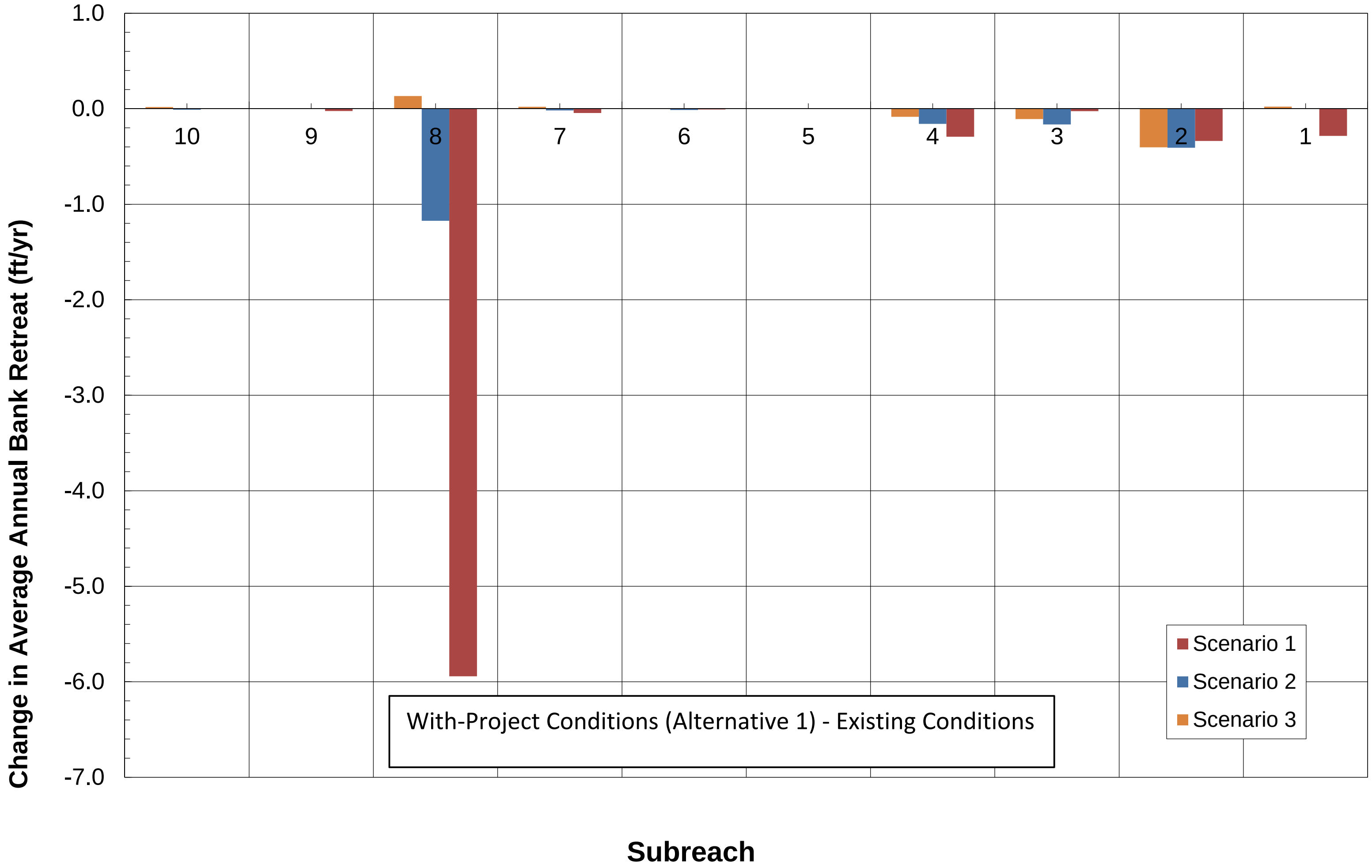


Figure 8-16: Change in Average Annual Bank Retreat from Existing Conditions to Alternative 1 Operations
Positive is increased average annual bank retreat of Alternative 1 compared to Existing Conditions.

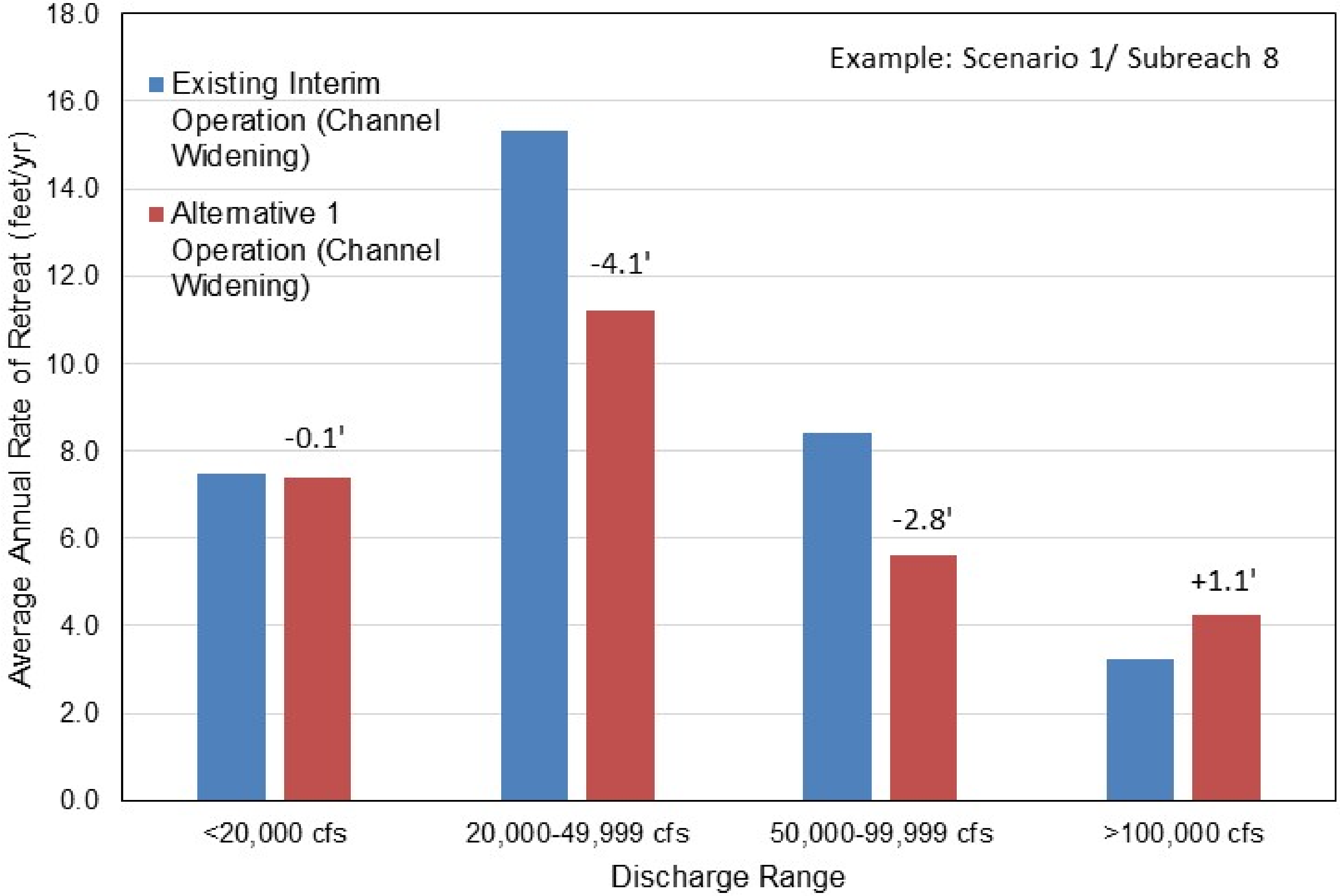


Figure 8-17: Example of Average Annual Bank Retreat Rates in Sub-reach 8 under Scenario 1 for a Series of Discharge Ranges for Existing Interim Operation and Alternative 1 Operation. (The increases above the Existing Interim operation are shown numerically as positive values in this figure.)

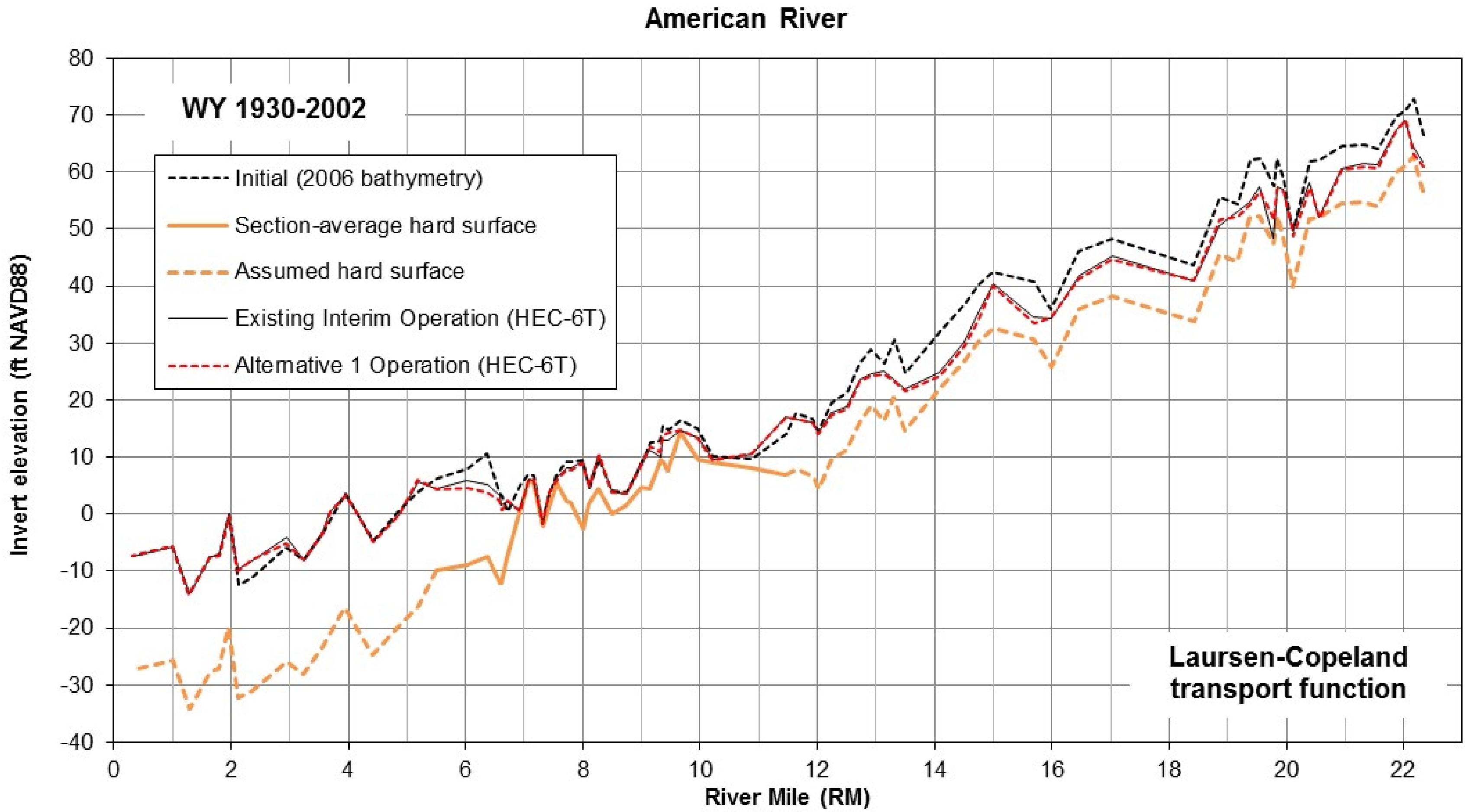


Figure 8-18: Comparison of Invert Profiles in American River Computed for Existing Interim Operation and Alternative 1 Operation (modified from NHC, 2015)

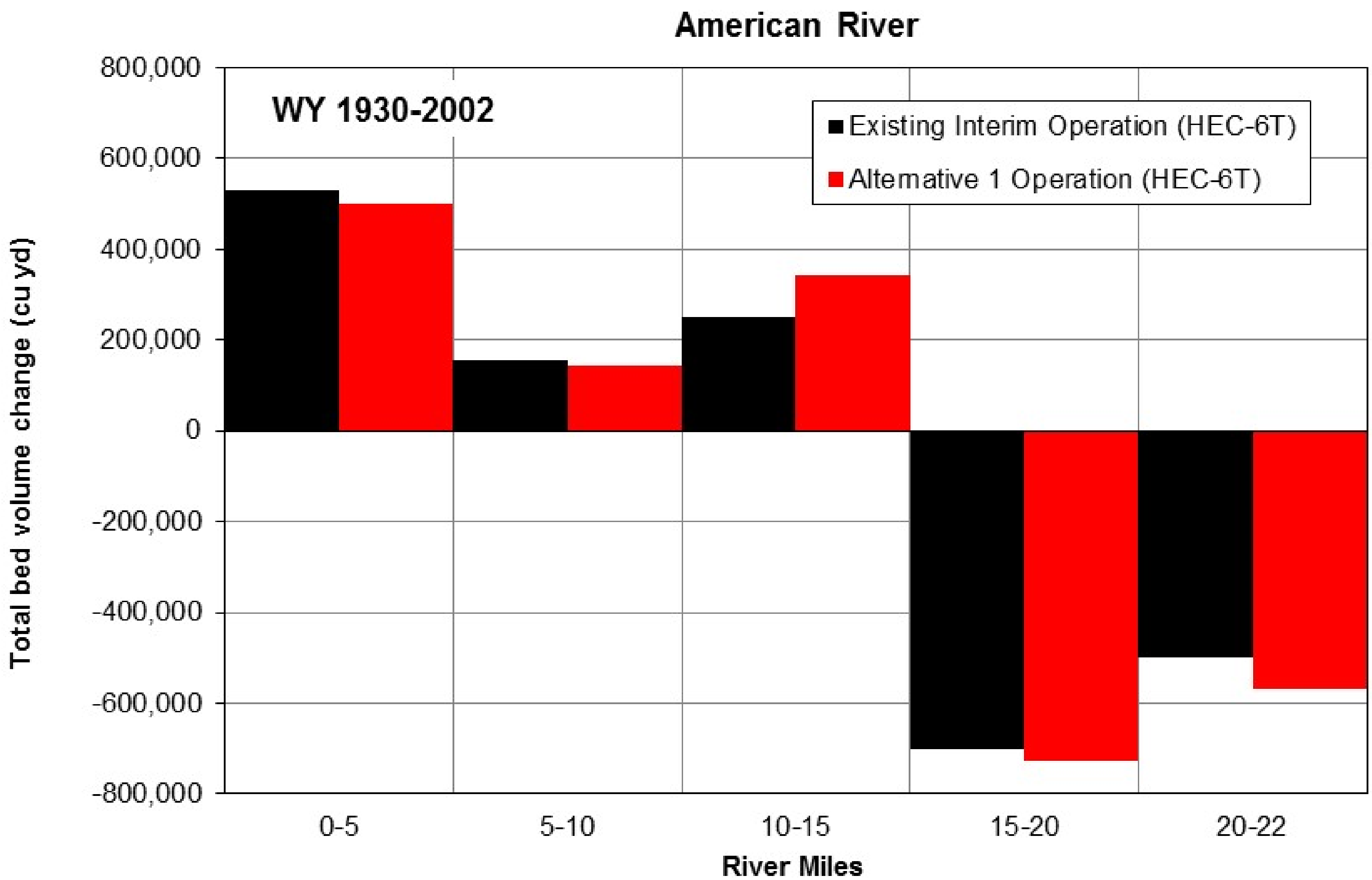
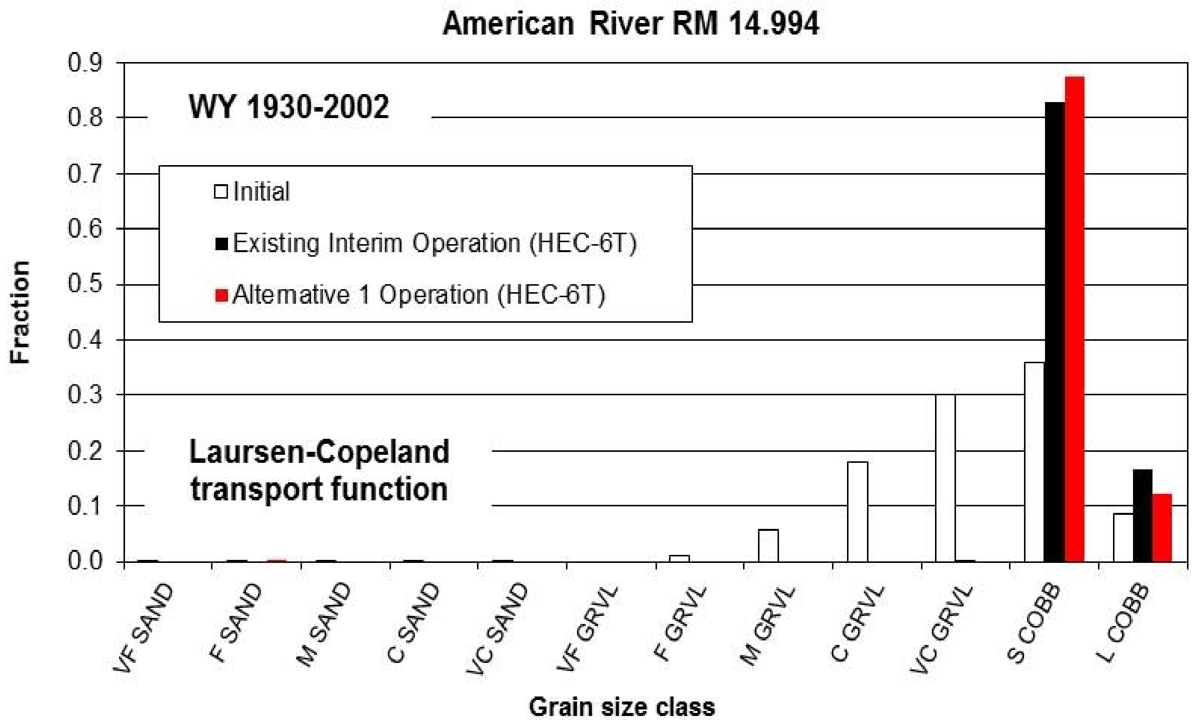


Figure 8-19: Computed Bed Volume Changes in American River for Existing Interim Operation and Alternative 1 Operation Conditions (Positive = aggradation; negative = degradation) (NHC, 2015)



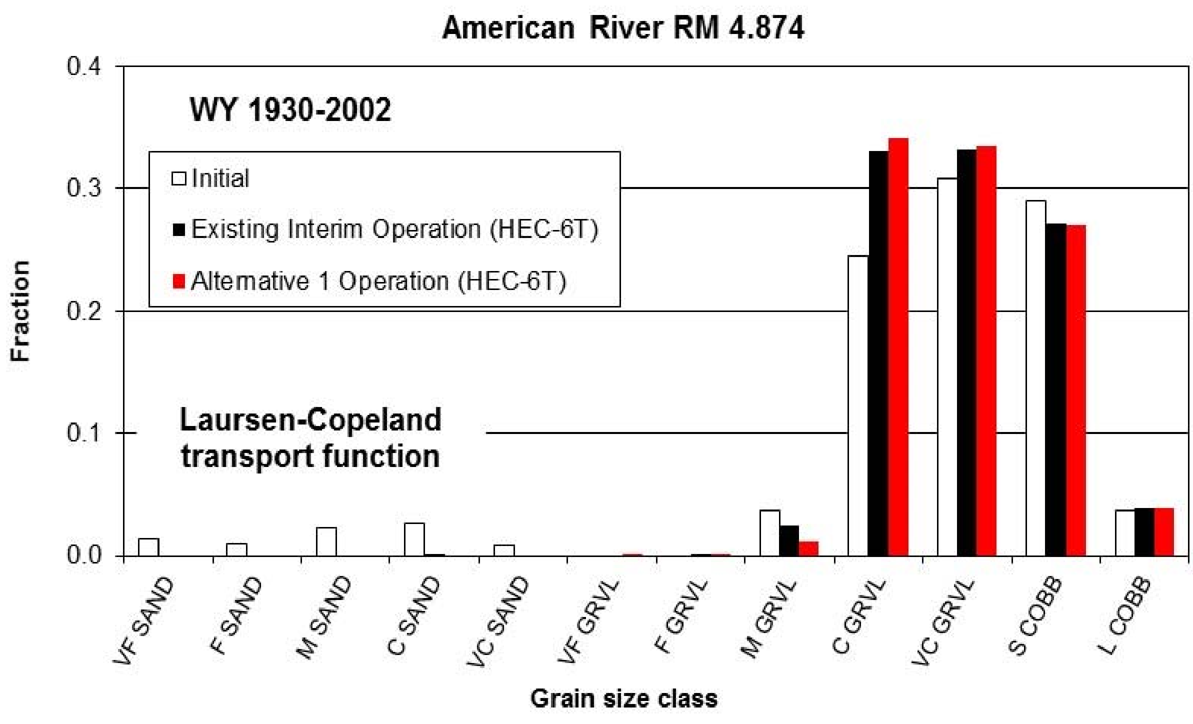


Figure 8-20: Computed Surface Bed Material Gradations in American River for Existing Interim Operation and Alternative 1 Operation (NHC, 2015)

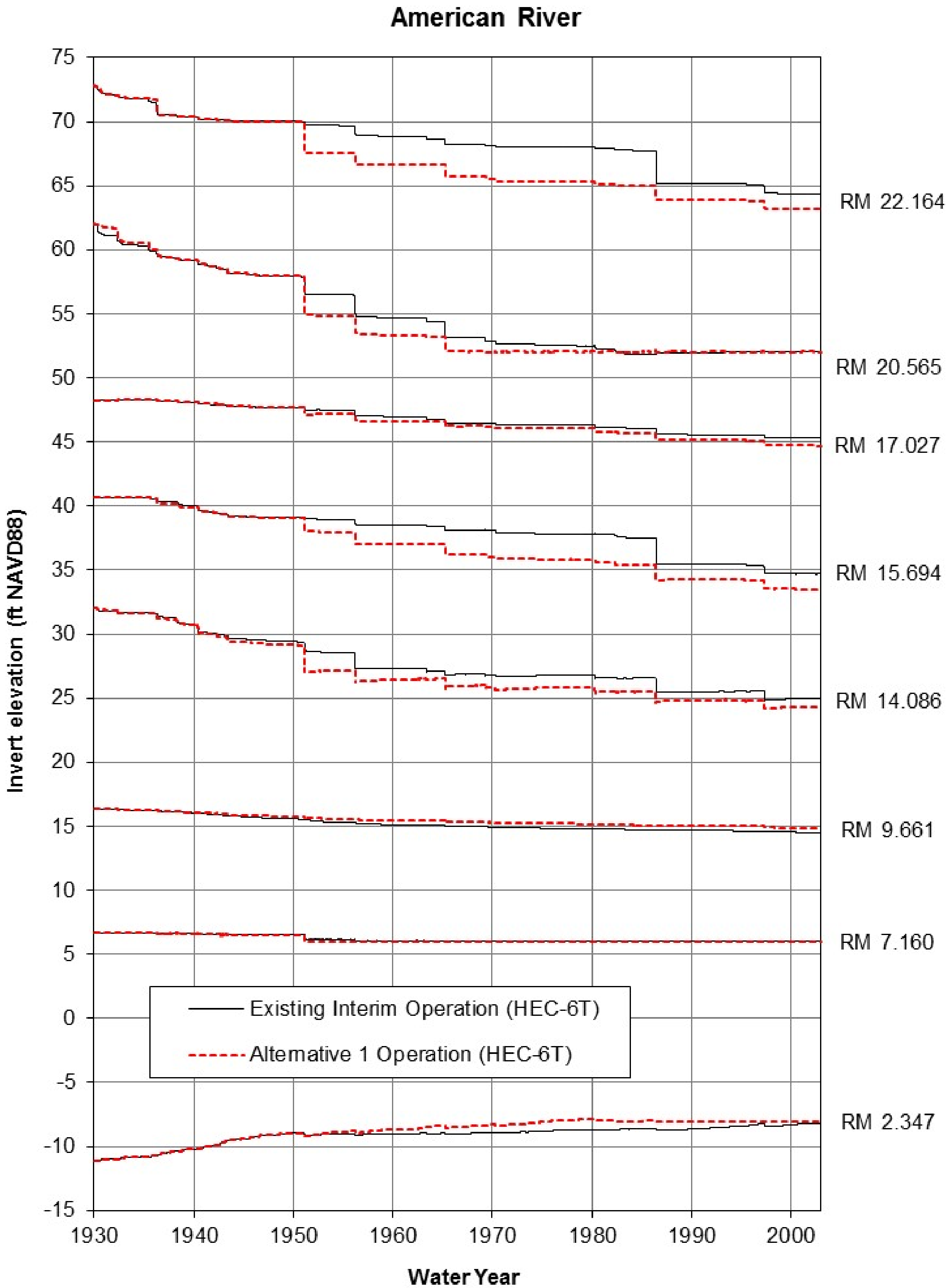


Figure 8-21: Comparison of Timeline Progression of Degradation and Aggradation Trends in American River Computed for Existing Interim Operation and Alternative 1 Operation (NHC, 2015)

8.3.4 Long-Term Erosion Assessment Conclusions

Channel widening and HEC-6T modeling was not performed for Alternative 2. The conclusions below for Alternative 2 are based on interpreting the results from available model runs and applying them to Alternative 2. This is done by considering the sensitivity of the analyses results and expected changes to erosion based on changes in flow frequency, magnitude, and duration.

8.3.4.1 Erosion of Riparian Habitat

1. Riparian habitat in sub-reach 8 is most at risk from systematic channel widening for Existing Interim, Alternative 1, and Alternative 2 operations.
2. Riparian habitat in subreaches 1 through 4 and 7 may also experience some loss of riparian habitat from systematic channel widening from Existing Interim, Alternative 1, and Alternative 2 operations.
3. All reaches could experience localized loss of riparian habitat due to site-specific conditions for Existing Interim, Alternative 1, and Alternative 2 operations.

8.3.4.2 Erosion of Spawning Gravel

1. Sub-reaches 1 through 4 are expected to see significant loss of spawning gravel for Existing Interim, Alternative 1, and Alternative 2 operations.
2. Sub-reaches 5 through 8 may also experience loss of spawning gravel for Existing Interim, Alternative 1, and Alternative 2 operations. However, the extent could be less than for sub-reaches 1 through 4.
3. Sub-reaches 9 through 10 likely will not experience substantial loss of gravel size material as these are generally aggradational reaches.
4. Alternative 2 may increase spawning gravel loss speed for sub-reaches 3, 7, and 8.

8.3.4.3 Erosion of Levees

1. The levees along the LAR are currently at a heightened risk of failure from erosion for Existing Interim operation and existing Corps operation.
2. Alternative 1 operation and Alternative 2 operation may increase erosion risk at some areas (without proposed ARCF GRR rock protection) and may not impact or could even reduce erosion risk at other areas relative to Existing Interim operation. However, the levees are only as strong as the weakest link, and therefore it is anticipated that Alternative 1 and Alternative 2 may increase the likelihood of long-term erosion contributing to a levee breach compared to Existing Interim operation without proposed ARCF GRR rock protection. However, because of the placement of ARCF GRR rock to protect against erosion, the low probability of failure associated with the increased flows (< 0.25 according to Figure 8-2 and Figure 8-3), and because the consequence portion of the risk equation (risk composed of likelihood of event and the consequences) will remain high, this is not expected to increase overall levee flood risk
3. The construction of ARCF GRR recommended erosion protection will improve levee erosion performance.

4. The erosion protection design as proposed in the ARCF GRR is unlikely to be affected by the changes in Folsom Dam operations proposed in the WCP because the design discharges are not changing and the design is based on a design discharge.
5. Based on the uncertainty of the erosion analysis and the magnitude of expected changes after the ARCF GRR erosion protection is constructed, the location and extent of erosion protection needed will not change.

8.3.4.4 Erosion of Bridges and Other Infrastructure

Alternative 2 operation is not expected to cause substantial increase in bridge scour or other in-channel infrastructure relative to Existing Interim operation.

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Appendix A – Acronyms

˚F	degrees Fahrenheit
2003 LRR	Folsom Dam Modification Project Final Limited Reevaluation Report
400 Fixed	400 Fixed Flood Control Diagram
400/600 Variable	Variable 400/600 Flood Control Diagram
400/670 Variable	Variable 400/670 Flood Control Diagram
500/800 Variable	Variable 500/800 Flood Control Diagram
1944 FCA	Flood Control Act of 1944
1991 Feasibility Report	American River Watershed Investigation Feasibility Report of 1991
1996 SIR	1996 American River Watershed, California, Supplemental Information Report

A

AAHU	annual average habitat unit
AAR	after action review
ac-ft	acre-foot, acre-feet
ACE	annual chance exceedence
A/E	architecture and engineering
AEP	annual exceedence probability
af	acre-foot, acre-feet
AFB	alternatives formulation briefing
AFRP	Anadromous Fish Restoration Program
AHPS	Advance Hydrologic Prediction System
ALT670	Interim Reoperation of Folsom Dam and Lake to a Maximum of 670,000 acre-feet of flood control space
ALT800	Interim Reoperation of Folsom Dam and Lake to a Maximum of 800,000 acre-feet of flood control space
APE	area of potential effects
AQAP	air quality attainment plan
AR	American River
ARCF	American River Common Features

ARBDA	American River Basin Development Act
AROG	American River Operations Group
ARWEC	American River Watershed Education Center
ARWI	American River Watershed Investigation
ARWP	American River Watershed Project
ASA(CW)	Assistant Secretary of the Army, Civil Works
ATR	U.S. Army Corps of Engineers Agency Technical Review
ATRT	U.S. Army Corps of Engineers Agency Technical Review Team

B

(b)(2) water	dedicated and managed water from implementation of Central Valley Improvement Act Section 3406(b)(2)
BA	biological assessment
Bay-Delta	San Francisco Bay – Sacramento-San Joaquin River Delta Estuary
BCA	benefit-cost analysis
BDCP	Bay Delta Conservation Plan
BMP	best management practice
BO	biological opinion
BOR	U.S. Bureau of Reclamation

C

CAAQS	California Ambient Air Quality Standards
CALFED	California Federal Bay-Delta Program
CAP	Continuing Authorities Program
CAR	coordination act report
CARB	California Air Resources Board
CCAA	California Clean Air Act
CDC	Climate Data Center
CDFG	California Department of Fish and Game; see also DFG
CEFMS	Corps of Engineer Financial Management System
CE/ICA	cost effectiveness/incremental cost analysis
Center	Center for Collaborative Policy

CEQA	California Environmental Quality Act
CERCLA	Comprehensive Environmental Response, Compensations, and Liability Act
CESA	California Endangered Species Act
CESPD	Corps of Engineers South Pacific Division
CESPD-ET-P	Corps of Engineers South Pacific Division, Planning Division; see also SPD
CESPK	Corps of Engineers Sacramento District; see also District
CESPK-ED-D	Engineering Division—Design Branch
CESPK-ED-E	Engineering Division—Environmental Engineering Branch
CESPK-ED-G	Engineering Division—Geotechnical Engineering Branch
CESPK-ED-H	Engineering Division—Hydraulics and Hydrology Branch
CESPK-ED-S	Engineering Division—Engineering Support Branch
CESPK-PD-R	Planning Division—Environmental Resources Branch
CESPK-PD-W	Planning Division—Water Resources Branch
CESPK-PM-C	Project Management Division—Civil Works Branch
CESPK-RD	Regulatory Division
CESPK-RE	Real Estate Division
CFR	Code of Federal Regulations
cfs	cubic feet per second
CIP	capital improvement program
CMR	Command Management Review
CNP	conditional non-exceedence probability (Note: consider CNE for conditional non-exceedence)
CNRFC	California Nevada River Forecast Center
CO	carbon monoxide
COA	coordinated operations agreement
Common Features	American River Common Features Project
Corps	U.S. Army Corps of Engineers
CSPA	California Sport-Fishing Protection Alliance
CVFPB	Central Valley Flood Protection Board
CVP	Central Valley Project

CVPIA	Central Valley Project Improvement Act
CVP-OCAP	Central Valley Project Operations Criteria and Plan
CWA	Clean Water Act

D

D-893	State Water Resources Control Board Decision 893
D-1485	State Water Resources Control Board Decision 1485
D-1594	State Water Resources Control Board Decision 1594
D-1641	State Water Resources Control Board Decision 1641
DDR	design documentation report
DEIS/EIR	draft environmental impact statement / environmental impact report
Delta	Sacramento-San Joaquin River Delta Estuary
DFG	California Department of Fish and Game; see also CDFG
District	Corps Sacramento District; see also CESPK
DPR	California Department of Parks and Recreation
DODAA	Department of Defense Appropriations Act
DQC	District Quality Control (Corps)
DR	dam raise
DrChecks	Design Review and Checking System
DST	District Support Team
DWR	California Department of Water Resources
DX	Department of Expertise

E

EA	environmental assessment
EBMUD	East Bay Municipal Utility District
EBRPD	East Bay Regional Parks District
EC	Engineer Circular
Econ	economics
ED	U.S. Army Corps of Engineers, Sacramento District, Engineering Division
EDF	Environmental Defense Fund

EDR	engineering documentation report
EDS&A	Engineering, Design, Supervision, and Administration
E/I Ratio	ratio of Delta exports to water inflow to the Delta, expressed by percentage
EID	El Dorado Irrigation District
EIR	environmental impact report
EIS	environmental impact statement
elevation xxx	elevation in feet above mean sea level
EM	Engineer Manual
EMS	Ensemble Member Specific
EO	Executive Order
EPA	U.S. Environmental Protection Agency
EPR	external peer review
EQ	environmental quality
ER	Engineer Regulation
ERDC	Engineer Research and Development Center (Corps Lab)
ESA	Endangered Species Act; environmental site assessment
ESRD	emergency spillway release diagram
ESU	evolutionarily significant unit
EMT	Ensemble Statistic
EWDAA	Energy and Water Development Appropriations Act

F

FACA	Federal Advisory Committee Act
FAQ	frequently asked questions
FCA	Flood Control Act
FCAA	Federal Clean Air Act
FCD	flood control diagram
FCSA	feasibility cost sharing agreement
FDA	flood damage assessment
FDR	flood damage reduction
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory Commission

FIO	Forecast-informed operations
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FLSRA	Folsom Lake State Recreation Area
FMS	flow management standard
Folsom Reop	Interim Reoperation of Folsom Dam and Lake
FONSI	finding of no significant impact
FOR	Friends of the River
FP	Floodplain
FPMS	Flood Plain Management Services Program
FRM	Flood Risk Management
FSG	Formulation Strategy Group
FWCAR	U.S. Fish and Wildlife Service Coordination Act Report
FWOP	future without-project
FWP	future with-project
FWS	U.S. Fish and Wildlife Service
FY	fiscal year

G

G	goal
GIS	geographic information system
GRR	general reevaluation report

H

HDR	HDR Engineering, Inc.
HEC	Hydrologic Engineering Center
HEMP	Hydrologic Engineering Management Plan
HEP	Habitat Evaluation Procedure
H&H	hydrology and hydraulics
HMR	Hydrometeorological Report
HMT	hydrometeorological test bed

HQUSACE	Headquarters, U.S. Army Corps of Engineers
HR	U.S. House of Representatives
HTRW	hazardous, toxic, and radiological waste
HU	habitat unit

I

IDP	Individual Development Plan (Training Plan)
IEPR	U.S. Army Corps of Engineers Independent External Peer Review
Interim Agreement	1995 Contract for operation of Folsom Dam and Lake
Interior	U.S. Department of the Interior
IPR	in-process review
IRC	issue resolution conference
IS	initial study
ISC	Interagency Security Committee
ITR	Independent Technical Review
IWR	Institute for Water Resources (Corps Lab)

J

JFP	Folsom Dam Joint Federal Project

K

KAF	Thousand acre-feet (see also TAF)
kV	kilovolts
kW	kilowatt

L

LAR	Lower American River
LEDPA	least environmentally damaging preferred alternative
LERRD	lands, easements, rights-of-way, relocations, and disposal areas
Long-term Study	American River Watershed, California Long-Term Study
LOP	level of protection
LOS	level of service
LPIII	log Pearson type III

LPP	locally preferred plan
LRR	limited reevaluation report
LWD	left wing dam

M

M&I	municipal and industrial
Manual Update	Folsom Dam Water Control Manual Update
MCACES	microcomputer-aided cost engineering system
mgd	million gallons per day
MIAD	Mormon Island Auxiliary Dam
MND	mitigated negative declaration
MOA	memorandum of agreement
MOU	memorandum of understanding
MSC	Major Subordinate Command
msl	mean sea level
mva	mega-volt amps or million volt amps
MW	megawatt

N

NAAQS	National Ambient Air Quality Standards
National Register	National Register of Historic Places
NAVD88	North American Vertical Datum of 1988
NCI	National Critical Infrastructure
NCPA	Northern California Power Agency
NED	National Economic Development
NEP	non-exceedence probability
NEPA	National Environmental Policy Act
NER	National Ecosystem Restoration
NFIP	National Flood Insurance Program
NGO	non-governmental organization

NGVD29	National Geodetic Vertical Datum of 1929
NHPA	National Historic Preservation Act
NMFS	National Marine Fisheries Service; see also NOAA Fisheries Service
NOA	naturally occurring asbestos
NOAA Fisheries Service	National Oceanographic and Atmospheric Administration National Marine Fisheries Service
NOI	Notice of Intent
NOP	Notice of Preparation
NOx	nitrogen oxides
NRCS	Natural Resources Conservation Service
NRDC	Natural Resources Defense Council
NTP	Notice to Proceed
NWS	National Weather Service

O

O&M	Operations and Maintenance
OC	Oversight Committee
OCAP	Central Valley Project Operations Criteria and Plan
OEO	Outside Eligible Organization
OMG	Oversight Management Group
OMRR&R	operation and maintenance, repair, replacement and rehabilitation
Ops Group	CALFED Operations Coordination Group
OS	opportunity statement
OSE	other social effects

P

PACR	Post Authorization Change Report
PAO	Public Affairs Office
Partner	For the Folsom Dam Water Control Manual Update, the Corps' partners are the Bureau of Reclamation, SAFCA, and DWR
PADD	Post Authorization Decision Document
PASS	Project Alternative Solutions Study
PCA	Project Cooperation Agreement

PCWA	Placer County Water Agency
PCX	Planning Centers Of Expertise
PD	U.S. Army Corps of Engineers Sacramento District, Planning Division
PDT	Project Delivery Team
PED	preconstruction, engineering, and design
PG&E	Pacific Gas and Electric Company
PGM	project guidance memorandum
PIA	Prison Industry Authority
PL	Public Law
PM	project manager
PM10	particulate matter of 10 microns or less in diameter
PMF	probable maximum flood
PMG	Project Management Group
PMP	Project Management Plan
PMS	probable maximum storm
POC	point of contact
POR	period of record
PPA	project partnership agreement
ppm	parts per million
PPMD	U.S. Army Corps of Engineers, Sacramento District, Programs and Project Management Divisions
PRB	Project Review Board
Principles and Guidelines (P&G)	principles and guidelines; Federal Water Resources Council’s Economic and Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies
PRP	Peer Review Plan
PROMIS	project management information system
Proposed Action	2004 Interim Reoperation Plan
PS	problem statement
psu	practical salinity unit

Q

QA	quality assurance

QC	quality control
QCP	quality control plan
QMP	quality management plan
QPF	quantitative precipitation forecasts

R

RAP	Refined Authorized Project
RCMP	River Corridor Management Plan
RD	Reclamation District
RDF	reservoir design flood
Reclamation	U.S. Department of the Interior, Bureau of Reclamation
Reclamation Board	State of California Reclamation Board
RED	regional economic development
RIT	U.S. Army Corps of Engineers Regional Integration Teams
RM	resource manager
RMO	Review Management Organization
ROD	record of decision
ROE	right of entry
ROS	reservoir operation set
RP	review plan
RPA	Reasonable and Prudent Action
rpm	revolutions per minute
RTS	Regional Technical Specialist
R&U	risk and uncertainty
RWQCB	Regional Water Quality Control Board
RWR	right wing dam

S

SACCR	schedule and cost change request
SACOG	Sacramento Area Council of Governments
SAFCA	Sacramento Area Flood Control Agency
SARA	Save the American River Association

SJRA	San Joaquin River Agreement
SJWD	San Juan Water District
SHPO	State of California Historic Preservation Office; State of California Historic Preservation Officer
SIP	State Implementation Plan
SIR	supplemental information report
SMAQMD	Sacramento Metropolitan Air Quality Management District
SMUD	Sacramento Municipal Utility District
SOW	scope of work (for contractors)
SOS	scope of service
SPA	U.S. Army Corps of Engineers Albuquerque District
SPD	U.S. Army Corps of Engineers South Pacific District; see also CESPD
SPF	standard project flood
SPK	U.S. Army Corps of Engineers Sacramento District
SPL	U.S. Army Corps of Engineers Los Angeles District
SPN	U.S. Army Corps of Engineers San Francisco District
Sponsors	Local entity entering into feasibility cost sharing agreement with the Corps to share the cost of the feasibility phase of a project or study. For the Folsom Dam Water Control Manual Update, sponsors include DWR (CVPFB) and SAFCA.
SRA	State Recreation Area
STG	submerged tainter gate
Stakeholder	Entity or individual with a stake or interest in the outcome of a project or study
Study	Flood Management Operations Study for Folsom Dam
SWP	State Water Project
SWRCB	State Water Resources Control Board

T

TAC	Traffic Advisory Committee
TAF	thousand acre-feet (see also KAF)
TNM	Traffic Noise Model
TOC	top of conservation
TRSS	Technical Review Strategy Session (Corps)

U

UAIC	United Auburn Indian Community of the Auburn Rancheria
USACE	U.S. Army Corps of Engineers
USBR	U.S. Bureau of Reclamation
USEPA	U.S. Environmental Protection Agency, see also EPA
USFWS	U.S. Fish and Wildlife Service
USGS	U.S. Geological Survey

V

VE	Value Engineering
VEST	value engineering study

W

WAPA	Western Area Power Administration; also known as Western
Water Forum	Sacramento Water Forum
WBS	work breakdown structure
WCD	Water Control Diagram
WCM	Water Control Manual
WCP	Water Control Plan
Western	Western Area Power Administration; also known as WAPA
WFA	Water Forum Agreement
WRCB	Water Resources Control Board
WRDA	Water Resources Development Act
WRDA 07	Water Resources Development Act of 2007
WRDA 08	Water Resources Development Act of 2008
WRDA 96	Water Resources Development Act of 1996
WRDA 99	Water Resources Development Act of 1999
WSE	water surface elevation
WY	Water Year

X

X2	distance upstream, in kilometers, from the Golden Gate Bridge to the tidally averaged near-bed, 2-psu isohaline

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Appendix B – Quality Assurance/Quality Control (QA/QC)

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Appendix C – Basin Wetness

This appendix describes development of the CNRFC basin wetness parameter. This parameter is computed by CNRFC, and is used in Alternative 1 to compute the corresponding storage credit at Folsom Lake. Equations for computing storage credit are found in the Section 4.1 of the main report.

C 1 Seasonal Variation of Top of Conservation

C 1.1 Basin Wetness Indices

C 1.1.1 Sensitivity Analysis of Initial Conditions on Large Floods in the American River Basin Using SAC-SMA and SNOW-17

As an initial step for the SAC-SMA/SNOW-17 "basin wetness" parameterization study for Folsom's new water control plan, some sensitivity analysis was done to help get a better understanding of the impact that extremely dry conditions have on inflow volumes. The 1997 event was primarily used as the flood event for this analysis. Water year 1977 was selected as the dry condition extreme.

The CNRFC watershed model consists of 15 sub-basins. Nine of the fifteen sub-basins are broken into two separate areas – upper and lower. The upper areas consist of elevations above 5,000 ft, and the lower areas consist of everything below 5,000 feet. The upper and lower sub-areas each have their own unique set of soil and snow parameters. The runoff from both areas is added and processed through a single unit hydrograph. The topology for the American watershed is represented in
Figure C- 1.

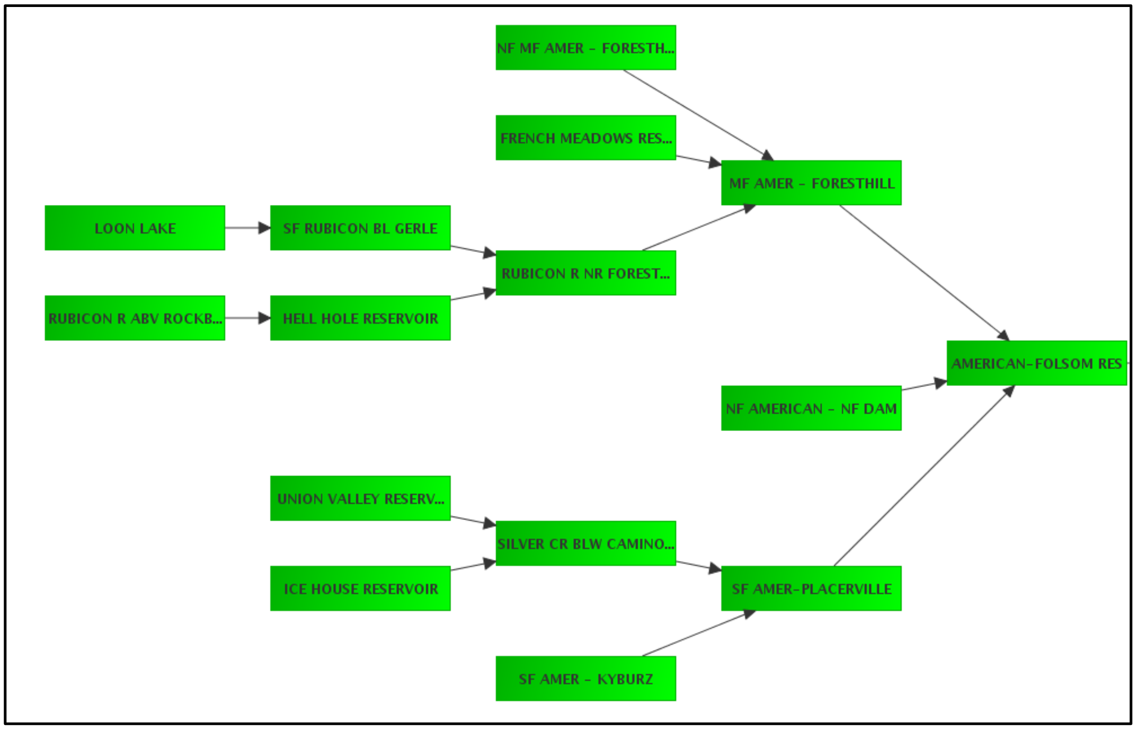

Figure C- 1: CNRFC Topology for the American River Watershed

First a historical simulation was performed for the entire American River watershed, which includes water years 1949-2010. The basin states (both soil and snow) were then saved off for 29 December 1976. This date was selected because it was within a reasonable time window when historic floods have occurred, and was also one of the drier days during that winter. Figure C- **2** and **Figure C- 3** show the soil states for the North Fork upper watershed for water years 1977 and 1978. Each subplot indicates the fullness of a particular soil zone. The Y axis is percent full and the X axis is time. The red vertical line indicates 29 December 1976. At this point in time, the upper soil zones (UZTWF and UZFWF) are quite depleted, with UZTWF less than 25 percent and UZFWF completely empty. Baseflows (LZFSF and LZFPF) and lower zone tension water (LZTWF) also have quite a significant deficit. And the dryness in the North Fork lower zone is even more pronounced. These trends are consistent throughout all of the American sub-basins.

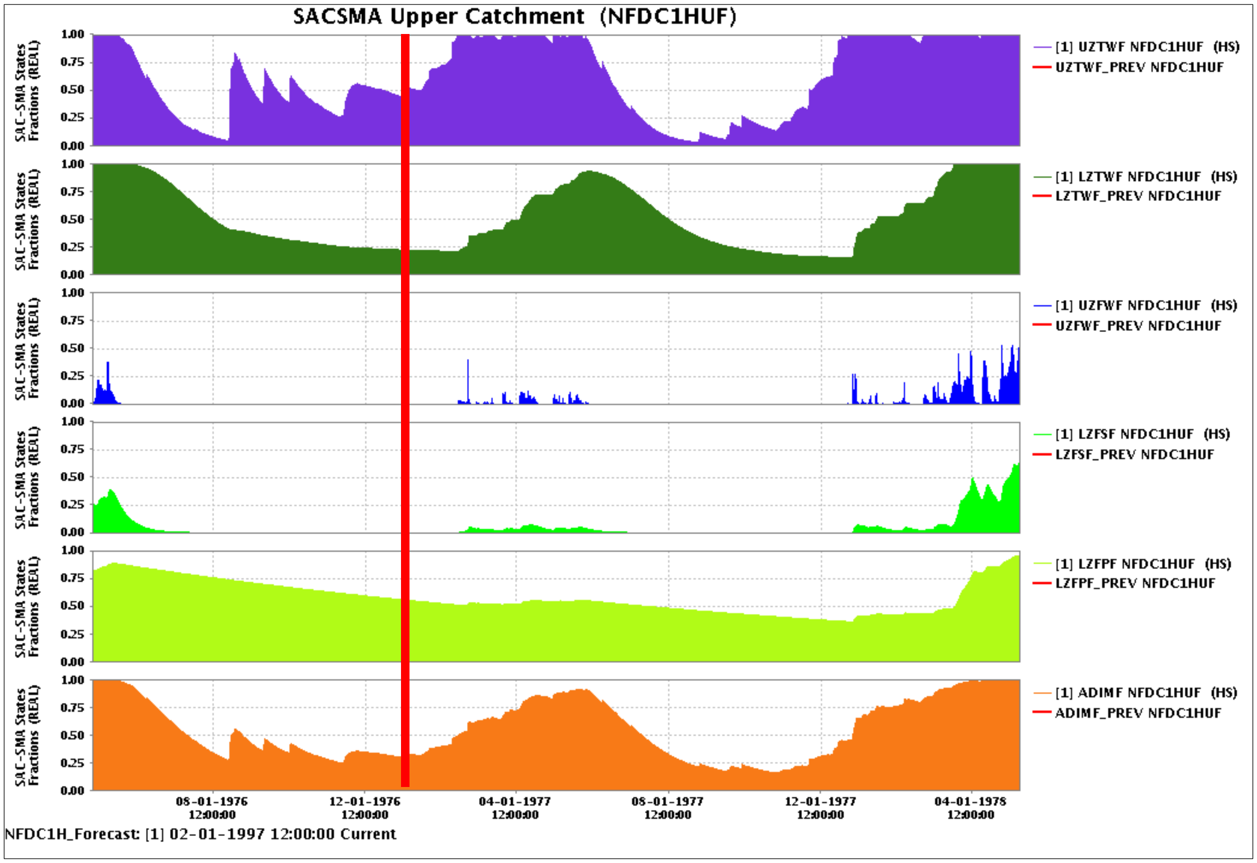

Figure C- 2: Water Year 1977 North Fork Upper Elevation Soil Conditions

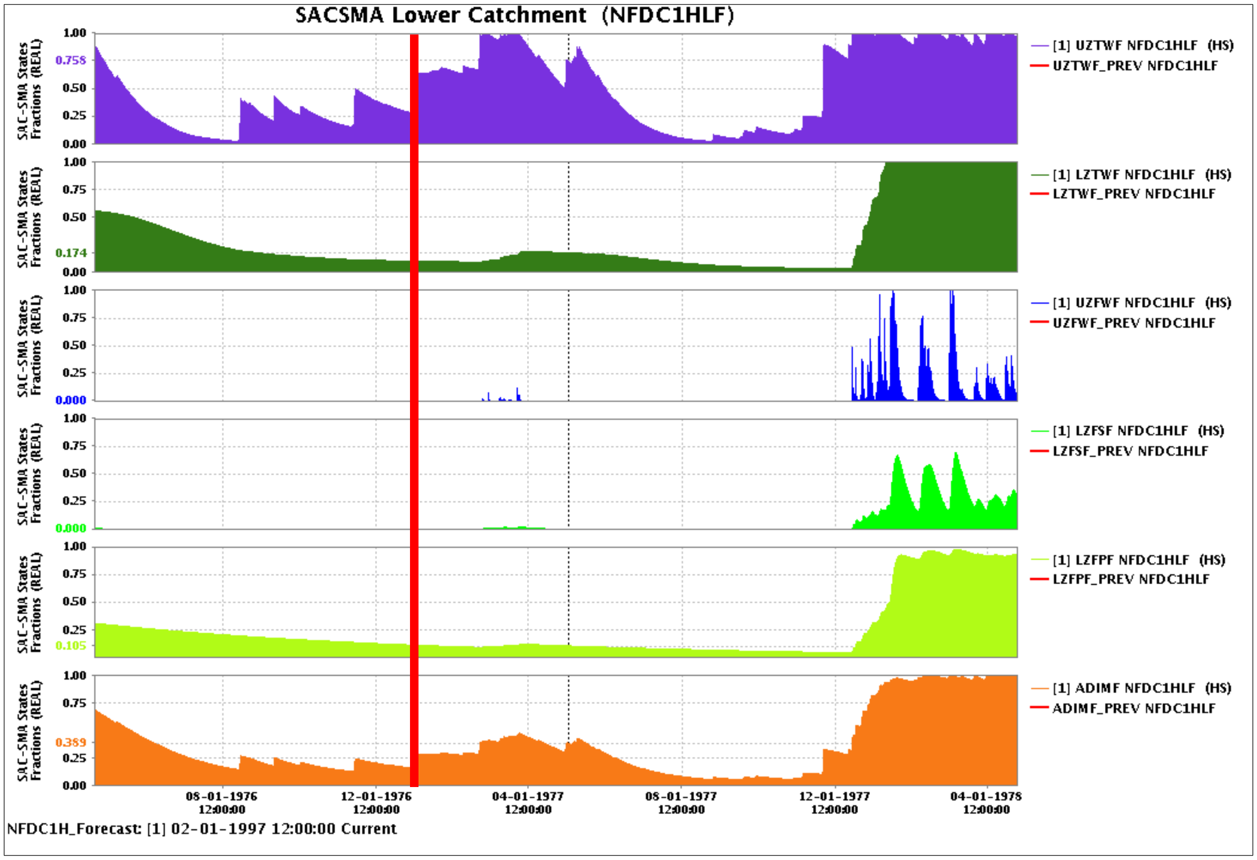

Figure C- 3: Water Year 1977 North Fork Lower Elevation Soil Conditions

These states were then used as initial conditions for the 1997 flood event. The heavy precipitation spanned approximately eight days and started on 26 December (see **Figure C- 4**). The December 1976 dry states were initialized prior to the start of the heavy precipitation on 25 December.

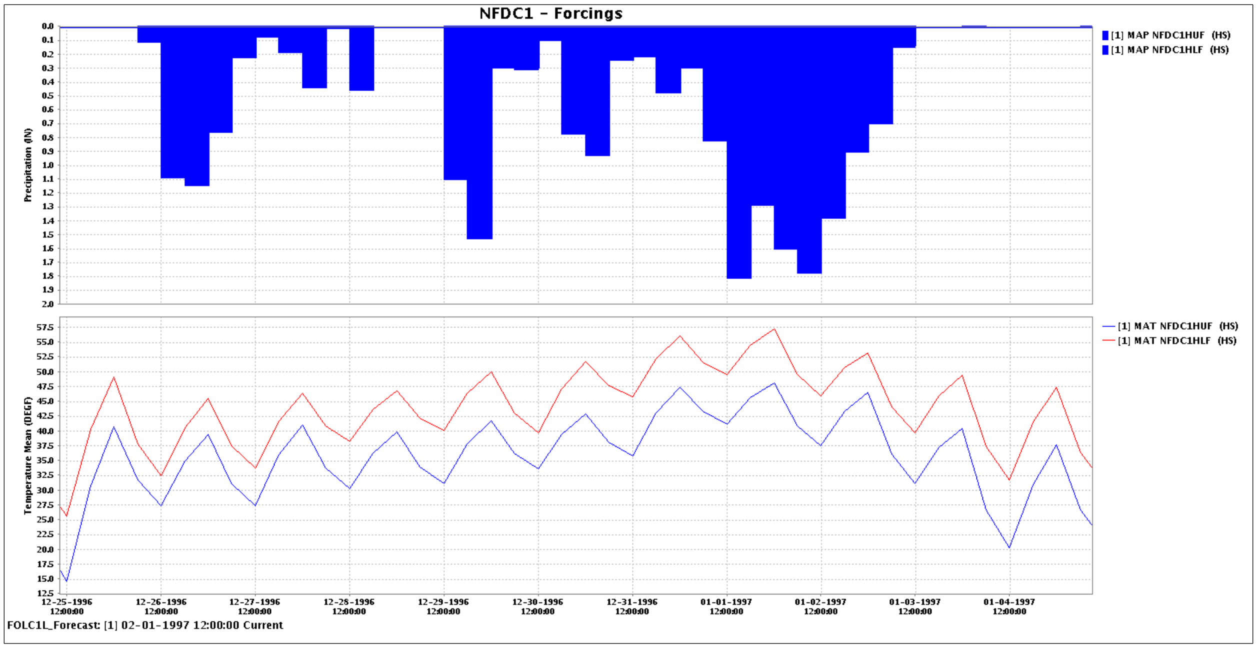

Figure C- 4: North Fork American 1997 Event Precipitation and Temperatures

The Folsom inflow volumes were reduced quite significantly when initialized with the December 1976 dry conditions. **Table C- 1** summarizes these differences.

Table C- 1: 1997 Initial Conditions Comparison

Condition	Max 1 day flow (cfs)	Max 3 day flow (cfs)
1997 Hist Simulation	226,000	162,000
1997 Dry Simulation	104,000	80,000
% Reduction	54%	51%

The main reason for this reduction is the soil conditions. The lower and upper soil zone deficits both contributed to the reduced runoff. **Figure C- 5** shows the difference in soil states for the upper elevations in the North Fork sub-basin. The red lines indicate the soil states for the dry simulation, and the shaded blocks are the states in the historic 1997 event simulation. Significant runoff is produced when the upper zone freewater “tank” is full (UZFWF). As you can see, this tank did not fill until 1 January during the dry simulation, and was completely empty at the beginning of the event. In the historic simulation, this tank began filling at the very beginning of the event and became completely full much earlier in the event. The lower zone baseflow, known as the lower zones (LZFSF and LZFPF), and the lower zone tension water affect the filling rate of the UZFWF. The fullness of these tanks has a pretty big effect on percolation rates from the upper to lower zones. In the dry simulation these zones were much more depleted, thus increasing the percolation rates from UZFWF to the lower zones. The deficit in the upper zone tension water tank (UZTWF) is another factor in the runoff reduction. This zone is the portion of the soil where moisture can only be removed due to the evapotranspiration processes. No runoff can occur until this tank is filled with the exception of the impervious areas in the watershed. This tank does not fill up until about 30 December during the dry simulations, but it is completely full at the beginning of the historic simulation. This is another reason why the

UZFWF did not fill as rapidly in the dry simulation. These same effects were observed in the lower elevation basins as well, and even more pronounced (see **Figure C- 6**).

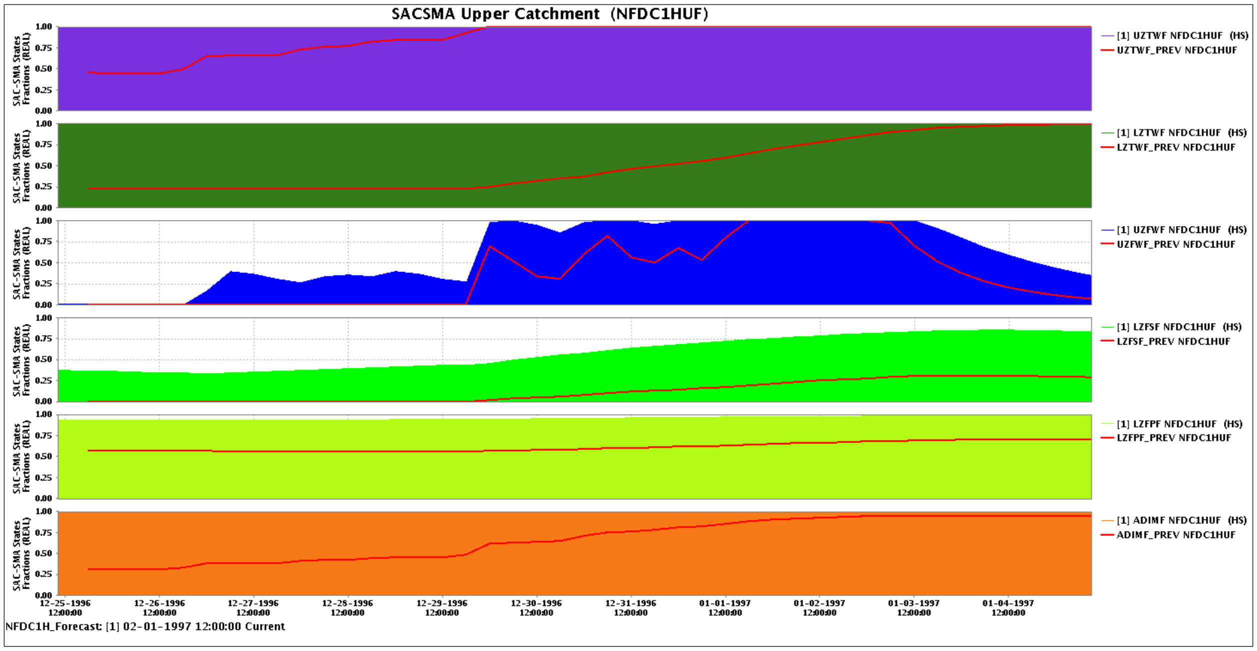

Figure C- 5: 1997 North Fork Upper Elevation Watershed Soil Condition Comparisons
Note: Red lines indicate 1977 initial condition simulation of the 1997 event

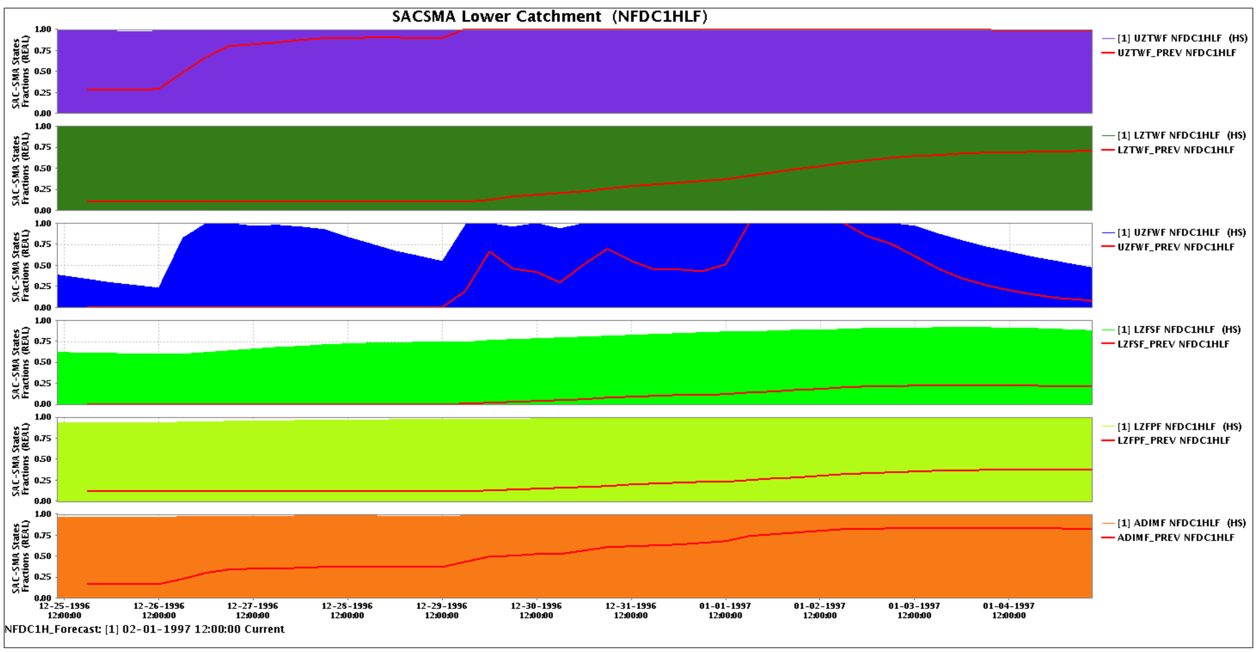

Figure C- 6: 1997 North Fork Lower Elevation Watershed Soil Condition Comparisons
Note: Red lines indicate 1977 initial condition simulation of the 1997 event

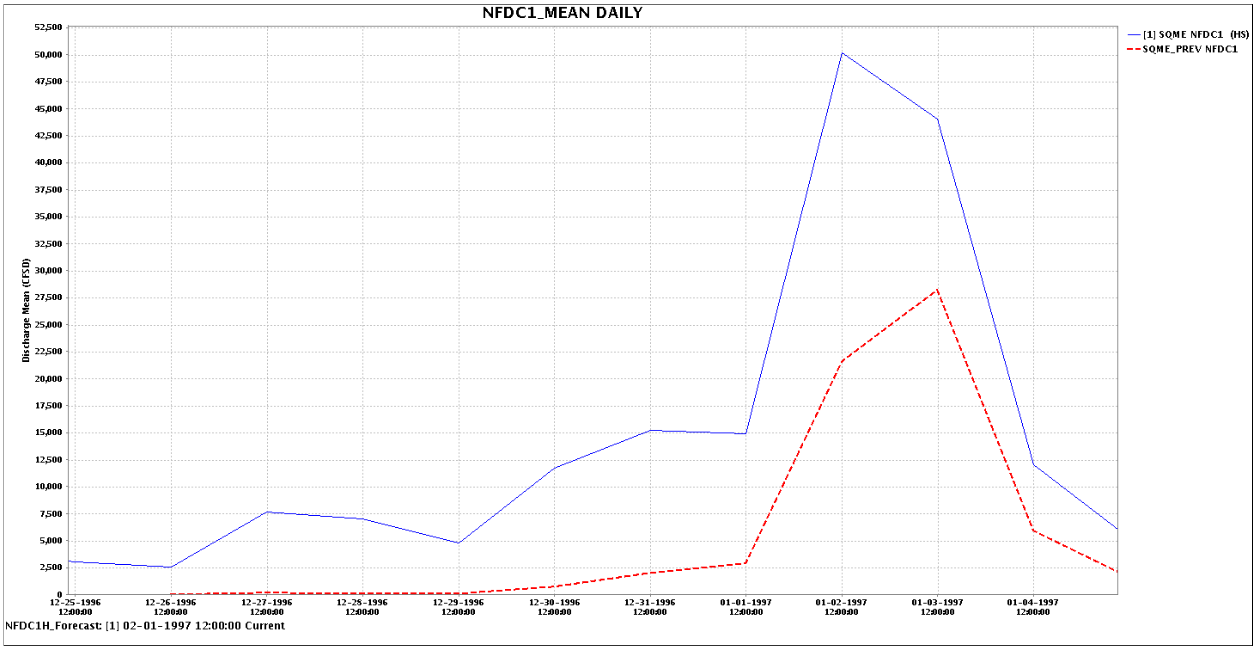

Figure C- 7: North Fork American 1997 Simulated Mean Daily Flow
Note: Red lines indicate 1977 initial condition simulation of the 1997 event

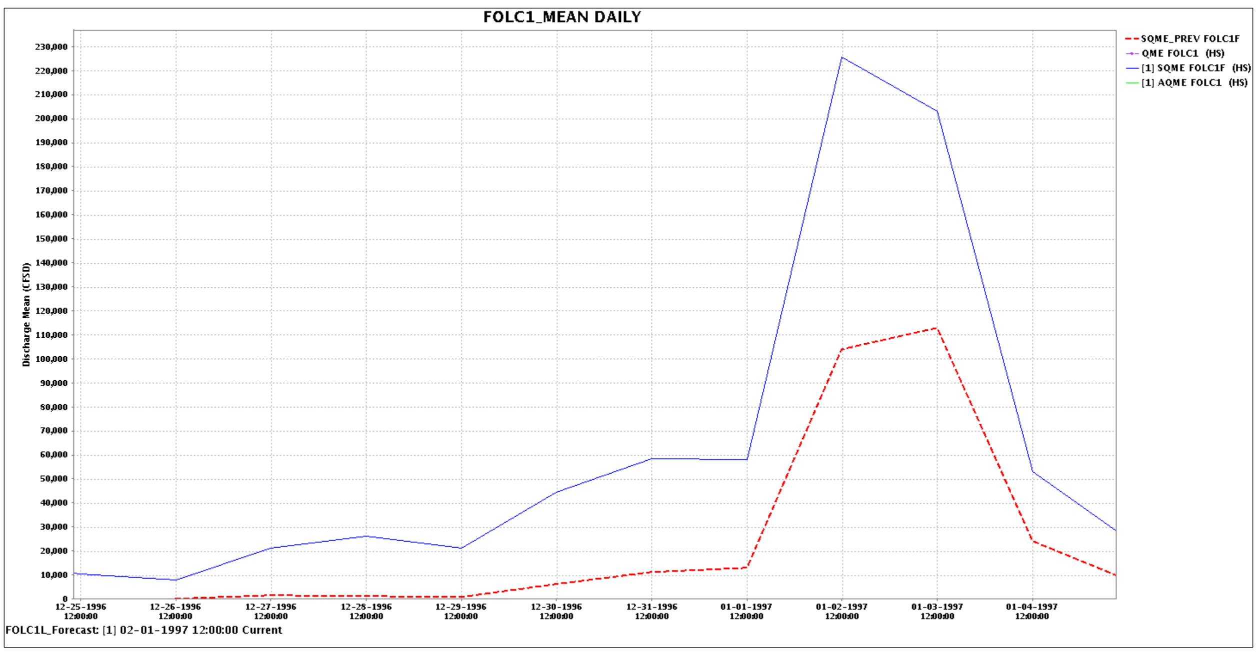

Figure C- 8: 1997 Simulated Folsom Mean Daily Inflow
Note: Red lines indicate 1977 initial condition simulation of the 1997 event

The 1986 event had drier initial conditions than the 1997 event, so the 1977 states were also used as initial conditions for the 1986 event to see if the runoff reductions were less dramatic (Figures C-7 and C-8). The main precipitation spanned approximately 8 days in the 1986 event as well, and started on 12 February 1986 (see **Figure C- 9**). The 1977 states were initialized on 11 February 1986.

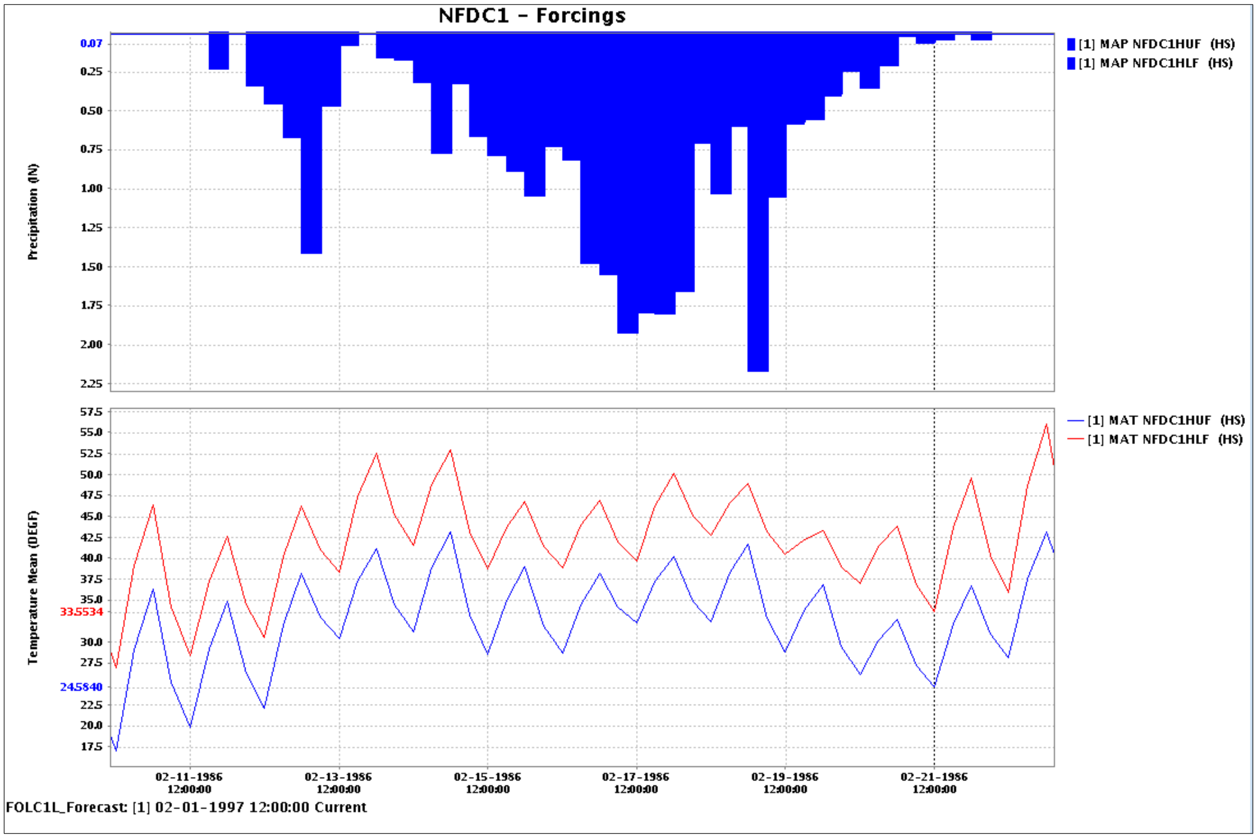

Figure C- 9: Precipitation and Temperature Forcings for the North Fork 1986 Event

Similar runoff volume reductions were observed for the 1986 event even though soil states were somewhat drier in 1986 when compared to 1997. Maximum 3-day Folsom inflows were reduced by about 50 percent. **Figure C-10** and **Figure C-11** show the soil state comparisons for the North Fork elevation zones. The initial base flow contents are pretty similar for the dry and historic simulations for the upper elevation areas. However, the upper zone tension water tank is almost completely full for the historic simulation. The lower elevation areas show similar differences for the tension water states, but there is also a larger difference in baseflow contents. Even though 1986 initial conditions are drier than 1997, they are still much wetter than what was observed in December 1976. The first part of February was quite dry in 1986; however, the earlier months of the water year were fairly wet when compared to water year 1977. Approximately 20 inches of basin-averaged precipitation fell in the American River watershed from 1 Oct 1985 to 1 February 1986. Only three inches fell during the period of 1 Oct 1976 through 28 Dec 1976. This difference is the main reason why soil states were wetter in February 1986 when compared to December 1977.

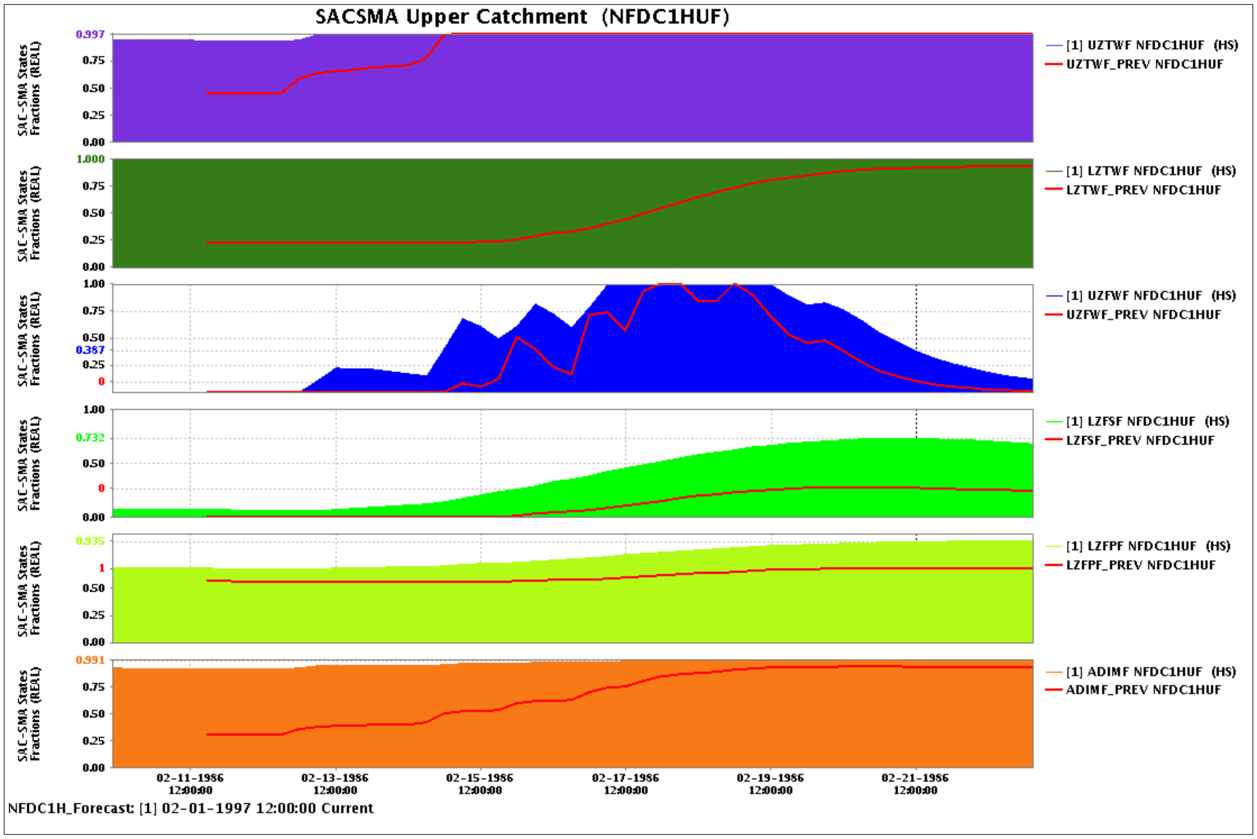

Figure C-10: 1986 North Fork American Upper Elevation Soil States
Note: Red lines indicate 1977 initial condition simulation of the 1997 event

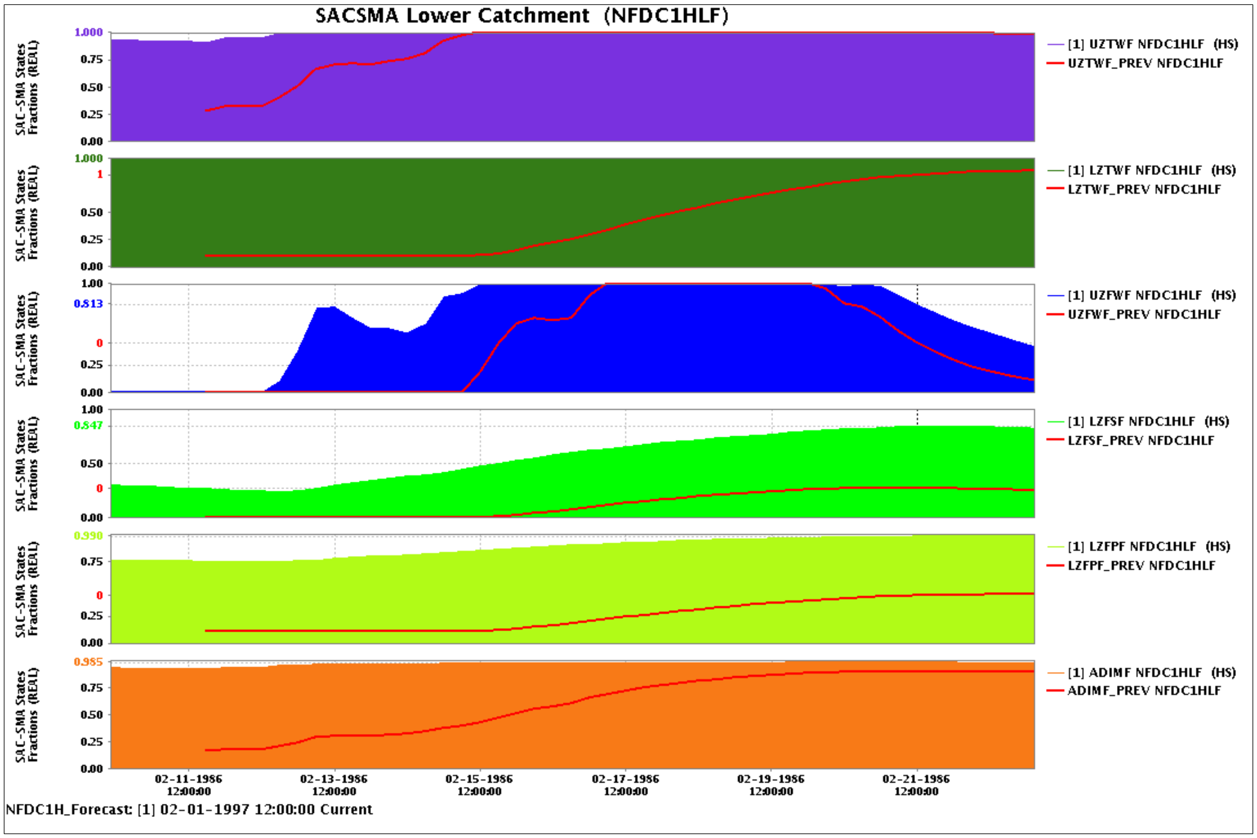

Figure C-11: 1986 North Fork American Lower Elevation Soil States
Note: Red lines indicate 1977 initial condition simulation of the 1997 event

A third scenario examining 200-year flows was evaluated to see what volume reductions could be expected for even larger floods. The 1997 event precipitation was incrementally increased over the entire Folsom watershed until a 200-year 3-day flow (235 kcfs) was produced. A factor of 1.4 applied to the eight days of precipitation starting on 26 December for all sub-basins resulted in a Folsom 3-day mean flow of 232 kcfs. The 1977 dry conditions were applied to this scenario resulting in a 3-day flow reduction of about 35 percent (149 kcfs). The volume reduction was not as substantial for the 200-year event because initial states become less of a factor as precipitation is increased, and soil tanks fill at a much faster rate.

A few additional scenarios were evaluated to test the sensitivity of snow conditions. Five inches of snow water equivalent (SWE) was added to every sub-basin for the 200-year scenario using historical 1997 states. The 3-day flow only increased by about 7 percent. Adding ten (10) inches of SWE increased the 3-day flow by only about 12 percent. In general, only about 2 to 3 inches of SWE were melted during the entire event. Based on this sensitivity analysis, it can be concluded that rainfall is the primary driver in wintertime floods, and runoff variability is much more dependent on soil states, not snow conditions.

C 1.1.2 Winter Basin Wetness Analysis

Historical Pattern Selection and Scaling

The basin wetness sensitivity study revealed that antecedent conditions have a significant effect on runoff for large wintertime storms. The next step was to generate a historical record of wetness indices based on the methodologies discussed in the basin wetness sensitivity chapter. Seasonal 1/200 year events were developed for the winter (December-February) using CNRFC SAC-SMA and SNOW-17 models for the American River watershed. The 1997 and 1986 scaled-up precipitation patterns were both used as 1/200 storm scenarios. The duration of the precipitation was 5 days for both historic events. Each of these historic event scenarios contained the scaled-up precipitation for each basin and the associated temperatures from the historic event. The 1986 precipitation scaling started on 14 February 1986 at 12:00 p.m. and ended on 19 February at 12:00 p.m. The 1997 precipitation scaling started on 29 December 1996 at 12:00 p.m. and ended on 3 January 1997 at 12:00 p.m. The 1986 precipitation was iteratively scaled up until inflow volumes matched the 3-day 1/200 flow, and the 1997 precipitation was scaled iteratively to match the 2-day 1/200 flow. These flow duration targets were selected based on the SPK critical duration analysis. The following **Table C- 2** shows the scaled precipitation amounts and the scaling factor used for both historic event patterns.

Table C- 2: Scaled Precipitation and Scaling Factors

Event	Factor	Max Precipitation (inches)					Duration (days)
		24-hour	48-hour	72-hour	96-hour	120-hour	
1986	1.335	8.3	13.2	18.1	22.2	24.6	5.00
1997	1.340	7.6	10.9	12.3	14.4	16.3	5.00

Period of Record Wetness Simulation

The entire daily period of record basin states (soil and snow conditions) that were generated in the Sensitivity Analysis chapter were used to assess runoff potential when these two 1/200 storms were applied to the historical basin conditions. The historical basin states spanned the period of water years 1949-2010. The CNRFC hydrology models were run in a batch mode where the model was initiated with the 1 October 1948 basin states, and the 1/200 storm was then applied to the basin to produce a Folsom inflow hydrograph. Then a new model run was initiated using the historic antecedent conditions from 2 October 1948 with the 1/200 storm applied to the basin. This process was repeated for every day through 30 September 2010. The end result was a set of conditional 1/200 inflow hydrographs for the entire historic period. Since this analysis was only focused on the winter period, the results for December-February were evaluated. The runoff potential for each day was summarized by comparing the conditional critical duration runoff (3-day for 1986 and 2-day for 1997) to the 1/200 runoff volumes from the SPK frequency curves. Dividing the daily conditional runoff volume by the frequency curve volumes gave a dimensionless saturation ratio (or “wetness index”) for every day in the wintertime from 1948-2010. **Figure C-12** shows exceedence wetness index levels for the wintertime for the period of record 1986 1/200 event. Based on the historical results, the driest period is at the beginning of December and the wettest period is around mid-February.

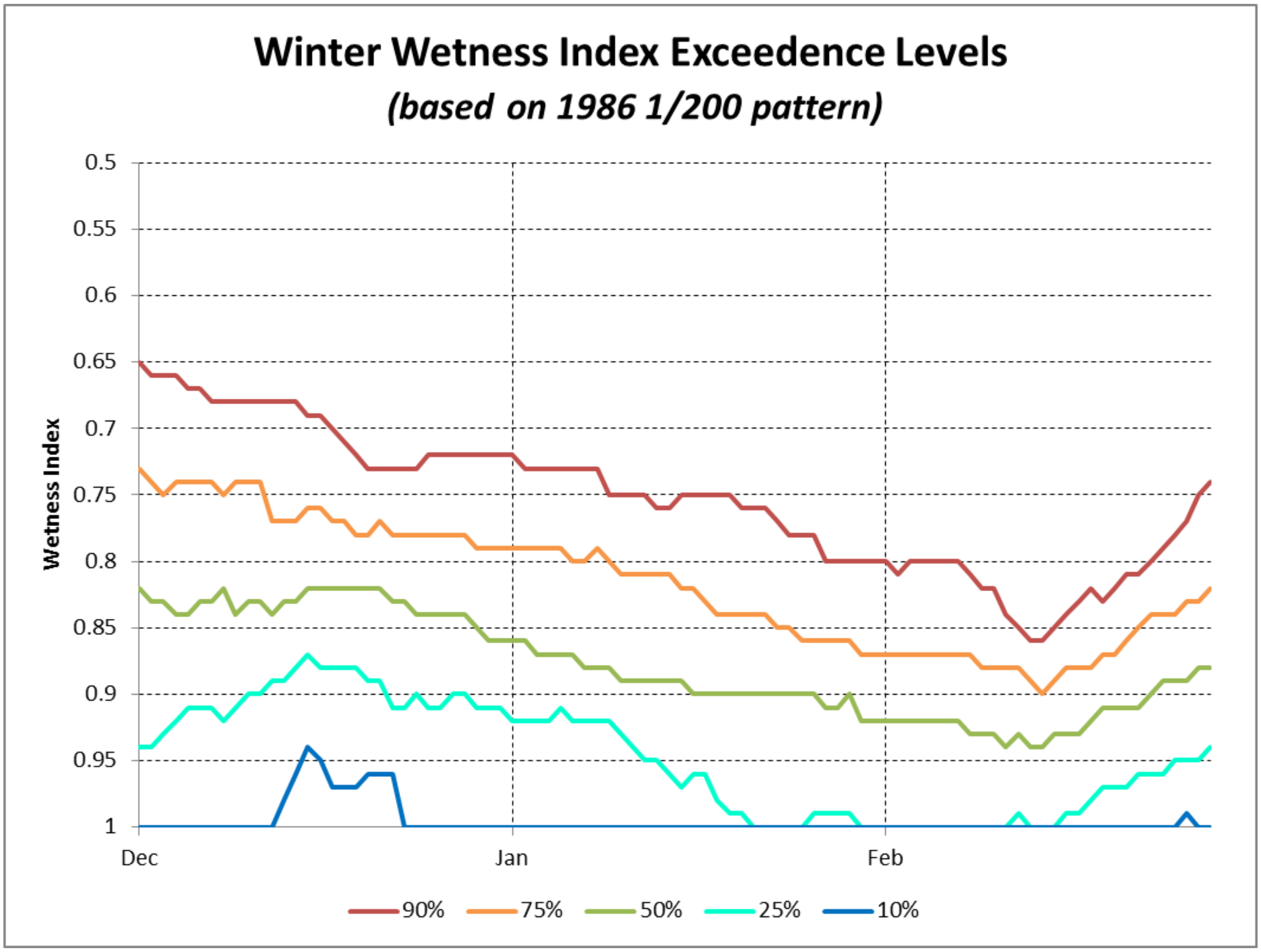

Figure C-12: Winter Wetness Index Exceedence

From this information, conditional 1/200 inflow hydrographs based on a wide variety of wetness indices could be evaluated by ResSim to determine how much flood space was required to handle these conditional floods. Both the results for 1986 and 1997 went through initial evaluation, but only one historic pattern was selected as the representative 1/200 pattern that was carried forward in the wintertime wetness index analysis. The 1986 pattern was chosen over the 1997 because it was a more naturally balanced hydrograph. Discussions related to wintertime basin wetness in subsequent chapters will focus primarily on the 1986 1/200 results.

C 1.1.3 Spring and Fall Basin Wetness Analysis

Historical Pattern Selection

Seasonal 1/200 year events were developed for October, November, March, April, and May using CNRFC SAC-SMA and SNOW-17 models for the American River watershed. Each 1/200 seasonal event contains precipitation and temperature forcings that are based on a historical pattern. The historical pattern selected for each month was determined by ranking the largest 3-day average Folsom inflows for each of the 5 months. The largest flood that occurred for a given month was usually the pattern that the 1/200 event was based off of, but not always. Some months the largest inflow occurred outside of the CNRFC precipitation and temperature record (water years 1949-2010). In those cases, a different event that ranked within the top five for that month was selected as the historical pattern. **Table C- 3** details the top five 3-day average inflows for each month. The cell highlighted in yellow is the event that was used as the pattern when scaling to the 1/200 event.

Table C- 3: Maximum 3-Day Folsom Inflow Rankings

	Maximum 3-Day Folsom Inflow Rankings									
	1		2		3		4		5	
Month	Year	Flow (cfs)	Year	Flow (cfs)	Year	Flow (cfs)	Year	Flow (cfs)	Year	Flow (cfs)
October	1962	38,400	2010	12,800	1982	11,200	1975	6,000	1945	3,400
November	1950	107,500	1983	31,800	1981	31,400	1973	25,800	1909	20,800
March	1928	98,200	1907	87,800	1995	55,300	1943	52,000	1940	51,000
April	1940	53,500	1982	52,900	2006	44,900	1958	33,600	1935	29,100
May	1995	42,200	1996	41,800	1915	36,000	2005	31,600	1938	26,400

Antecedent Conditions
When sizing the winter 1/200 events, the initial conditions from 1986 and 1997 were used because they were determined to be quite wet for that particular time of year. The same thinking was used when selecting initial conditions for the seasonal events. The definition of wet conditions is quite different depending on the time of year, so season-appropriate wet antecedent conditions were selected for each of the five seasonal events. Simulated soil and snow output from the period of record (1949-2010) historical simulation was used along with historical precipitation information to identify seasonally appropriate wet conditions.
The wettest conditions (outside of during an event) for October occurred after the 10-14 October 1962 event. This was an extremely rare event that resulted in extremely wet soil conditions for the rest of the month of October. These conditions were not selected as the initial conditions for the October 1/200 scaling because it is unlikely that a 1/200 event would follow a 1/400 event. Wet conditions in October are influenced by both precipitation occurring in October, and also the amount of precipitation from the previous water year. Water year 1982 was the wettest on record for the American River Basin. Baseflows in October 1982 were extremely high due to the extremely wet previous water year. Therefore, 1 October 1982 was selected as the antecedent conditions for the 1/200 year October event.

The November wet conditions are similar to October because they are a function of both precipitation occurring in the fall, and the amount of precipitation that occurred in the previous water year. Water year 1983 was the second wettest year on record in the American River Basin, and October 1983 was also fairly wet. Therefore, antecedent conditions from 15 November 1983 were selected as initial conditions for the 1/200 year November event.

Soil conditions in the spring do not vary as dramatically when compared to the fall. During the winter months, soils tend to become quite saturated even for below-normal winters. However, there are variations from year to year and month to month. For the 1/200 March event, antecedent conditions from 10 March 1983 were selected as initial conditions. The actual antecedent conditions prior to the April 1982 event and the May 1995 event were some of the wettest conditions in the period of record, so those conditions were used as the initial conditions for the 1/200 April and May events.

Hypothetical 1/200 Event Scaling
Once the antecedent conditions were determined for each 1/200 seasonal event, the size and duration of the 1/200 events were determined. The duration of the precipitation varied somewhat for the selected historical events, but they all ranged from 4 to 5 days. The precipitation for the selected time periods was iteratively scaled until the CNRFC model simulated a 3-day inflow that matched the 1/200 flow determined from the Corps seasonal flow frequency curves. The historical temperatures remained unchanged for the 1/200 hypothetical events. **Table C- 4** shows the beginning and ending times for each historical event selected, scaling precipitation factors, and the 1/200 3-day flow targets.

Table C- 4: Historical Events Details

Month	Event Start (GMT)	Event End (GMT)	Event Duration (days)	Scaling Factor	1/200 3-day flow target (cfs)
October	10/10/1962 @ 12:00	10/14/1962 @ 12:00	4	0.44	22,300
November	11/16/1950 @ 06:00	11/21/1950 @ 12:00	5.25	0.50	67,800
March	3/8/1995 @ 12:00	3/12/1995 @ 18:00	4.25	1.53	124,300
April	4/9/1982 @ 12:00	4/13/1982 @ 12:00	4	1.61	83,600
May	4/27/1995 @ 12:00	5/2/1995 @ 12:00	5	1.20	50,600

Period of Record 1/200 Event Simulations
Once the 1/200 events were determined, a period of record simulation (1949-2010) was generated for each of the 1/200 seasonal events where the hypothetical events were applied to the historical basin conditions for each day in the period of record. Max 24-, 48-, and 72-hour flows were exported for each day along with the hourly Folsom hydrographs. Ratios of conditional flows to “wet condition” flows were calculated.

Saturation Ratio Calculation and Weighting Scheme
Since the seasonal events are smaller than the wintertime 1/200 event, the saturation ratios (conditional flow divided by “wet condition” flow) were generally less than the results when the wintertime 1/200 event was applied to the fall and spring months. However, since different size and shaped events were used for each month, there are discontinuities between adjacent months. For example, a saturation ratio might be 0.9 on 31 October, and then drops to 0.5 on 1 November. This discontinuity was much more obvious in the fall than the spring, most likely because the soil conditions change so much during the fall months. To smooth these discontinuities out, a weighting scheme was developed. 1 October received 100 percent of the October saturation ratio, but each day forward, the November event ratio was included in the weighting by an additional 1/31 fraction each day. So by 1 November, the saturation ratio was 100 percent from the November event results. This same scheme was developed for the transition period from November to December. The winter period from 1 December through 14 February did not require any weighting.

For the spring months, the weighting scheme was a little different since the spring frequency curves are based on 3-month moving windows. Starting on 15 February, the saturation ratios were weighted using a portion of the winter and March saturation ratios. On 15 February, only 1/31 of the March ratio was used in the weighted saturation ratio, but by 15 March 100-percent of the saturation ratio came from March. This same weighting scheme was used for April and May where the midpoint of each month resulted in a saturation ratio that was completely based on results from the current month's saturation ratio.

The duration used in the ratio weighting was the 3-day for all the fall and spring events since that was the target duration used when scaling to the 1/200 event. For the winter events, the 3-day was used for the 1986 scenario and 2-day for the 1997 scenario, since those were the respective duration targets used in the 1/200 scaling process.

C 1.1.4 Combining All-Season Wetness Indices

Weighting Scheme

Once the historical wetness indices were derived for the fall, winter, and spring, they needed to be combined to create a continuous time series that could ultimately be used to derive a period of record (1948-2010) basin-wetness flood space adjustment for historical ResSim routing analysis. Simply merging the indices from the different seasons would not be a good idea because of large shifts that could occur when moving from one month and from season to season. Therefore, a weighting scheme was used to blend the wetness indices together. The monthly wetness indices were weighted with the adjacent month's indices to help smooth the transition. The center of each monthly frequency curve window was determined to be 100 percent of that month's index. For example, the wetness index on 15 March was 100 percent the March wetness index. However, 1 March was the combination of 1/15th of the March index and 14/15^{ths} of the wintertime index. The spring frequency curves were moving 3-month windows, but the fall frequency curves were discrete one month windows. Therefore, the weighting scheme was a little different for October and November. The mid-month value was still 100 percent that month's wetness index, but the first half of the month was also 100-percent of that month's index. The second half of the month was weighted towards the next month. For example, 15 October was 100 percent the October basin wetness index. However, 16 October was 1/16th the November index and 15/16^{ths} of the October index.

Fall Wetness Index Evaluation

The fall wetness indices varied the most out of all the seasons. The variations were quite significant for many water years even with the smoothing technique applied. The soil states are changing significantly during this period. Soils can range from very dry at the beginning of October to very saturated at the end of November. This results in big swings in runoff potential. The size of the 1/200 event is also changing the most during the fall which also causes large swings in wetness indices when transitioning from October to November to December. Historical Folsom storage information revealed that it was highly unlikely that storage levels could ever be high enough to take advantage of any flexible storage due to wetness conditions in the fall. Therefore, the wetness index for the fall season was not pursued any further, and a constant drawdown similar to the existing flood control diagram would be applied in the fall.

Spring Wetness Evaluation
The spring wetness index monthly transitions were smoother than the fall, but there were still some fairly large shifts that occurred that were undesirable. There are many factors involved with these shifts, but the duration of the different seasonal events was one of the more significant factors. The duration of the seasonal events (see **Table C- 4**) in the Seasonal Basin Wetness chapter) ranged from 4 to 5.25 days. These duration variations had fairly significant effects on the runoff sensitivity. Therefore, it was deemed necessary to have a consistent event throughout the entire season to help with smooth transitions from month to month. The 1986 pattern was selected to be the pattern for both the winter and spring. The spring 1/200 precipitation amounts were rescaled using the procedures discussed in the previous chapters. The scaling factors and precipitation totals using the 1986 pattern are presented in **Table C- 5**.

Table C- 5: Maximum Precipitation

		Max Precipitation (inches)					Duration
Event	Factor	24-hour	48-hour	72-hour	96-hour	120-hour	(days)
Winter-86	1.335	8.3	13.2	18.1	22.2	24.6	5.00
Mar-86	0.75	4.6	7.4	10.2	12.5	13.8	5.00
Apr-86	0.535	3.3	5.3	7.3	8.9	9.8	5.00
May-86	0.36	2.2	3.6	4.9	6.0	6.6	5.00

The period of record simulations where the seasonal 1/200 events were applied to the daily historical basin conditions were regenerated, and new spring wetness index values were determined. The monthly wetness indices were then combined using the weighting scheme described previously. The winter and spring wetness index exceedence levels are presented below:

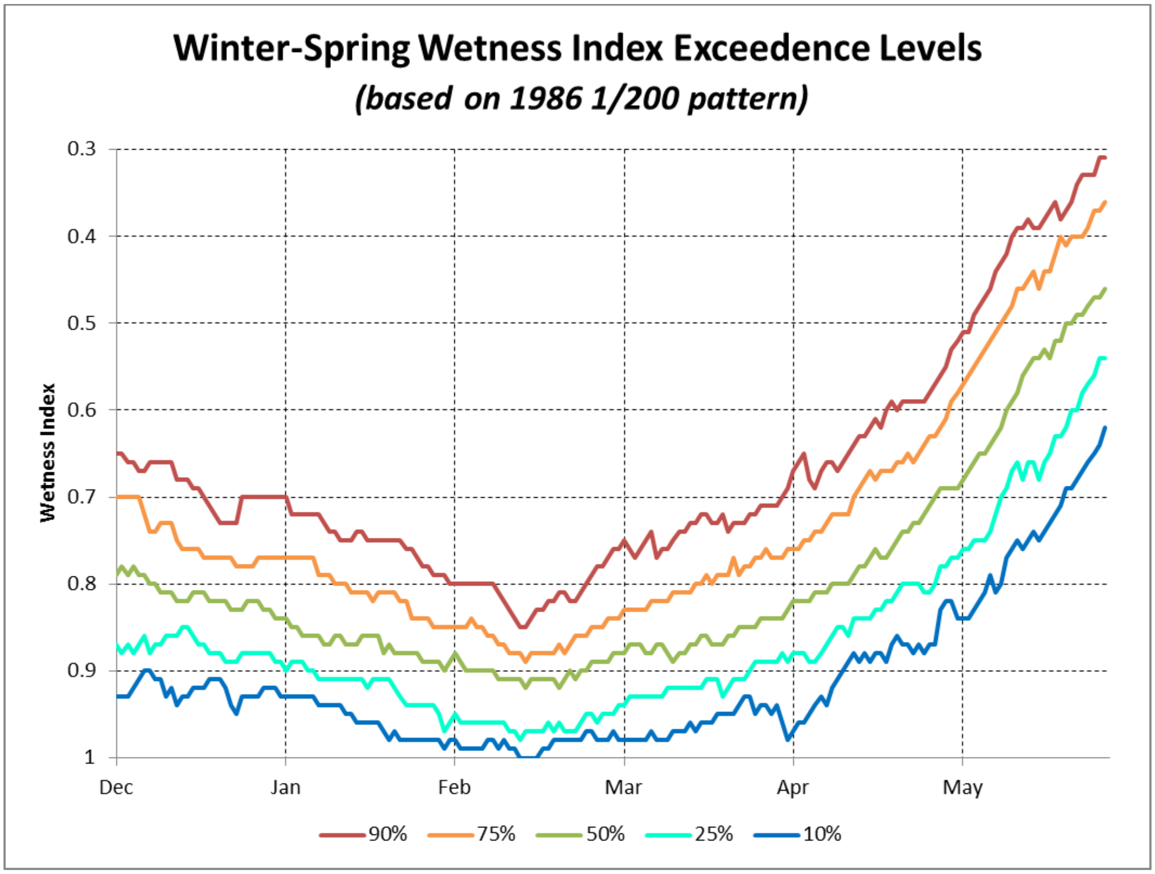

Figure C-13: 1986 North Fork American Lower Elevation Soil States

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Appendix D – CNRFC Forecasting

This appendix describes the ensemble forecast (and hindcast) product. This product is used in Alternative 2 to compute top of conservation (TOC) and forecast-based releases. The process by which the ensemble product is used in Alternative 2 is described in Section 5.0 of the main report.

D 1 Overview of CNRFC Forecasting in American River watershed

As a part of its normal duty, the CNRFC forecasts runoff from precipitation throughout the American River watershed. Forecasts are made year-round at 15 locations in the watershed, culminating in a forecast of inflow to Folsom Lake. The area modeled for forecasting is shown in **Figure D- 1**. The yellow lines in the figure represent boundaries of sub-basins included in the model. Folsom Lake is labeled, and smaller headwater reservoirs can be seen in the upper watershed. Much of the eastern portion of the watershed is above 5,000 feet elevation, with both rainfall and snowfall/snowmelt driving runoff.

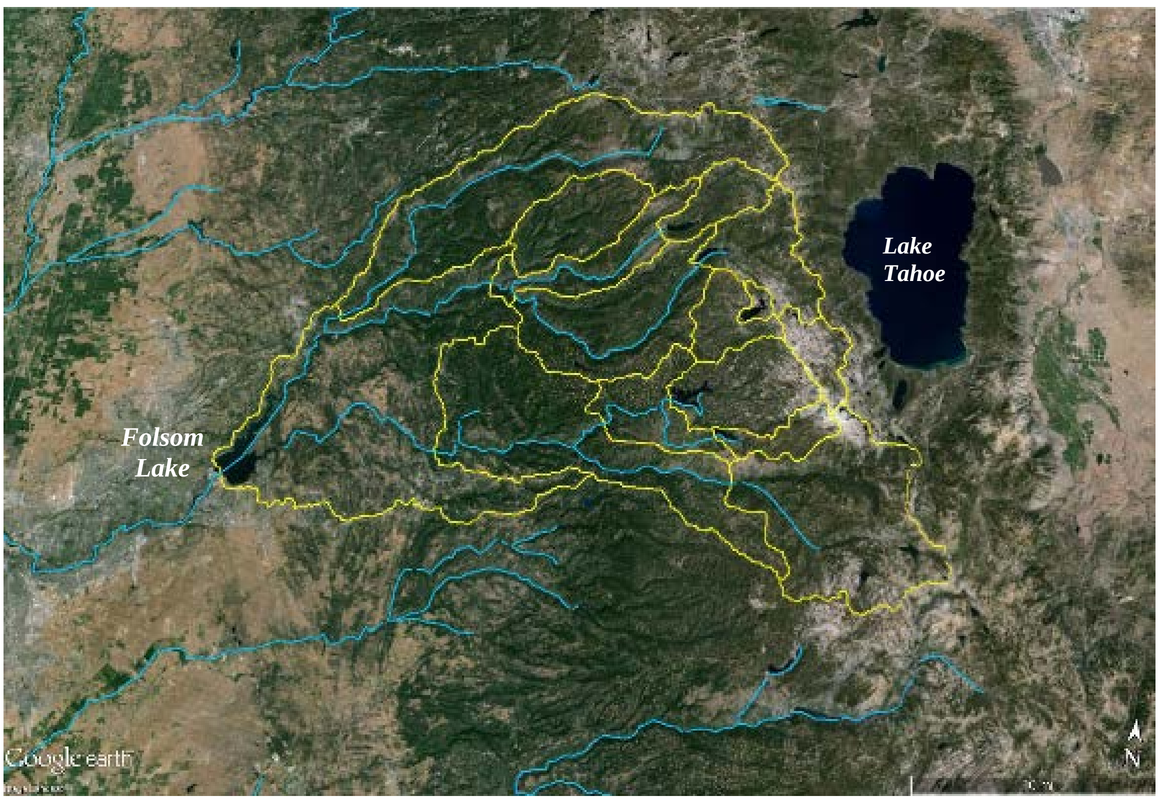


Figure D- 1: American River Watershed CNRFC Model Configuration

CNRFC forecasts account for both recently observed and forecasted future rainfall and snowmelt contributions to runoff. Rain, snow, and temperature forecasts are developed by NWS climate and weather prediction centers and refined by meteorologists at the CNRFC to capture local conditions. The precipitation and temperature inputs drive a numerical modeling system developed within the framework of the NWS Community Hydrologic Prediction System (CHPS). Execution of the model is managed and reviewed by hydrologists and forecasters at the

CNRFC. The precipitation-runoff-routing model simulates snow accumulation and snowmelt, soil moisture accumulation and excess runoff, overland flow, channel routing, and operation of headwater reservoirs and diversions within the American River watershed.

CNRFC develops and provides to its customers a best-estimate deterministic forecast using best estimates of current states of the watershed and best estimates of future precipitation and temperature. These deterministic forecasts, prepared and issued at least twice a day and more frequently during flood events, include runoff hydrographs at key locations for five days following the time at which the forecast is issued. Hydrograph ordinates are computed at a 6-hour time step and reported at an hourly time step, thus providing the temporal resolution necessary for reservoir flood operation decision making.

CNRFC forecasters, recognizing the uncertainty associated with forecasts of future precipitation and temperature that are critical drivers of the runoff forecast, also provide an ensemble forecast that provides information about other possible outcomes. Commonly, this ensemble is displayed with a so-called "spaghetti plot," as illustrated in **Figure D- 2**. The ensemble forecast are initiated with the same estimates of current watershed states as the deterministic forecasts, and use both the CNRFC deterministic precipitation and temperature forecasts, and information from an atmospheric model called the Global Ensemble Forecast System (GEFS). This forecast information is run through the Meteorological Ensemble Forecast Preprocessor (MEFP) resulting in multiple series of future precipitation and temperature inputs. These meteorological ensembles are processed through the American River hydrology models, producing a set of hydrographs that are considered equally likely future conditions in the watershed. For days 1-15 into the future, each ensemble uses information from the short-term forecasts described previously. Days 16-365 use historical climate records. For Folsom, 60 years of climate data are available, so 60 alternative hydrographs are included as ensemble members.

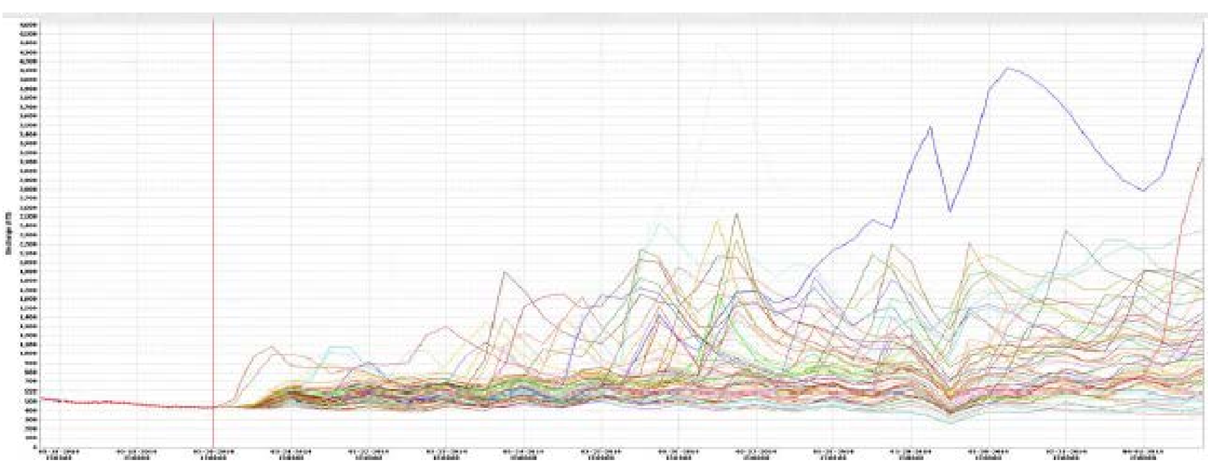


Figure D- 2: Illustration of Ensemble Forecast at a Single Location in the Watershed

Information about likely future inflows to Folsom—accounting for uncertainty in weather forecasts—can be derived from analysis of the ensemble. For example, 3-day volumes can be computed for the traces, and frequency analyzed to assess probability of exceeding specified volume thresholds in the future.

D 1.1 Hindcasts

In order to assess forecast quality and reliability, retrospective forecasts (hindcasts) of Folsom inflow were generated by the CNRFC using the Hydrologic Ensemble Forecast Service (HEFS) software. To create the hindcasts for Folsom, CNRFC analysts first generated historical flows by running the American River forecasting model continuously with observed weather conditions (temperature and precipitation) from water year 1981 through 2010. The analysts then stored the watershed states (warm states) of the model for every day during that period prior to running the hindcasts.

The inflow hindcasts were generated by looping through the American River forecasting model one day at a time. For a given hindcast day, appropriate warm states were selected from the stored data set. The hydrology models were then forced with the meteorological forecasts (precipitation and temperature) from the NCEP operational GEFS reforecast dataset. The inflow hindcasts were computed 1 day at a time, and archived for verification purposes.

The hindcast dataset contains daily ensemble hindcasts for the 1985-2010 period, resulting in over 9,000 days of ensemble inflow forecasts at lead times 1-15 days. The hydrology and atmospheric models used in the hindcast process are consistent with what is used operationally; however, the hindcasting procedure is automated, so it does not include information added in practice by forecasters and hydrologists. Thus, it is not an exact representation of the forecasts provided. Nevertheless, the resulting hydrographs provided a large, consistent, realistic sample of forecasts for testing alternative Folsom operation strategies. The corresponding hydrographs also provide a systematic dataset that can be compared with observed hydrographs to assess forecast quality.

D 1.1.1 Forecast Quality and Reliability of Ensemble Data

After creating all of the hindcast data, a large set of verification graphics were generated using the Ensemble Verification Service (EVS) program. Many statistical metrics were examined including correlations, Brier Scores, reliability diagrams, root mean square error, and many more. Forecast quality was assessed at different lead times, flow thresholds, and seasons. **Figure D- 3** describes the correlation between mean ensemble 5-day volume forecasts and the corresponding observations. Correlation values are very high for shorter lead times, and fall steadily out to day 15 as forecast skill diminishes.

A correlation metric does not say anything about reliability and bias. A reliability diagram helps describe conditional bias for discrete events. If a forecast is perfectly reliable, an observed inflow volume should occur with the same relative frequency as the forecast probability over a large sample size. This is indicated by the black line in **Figure D- 4**. The dashed red line represents hindcast results for the top 10percent of all forecast 5-day volumes (forecast volumes greater than 90 thousand acre-feet (KAF)) at lead time of 140 hours.

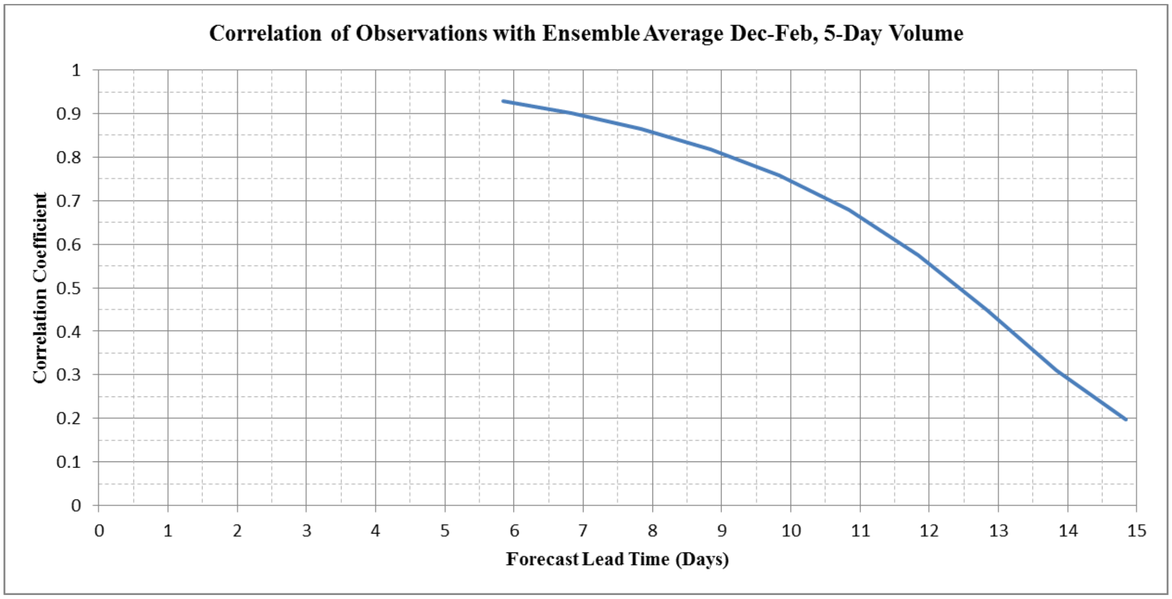


Figure D- 3: Illustration of Correlation between 5-day Forecast Ensemble Mean with the Observations for Forecast Lead Times Going out to 15 Days, for Forecasts Greater than 90 KAF

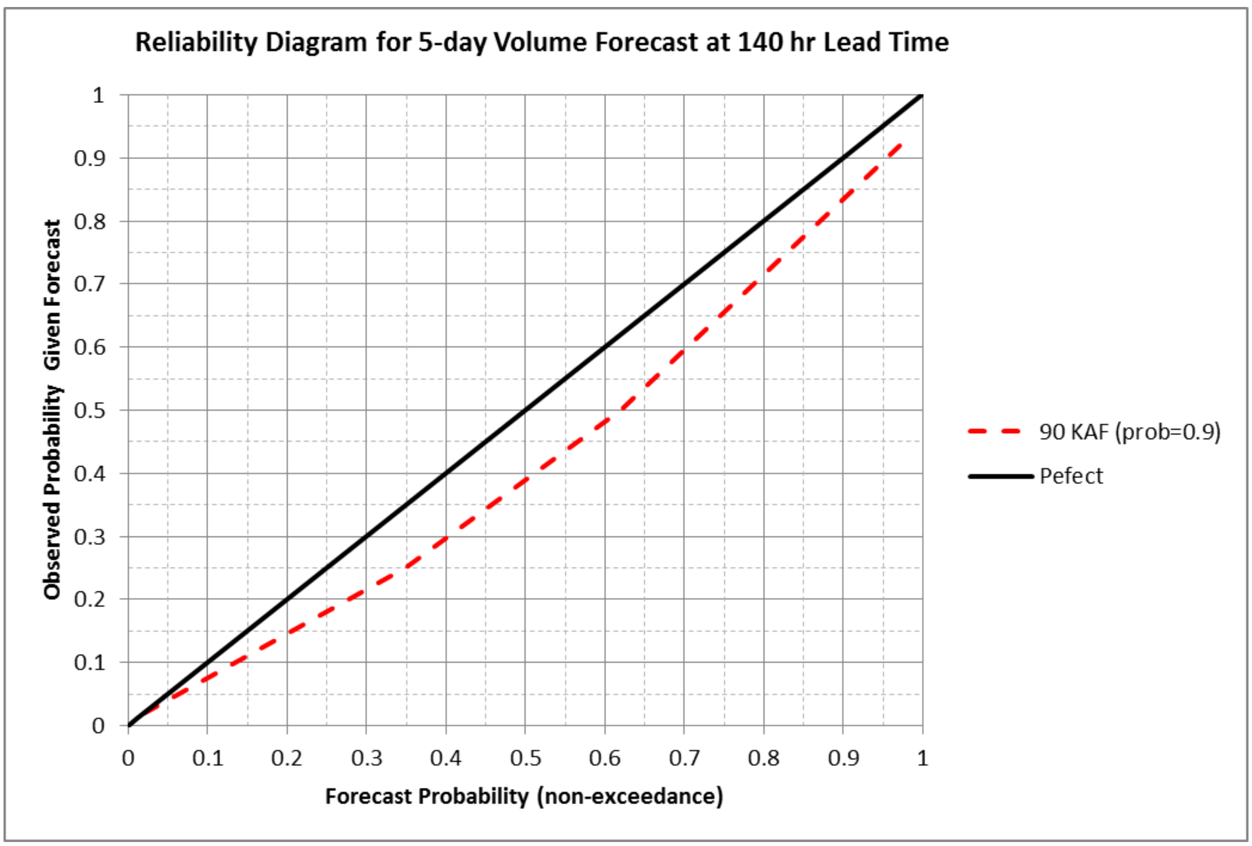


Figure D- 4: Illustration of Forecast Reliability for a 5-Day Volume Forecast at a 140-hour Lead Time for the Dec-Feb Time Period

In order for these types of verification metrics to be statistically meaningful, adequate sample sizes are necessary. Therefore, these metrics do a good job of describing forecast quality for a fairly broad range of events. However, a main focus of the forecast alternative (J602F1) is to safely manage very large, rare floods. Therefore, the statistical verification metrics desribed previously are not appropriate when examining forecast quality for extremely large events. Qualitative visualization graphics, such as modified box plots, are more appropriate when dealing with very small sample sizes. The modified box plots in **Figure D- 5** and **Figure D- 6** compare forecast errors to the observations. Each box plot in the figures represents the forecast errors (forecast volume minus observed volume) for a given daily forecast. The lead times do not line up exactly with the forecast volume duration. For example, **Figure D- 5** describes the forecast error for 3-day volume forecasts, but the associated lead time is 92 hours, not 72 hours. This has to do with the daily time scale of the observations (midnight to midnight PST). Hindcast datasets are initiated at 12Z each day, so the first 20 hours of forecasts were ignored, and the hindcast datasets were aggregated on an 8Z-8Z scale to align with the observed daily time scale.

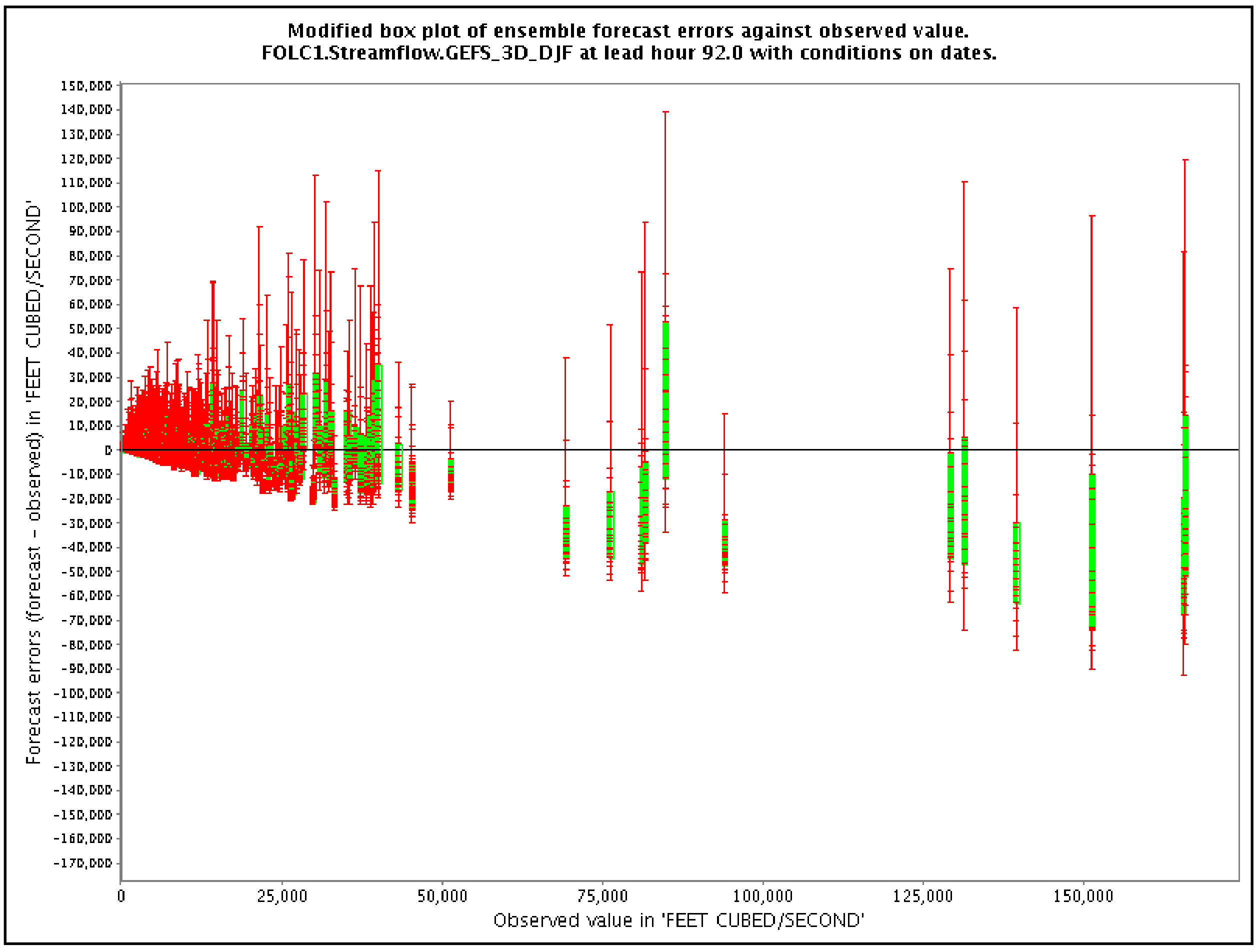


Figure D- 5: Illustration of the Folsom Forecast Error for a 3-day Average Flow Forecast at a 92-hour Lead Time for the Dec-Feb Time Period

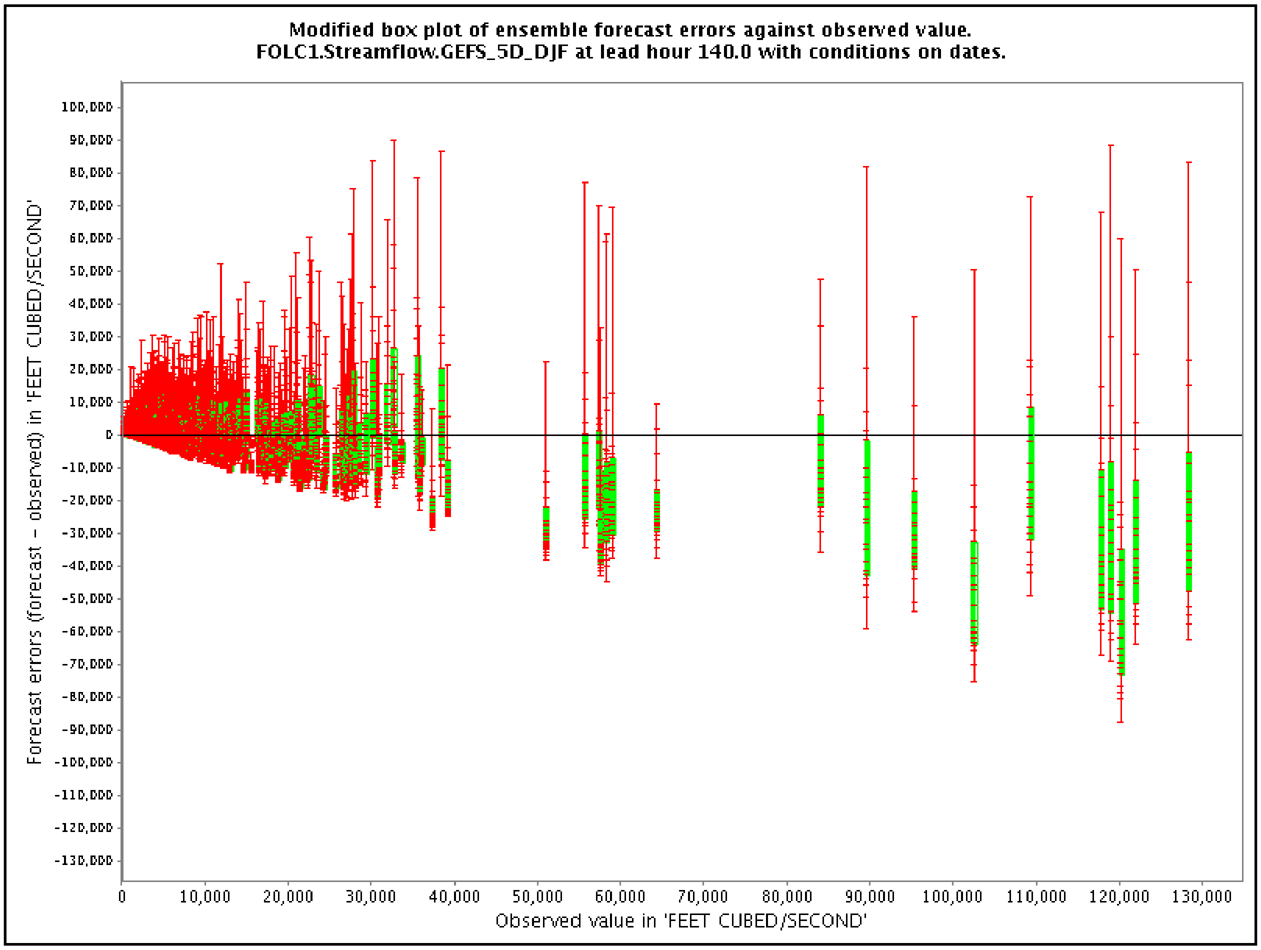


Figure D- 6: Illustration of Folsom Forecast Error for a 5-day Average Flow Forecast at a 140-hour Lead Time for the Dec-Feb Time Period

The box plots are arranged by increasing observed value. The green shading represents the 20-80 percent range. The largest and smallest tick marks represent the maximum and minimum, and the other tick marks represent error at 5 percent increments. It is evident in both graphics that there is a dry bias for the large flood events, where the median error is always below the horizontal zero line. However, the spread of the ensembles is adequate. The observation always falls within the spread of the errors for all of the medium to large events.

Folsom is not the only watershed in northern California where similar hindcast verification has been completed. The Feather-Yuba Forecast Coordinated Operations Study is another ongoing effort in the Sierra that is also investigating the value of using forecasts to improve reservoir management. As part of that study, hindcasts and verification analysis was also done for Lake Oroville, New Bullards Bar Reservoir, and Englebright Lake. **Figure D- 7** through **Figure D-10** show similar modified box plots for both Lake Oroville and New Bullards Bar Reservoir.

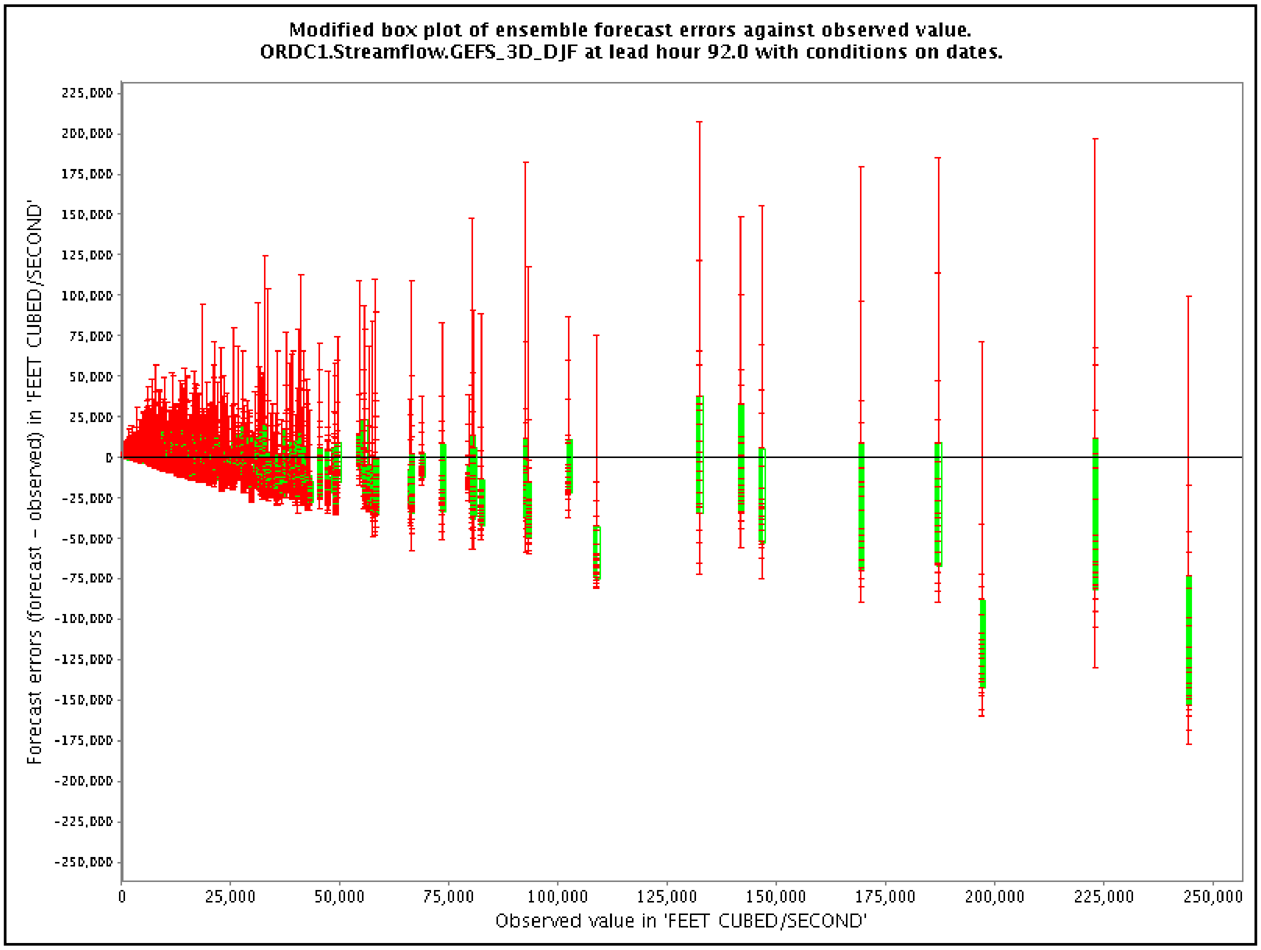


Figure D- 7: Illustration of Lake Oroville Forecast Error for a 3-day Average Flow Forecast at a 92-hour Lead Time for the Dec-Feb Time Period

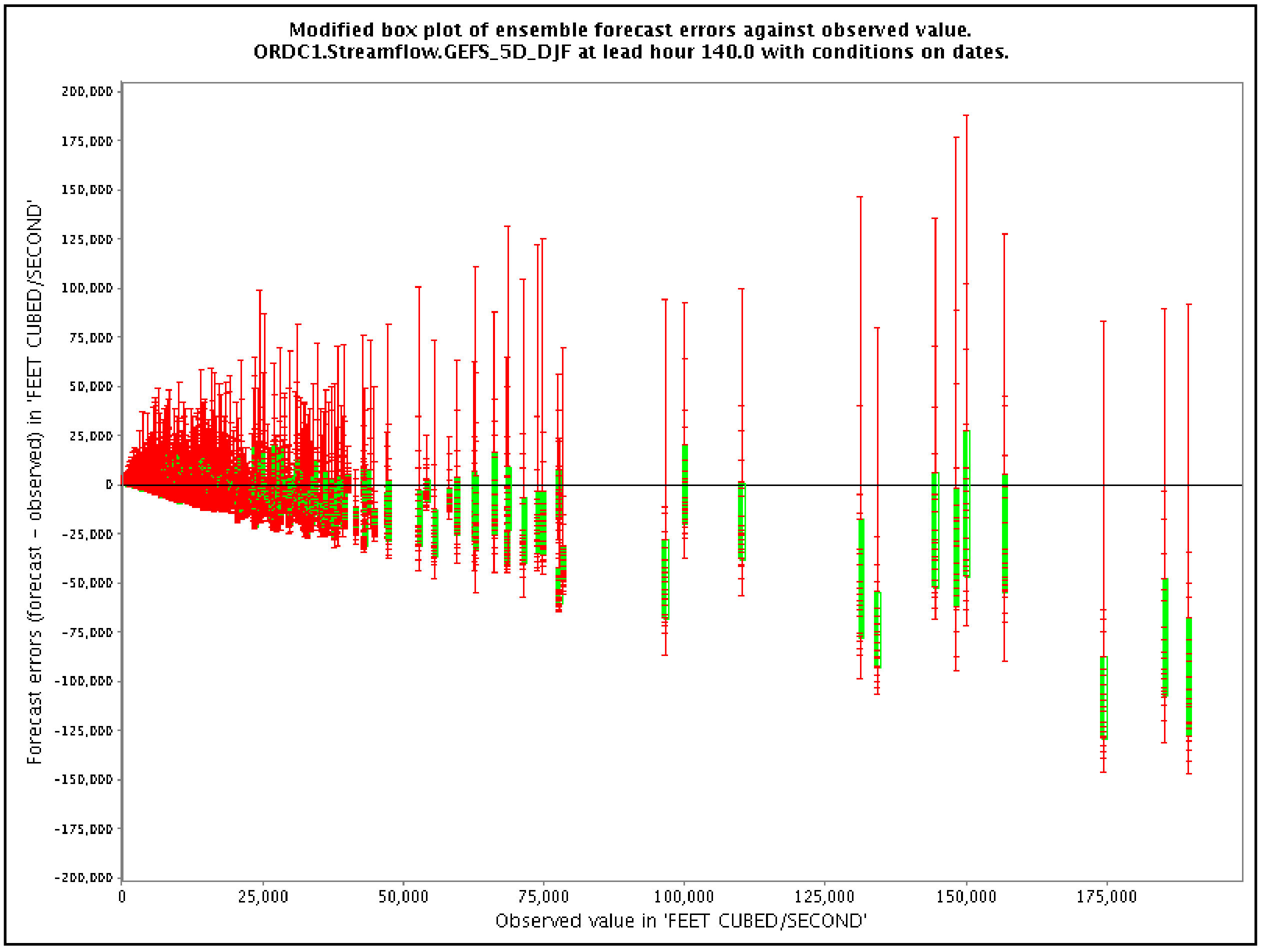


Figure D- 8: Illustration of Lake Oroville Forecast Error for a 5-day Average Flow Forecast at a 140-hour Lead Time for the Dec-Feb Time Period

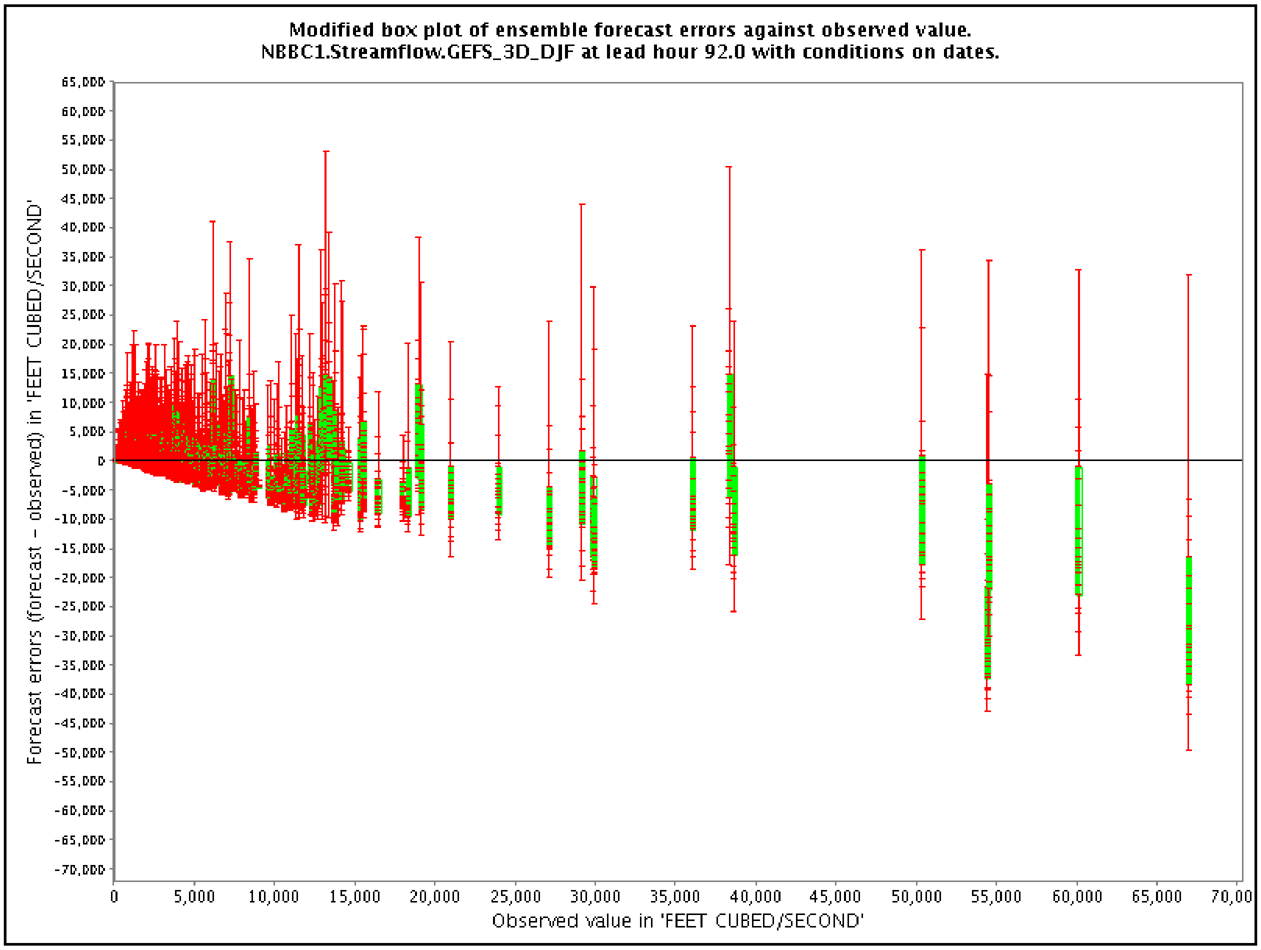


Figure D- 9: Illustration of New Bullards Bar Reservoir Forecast Error for a 3-day Average Flow Forecast at a 92-hour Lead Time for the Dec-Feb Time Period

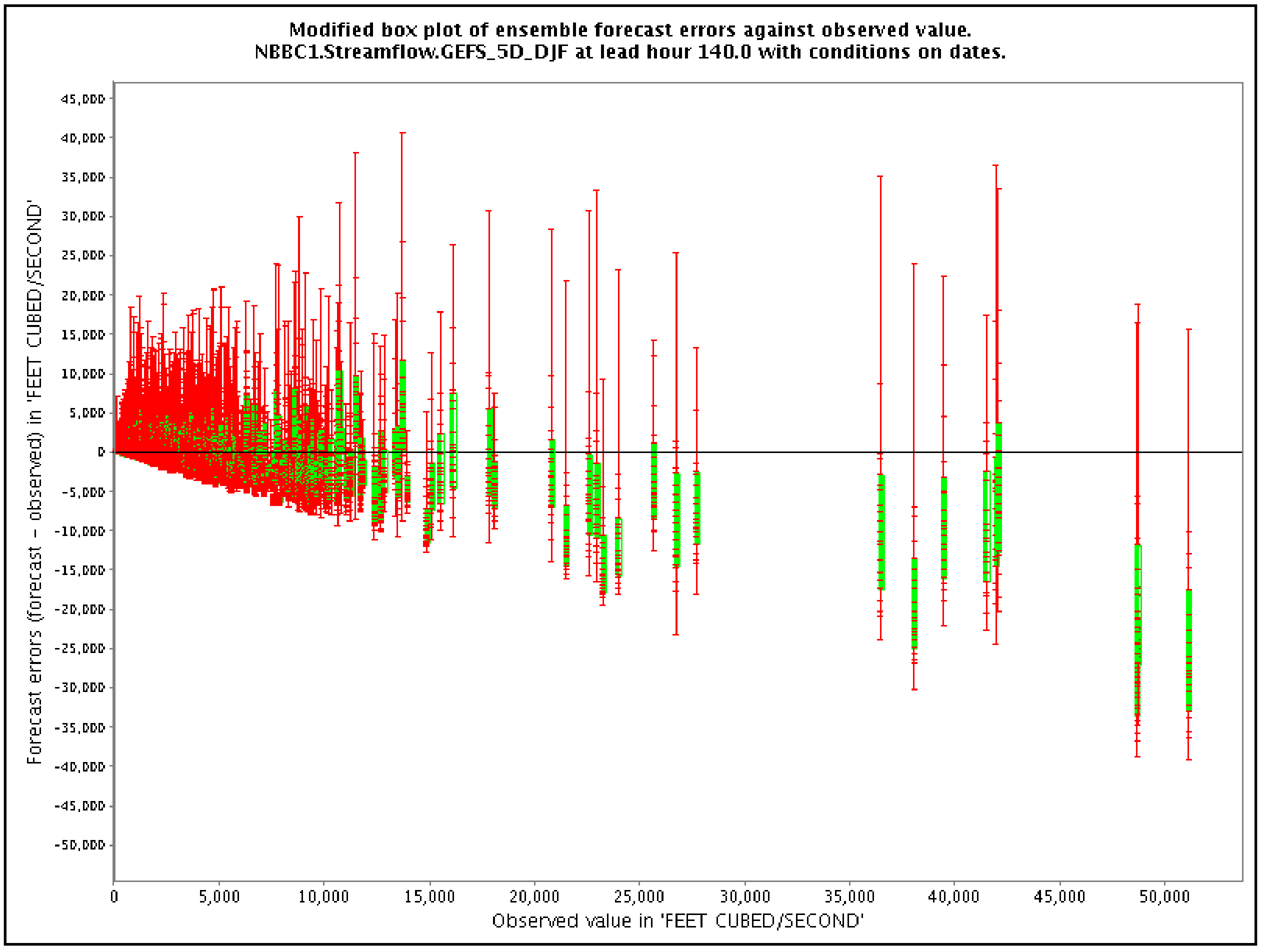


Figure D-10: Illustration of a New Bullards Bar Reservoir Forecast Error for a 5-day Average Flow Forecast at a 140-hour Lead Time for the Dec-Feb Time Period

Very similar forecast quality characteristics are evident between all three of these northern Sierra reservoirs. There is a tendency for the median to be under-forecasted, but the spread of the ensembles appears to be appropriate for the largest events.

Safely managing the 1/200 ACE event is a primary goal for the forecast alternative. Unfortunately, there has not been an event that large in the period of record at any of these three major reservoirs. So there is uncertainty as to what the forecast characteristics will be for events larger than what has been observed. However, the consistent forecast characteristics observed in the largest floods for all three of these reservoirs strengthens confidence that similar behavior can be expected for larger events such as the 1/200 ACE flood.

D 1.1.2 Hindcast Dataset

CNRFC provided a complete set of hindcast data (probabilistic volumes received on 14 May 2015) spanning the time frame from 1 October 1985 to 30 September 2002 (NOAA NWS 2013).

In order to test the forecast-based operation for extreme events, the largest flood events in the record hindcast record were identified and scaled ensemble hindcast datasets developed. The largest two events were the 1986 and 1997 events. Scaled versions of these events were developed for the 1/100 ACE and 1/200 ACE critical volumes. For this, the 3-day volume was used for the 1986 pattern and the 2-day volume was used for the 1997 pattern.

To develop the four scaled hindcast datasets, the historical events were first reconstituted using the CNRFC American River SAC-SMA/SNOW-17 model. The initial conditions for these model runs used historical model states from the period of record (model run that spanned 1948-2010). Initial conditions were extracted from this period of record run, and used as “cold states” to initiate the shorter 1986 and 1997 event historical runs. The initial model states for the 1986 event were from 2/6/1986 and 12/22/1996 for the 1997 event. This historical model run was done in full natural flow mode. A precipitation modifier was applied to the maximum 5-day window for each event (i.e., scaling factor). The historical simulation was re-run iteratively until an unregulated 1/100 ACE 3-day average flow was met for the 1986 event and a 1/100 ACE 2-day average flow for the 1997 event was met. This same procedure was done for the 1/200 ACE events. The 5-day maximum precipitation window for the 1986 event pattern is 2/14/1986 18:00 GMT through 2/19/1986 12:00 GMT, and for the 1997 event pattern is 12/29/1996 18:00 GMT through 1/3/1997 12:00 GMT.

In order to achieve the 1/100 ACE 3-day flow of 185,825 cfs, the 1986 max 5-day precipitation had to be scaled up by a factor of 1.08. Similarly, the 1997 needed a scaling factor of 1.08 to match the 1/100 ACE 2-day flow (215,117 cfs). In order to achieve the 1/200 ACE 3-day flow of 235,628 cfs, the 1986 max 5-day precipitation had to be scaled up by a factor of 1.335. Similarly, the 1997 needed a scaling factor of 1.34 to match the 1/200 ACE 2-day flow (272,642 cfs).

Once these scaling factors were determined, new basin states (i.e. warm states) were saved for every day in the event window. So the warm states reflected the conditions from the scaled down version of the historical event, not the actual historical event. The warm states saved for the 1986 event pattern are 2/6/1986 through 2/28/1986, and for the 1997 event pattern are 12/22/1996 through 1/10/1997. These model states were then imported to the regulated CNRFC American River model. This model includes Union Valley, Hell Hole, French Meadows, Ice House, and Loon Lake. The initial conditions for these reservoirs were set to “20 percent storage.” Lower level outlet releases and tunnel releases were assumed to be near maximum capacity release, and spillway gates were assumed to be completely open.

The regulated CNRFC model was then run in hindcast mode applying the same precipitation modifiers used in the full natural flow historical simulation run. These modifiers were applied to every precipitation ensemble member for every hindcast day. Daily hindcasts were generated for 1/100 and 1/200 year historical patterns (1986 and 1997). The daily hindcasts for the 1986 pattern spanned 2/7/1986 through 2/28/1986 and the 1997 hindcasts spanned 12/23/1996 through 1/10/1997.

For both the period of record hindcast dataset and the scaled hindcast event dataset, the ensemble members were provided. For the forecast rule development initially, these members were processed to produce probabilistic volumes corresponding to the durations of interest. As noted above, these hindcasts are developed one per day. CNRFC is developing the capability to generate hindcasts on more frequent intervals and to implement this capability to support a

Folsom forecast-based operation. Until this capability is implemented, hindcasts on 6-hour interval are estimated by offsetting the averaging periods of interest at 6-hour blocks. This offsetting mimics a new hindcast volume set being developed every 6 hours.

Appendix E – Regulated Frequency Curve Development

This appendix describes how the regulated frequency curve is developed from simulated results of scaled events. Key concepts it's development are the computation critical duration for each event pattern, and weighting of the pattern-specific regulated curves to obtain a single curve. The "analytical curves" seen in Figure 6-11 in the main report were developed using this method.

E 1 Use of Regulated Frequency Curves in Water Control Manual Update

This section describes the use of regulated flow frequency curves to support flood risk analyses and to provide an estimate of the level of protection provided by a reservoir. As such, these regulated flow frequency curves reflect discharges resulting from scaled versions of historical events having assigned probabilities. Development of these curves, and the process for assigning probabilities are described in later sections.

Regulated frequency curves are defined at each analysis location by specifying the analytical unregulated flow frequency curve and corresponding unregulated to regulated flow transform. The regulated flow frequency curves will be used to support the comparison of alternatives. For a specific event pattern, Folsom outflow frequency curves, corresponding to alternatives of interest, can be overlaid. This will provide an informative comparison of reservoir operations over a full range of event magnitudes.

For this engineering report, two types of regulated frequency curves are produced. Simplified, event-specific flow frequency curves will be produced with the assumption that all scale factors of the pattern flood have the same critical duration (i.e., two or three days). This simplified event-specific curve is relatively easy to produce and is a useful method to compare reservoir alternatives to each other for performance. For discussion purposes in this appendix, these curves shall be referred to as "simplified event-specific curves". Again, these curves are preliminary curves that are useful to compare reservoir alternatives to each other.

In reality, critical duration is not only sensitive to the pattern of the unregulated hydrograph, but also to the size of the flood. The volume that is important in driving the peak outflow (i.e., the critical duration) in a 1/200 ACE event is not necessarily the same duration that drives the peak outflow in a more common flood such as the 1/5 ACE event. Reservoir operations during a common event are significantly different than those for a rare flood. To capture the performance and level of protection provided by the final selected alternatives, a more detailed analysis of critical duration is performed, and each scale factor of a pattern flood is assigned its own unique critical duration. A frequency curve is developed for each pattern event, then all four regulated frequency curves are weighted and combined to produce a final adopted regulated peak flow frequency curve. For discussion purposes, this type of curve shall be called a "final adopted curve".

E 2 Overview of Regulated Frequency Curves

The following paragraphs describe the approach used for developing regulated frequency curves for the analysis of flood risk. The level of protection from flooding provided by the Folsom project must be assessed as part of the Manual Update. Part of the assessment requires examining whether operating rules can safely provide a minimum level of protection for downstream areas throughout the year. For Folsom Dam, a successful operation set should achieve two goals: the 1/200 ACE peak outflow should be kept to a maximum of 160,000 cfs or less, and the 1/100 ACE peak outflow should be 115,000 cfs or less.

One indicator of project performance is a regulated (peak) flow frequency curve, in which the peak outflows of the river system impacted by Folsom Dam and the upstream system of reservoirs and diversions are mapped onto a range of probabilities. Since the peak outflows with the JFP and the new operating set are only hypothetical, a collection of large historical events and scaled versions of these events were developed (see Chapter 4 for details) and simulated to test the regulated system models (Chapter 7). The scaled events are needed to understand the response of the system to a wide magnitude of floods, especially events that are rarer than represented in the historical record.

From these simulations, a method is needed to develop the regulated flow frequency curve. The challenge is to relate unregulated event flows of known probability to regulated peak flows. As described in Chapter 4, the Manual Update applied the methods of Bulletin 17B to create a family of volume-duration curves for the American River at Fair Oaks. The underlying assumption is that for a given event temporal distribution (pattern), there is a “critical” duration that is the best indicator of regulated peak flow. Once the critical duration is identified, the probability associated with the unregulated flow volume of critical duration is assigned to the regulated peak flow.

E 3 Critical Duration: Volume-Window Approach

E 3.1 Theoretical Background

An early use of the concept of critical duration, if not the term itself, appears in Beard’s Statistical Methods in Hydrology (1962). An example of sizing a reservoir capacity is given by plotting a constant project release volume tangent to the 100-year volume-duration mass curve (i.e. a curve composed of return period volumes plotted against discrete durations). The section concludes by stating that “[t]he curve also indicates that durations of 4 to 7 days are critical for this project release and flood control space.” Here, a connection is made between a volume associated with a targeted ACE (in this case, 1/100) and the ability of the controlled system to release at a designed rate.

Later guidance further develops this concept of critical duration. Throughout EM 1110-2-1420 (“Hydrologic Engineering Requirements for Reservoirs”) use is made of critical duration. In Section 10-1 (c), the factors for critical duration are described: “The critical durations will be a function of the degree of flood protection selected and of the release rate or maximum rate of flow at the key downstream control point." With this wording both a level of desired protection, or recurrence interval, and an objective flow are implicated as factors in finding the duration.

Generally speaking, then, there is an unregulated volume that stresses a regulated system beyond its designed capacity. However, since regulated outflow means that the available storage of the regulated system is in flux during an event, a method is needed to identify a time-span during which an inflow accumulates to a point that an undesired outflow occurs. Taken in this sense, critical duration is a means of expressing the accumulated time that an average inflow takes to force the system above the targeted controlled flow.

Presented here is a method for determining critical duration that quantifies the degree to which a volume associated both with a targeted ACE and a specific n-day duration occurs before a peak regulated outflow. Briefly, though, it is worthwhile to look at two methods employed in Corps studies, both to see how the proposed method builds upon the principals of each and how it overcomes some shortcomings.

E 3.1.1 Non-Sequential Mass Curve

Though not identified by this name, the term "non-sequential mass curve" may be used to describe the method presented in Beard's work, as described above. In this graphical technique, volumes of a given return period are plotted for various durations, with line segments connecting these points:

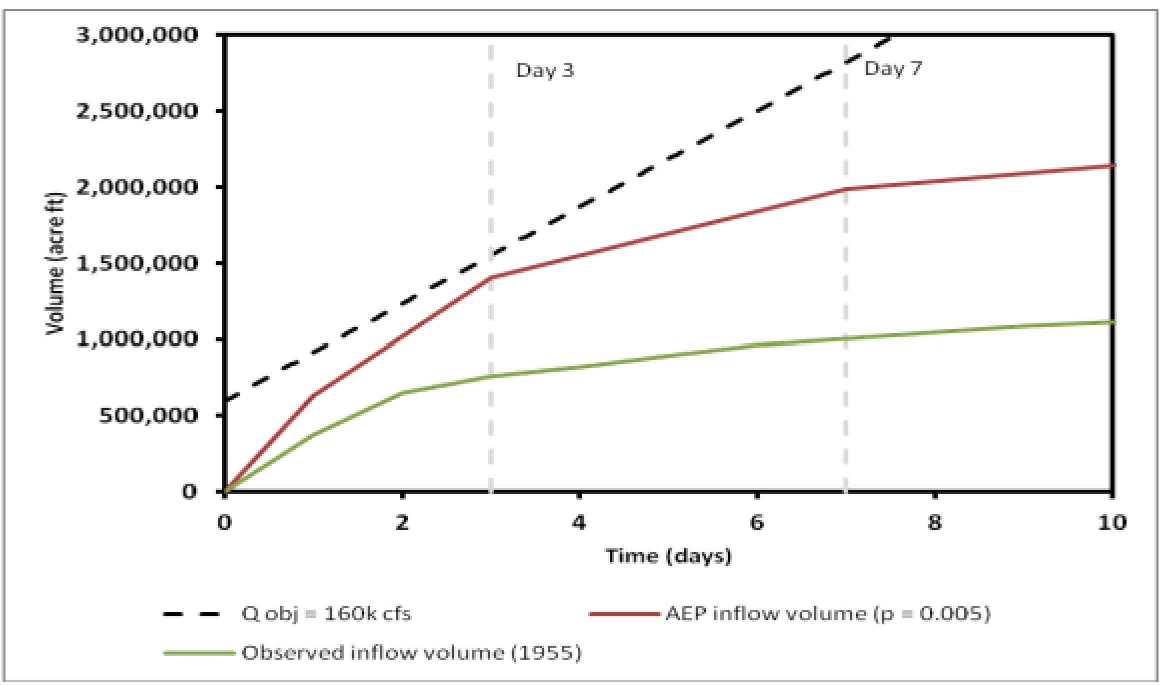


Figure E- 1: The Non-Sequential Mass Curve Method, with Both a Fully Balanced 1/200 ACE Inflow Volume and an Observed Event Plotted

It is important to note, however, that the x-axis scale represents discrete durations and that the line segments can provide an approximation of the ACE volumes that would be found for intermediate durations developed through frequency analysis. Starting from a fixed storage volume (total flood storage), a line, the slope of which shows the fixed rate at which storage volume can be passed downstream, is plotted against the inflow mass curve. The rate of outflow would be known, presumably, from reservoir operations. Note: The outflow is often described by the objective downstream flow rate or perhaps channel capacity. The point at which the inflow curve is tangent to the reservoir storage curve thus yields the critical duration. In **Figure E- 1**, we see that the 3-day volume would first come in contact with the outflow line; hence, by this analysis, a duration of 3 days would be critical.

This method provides a clear picture of the relationship between inflow over the course of an event and the point at which storage is maximized, thus potentially forcing outflow above objective targets. Although the original method, as applied for Beard, used the same return period for all durations (i.e. balanced events), the method may be applied to historical events which typically have different return periods for each duration (i.e. unbalanced events), sometimes markedly so. In **Figure E- 1**, using the maximum n-day volumes for the 1955 flood event, we see that on the mass curve the points closest to the outflow volume curve are found at 1 and 2 days.

While a real (or scaled) event can be used, real (or hypothetical) operations cannot, unfortunately. Because of the non-sequential nature of the plot, the diagram allows for only a simple constant outflow and starting storage model. A constant inflow is also assumed which misses critical details such as the timing of peaks, the number of peaks, etc. As a result, too, it is well-defined for an analysis point below a single reservoir system, for which local flows play no significant role, such as American River at Fair Oaks.

E 3.1.2 Ranking Correlation Method

Another method has been utilized in Corps studies along the Lower Mississippi and Des Moines Rivers, among others. Termed here as the “ranking correlation method,” the concept behind this technique is that, for some n-day duration, the maximum inflows for the largest events of record (or simulation), when ranked in descending order, will tend to rank in the same order as the ranked maximum outflows for those same events. The duration, n, that most often follows this order will thus be critical. In the hypothetical example shown in Table E-1, the 3-day unregulated flow is chosen as the critical duration.

Table E-1: Ranking Correlation Method

Flood Ranking by Historic Peak Outflow	Historic Peak Outflow (cfs)	Flood Ranking Based on Maximum 1-day Unregulated Flow	Flood Ranking Based on Maximum 2-day Unregulated Flow	Flood Ranking Based on Maximum 3-day Unregulated Flow
1969	135,000	1983	1910	1969
1910	115,000	1973	1969	1910
1973	93,000	1910	1973	1973
1983	87,000	1969	1983	1983

The appeal of this method is that, given several large events of interest, a consistent duration may be found that drives the maximum system response. Besides a visual inspection of ordered lists, the analyst may make use of statistical tests (e.g. null-hypothesis tests). Unlike the non-sequential mass curve, this approach may be used broadly for any given index point, as no system conditions need be known in order to rank the volumes. However, a pitfall of correlating the total inflow volume with the peak is the lack of consideration given to the role of timing. Although a duration correlates well, if only a portion of the total volume is used, it is debatable as to whether the entire duration is required to stress the flood storage capacity of the system. Also, it may not be desirable to characterize a point with a single critical duration. As cited in the example from Beard (1962), or similar conclusions in other Corps reports, several durations may need to be considered as a result of very different inflow hydrographs shapes.

E 3.1.3 Volume-Window Method

The volume window method, the approach for quantifying critical duration for the Manual Update, relates the timing of the peak storage to the maximum n-day unregulated inflow volumes for an event. Within Section 3 of Appendix E, the term “volume” should be thought of as unregulated inflow volume to the reservoir. The timing of peak storages suffices as a stand-in for the time of peak outflow. Unlike regulated flow, which may hold at a certain objective threshold, reservoir storage clearly peaks. In absence of significant runoff below the dam, total flow will decrease only after the point of maximum storage. So in that sense, the maximum storage is the driver of maximum outflow.

A “volume window” is simply the period of “n”days during which the average volume is of a greater magnitude than any other n-day period. Since a storm can have only one maximum 1-day, 2-day, 3-day, etc. period, the n-day volume windows are a characteristic of a hydrograph. It is the relationship between the timing of the peak storage and these windows that is crucial to understanding why one duration may be critical. Figure E-2 will be used to illustrate this concept. The method supposes that, in general, the closer that the peak regulated flow occurs to the end of a given volume window, the more “critical” that duration is in stressing the regulated system. However, since it is the cumulative unregulated inflow volume that is the driver, a way of quantifying the timing in terms of volume would better describe the relationship, such as:

$$\mathbf{VW(n) = Vp/Vmax\ \%}$$

where **VW(n)** is the volume-window percentage for duration n,
Vp is the volume from the beginning of the window to the time of peak storage, and
Vmax is the total n-day volume.

In this formulation, Vp could represent (Vmax + Vex), where Vex is the volume in excess of the n-day volume, and as such, would produce a VW(n) greater than 100%. A negative percentage is used to represent cases for which the peak occurs before the window start.

Per Figure E-2, the following computation is made for the 2-day duration:

$$Vp = 218 \text{ kcfs} \times 1.9835 \times 1.8 \text{ days} = 778,325 \text{ ac-ft}$$
$$Vmax = 218 \text{ kcfs} \times 1.9835 \times 2 \text{ days} = 864,806 \text{ ac-ft}$$
$$VW(2) = Vp / Vmax \,\% = 90\%$$

The volume window builds off the strengths of the two methods described above as used in Corps documents. Like the non-sequential mass curves, a clear visual relation between the inflow and outflow can be drawn. But, while the windows are essentially non-sequential, the method uses inflow and outflow hydrographs, thereby making use of actual regulated operations, like the ranking method. However, whereas the ranking method doesn't consider the role of timing, the volume window method does.

The major benefits of the approach are summarized in the following:

- Relates timing of the peak to the maximum n-day volume for the event,
- Provides easy visual identification of the how these maximum windows vary according to the hydrograph shape, and
- Makes use of actual reservoir operations/simulations thus allowing for variable outflow.

The method therefore lends itself to an approach based on reservoir modeling of historical and scaled events. This keeps with the guidance from EM 1110-4-1420: "If this critical duration corresponds to the duration of a single rainstorm period or a single snowmelt event, the computation of hypothetical floods from rainfall and snowmelt can constitute the principle hydrologic design event."

E 3.2 Application to Historical Events on the American River

Several historical hydrographs were scaled by factors ranging from 0.2 to 3.0 and processed with an HEC ResSim model, thereby producing a regulated-unregulated pair for each scaling factor. (See Chapter 4 for a description of the input hydrology and Chapter 5 for background on the models.) Four events of significance (known damaging floods, characteristic shapes, etc.) were selected. Using the unregulated input hydrograph and the output of the ResSim model for each simulation, the steps below determined a critical duration.

1. Determine the maximum n-day volume (Vmax) and the beginning and ending of the period over which the volume accumulates for the unregulated hydrograph. (The magnitude of each window scales directly with the factor applied, while the timing is not altered.)
2. Calculate the total unregulated volume from the beginning of the n-day maximum volume window to the time of peak storage (Vp) for the unregulated hydrograph.
3. Calculate VW(n) by dividing Vp by Vmax for each duration of interest; if the peak occurs after the end of the n-day volume window, this ratio should be expressed as a value greater than 100%; if it occurs before the start of the window, it is expressed as a negative percentage.

4. Rank the n-day ratios according to their proximity to 100%; for example, given four percentages – 118, 90, 88 and 85 percent – a ranking of “1” will be given to the n-day that yields a 90 percent, a ranking of “2” for the 88 percent, a ranking of “3” to the 85 percent and a ranking of “4” to the 118 percent.

The graphical results of this approach may be demonstrated in **Figure E- 2**, which is based on routing an unregulated 1997 event inflow hydrograph through a reservoir model.

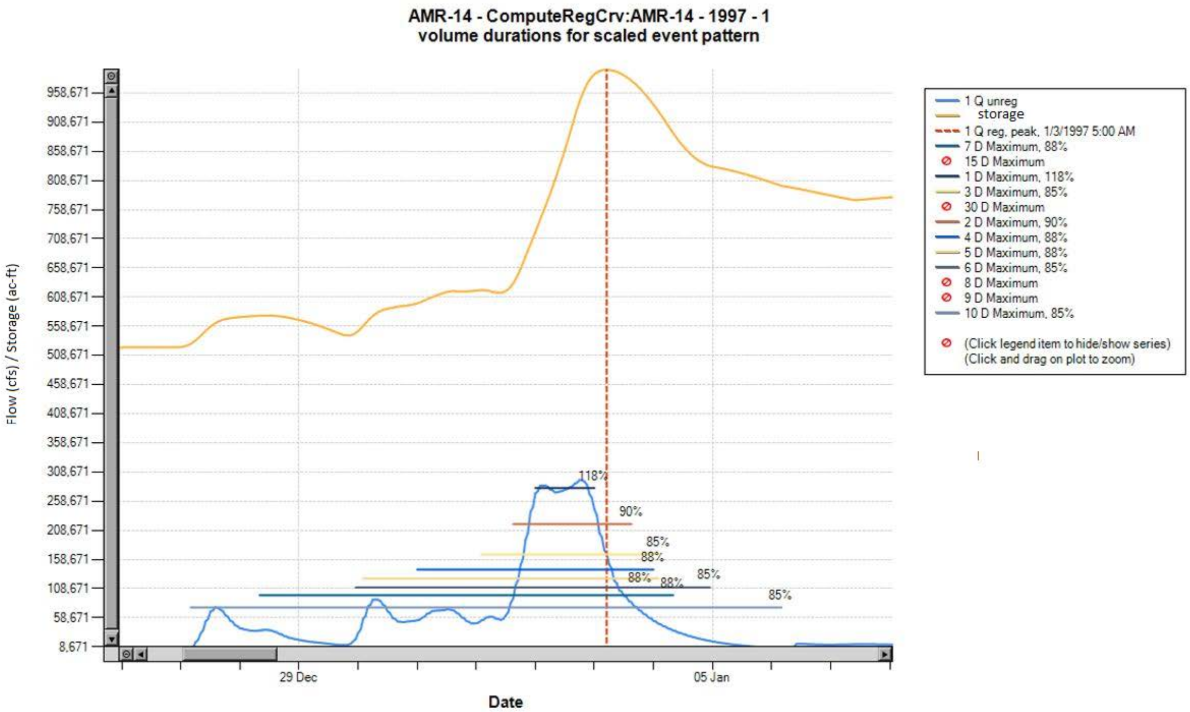


Figure E- 2: Critical Duration Method
Note: Critical duration selected from the timing of the peak reservoir storage. Peak storage in this instance provides a strong indicator that the 2-day (90-percent) and 3-day (85 percent) durations are stressing the reservoir. The 2-day duration is selected because its volume window percentage is closer to 100-percent.

Figure E- 2 demonstrates the volume window method. The unregulated hydrograph is plotted in blue. Reservoir storage over time is plotted at the top of the figure in yellow. The time of peak storage is indicated by a dashed vertical line. The various colored horizontal lines indicate the timing and magnitude of the maximum n-day unregulated volume in the unregulated hydrograph. The 2-day duration is given a ranking of 1 in the above example since the percent of volume that occurred at the time of peak storage was closest to 100%.

The critical duration is assigned by ranking the n-day volumes based on the closeness of VW(n) to 100%. For a single hydrograph, the critical duration may be thought of as the n-day volume with the highest ranking. However, a few considerations may still be necessary in selecting a critical duration. Outside of a given window (e.g. a range of 90 to 110%), it may be less informative to identify a duration as critical. For example, while the 2-day duration may have a ranking of 1, if only 50% of the 2-day volume was used, it may not be correct to say two days is critical for that pattern. On the other hand for several durations which have very close percentages, the ranking may be influenced by the concurrent timing of the ends of the windows (e.g. the 1-day window ends at nearly the same time as the 3- and 5-day windows), in which case it is recommended that the reservoir operations and/or the hydraulic model simulations should be examined closely to make a judgment as to which duration is truly driving the peak outflow.

It is also good to note that several, or every, event-scaled historical hydrograph may have a unique critical duration based on the above criteria. It is up to the analyst to decide whether a single duration may be used to characterize the analysis location, or whether it is more appropriate to regard the events as different enough to warrant a separate critical duration for each. Therefore, it may be necessary, in order to characterize a location, to select a representative range of observed or probable hydrograph shapes.

In order to compare one ROS to another simplified event-specific frequency curves will be generated in which one critical duration is assigned to all scale factors of a pattern flood. For the final adopted curves, critical duration will be allowed to change for every scale factor. The sections that follow will demonstrate the difference between how a simplified event-specific curve is generated versus the process to create a final adopted curve that will be placed in the Water Control Manual.

E 4 Simplified Event-specific Frequency Curves

The previous section describes a procedure for identifying critical duration. For the purposes of comparing various ROS to each other, one critical duration was assigned to each pattern flood. This was done by routing scaled versions of each flood pattern through the J602 model. The critical duration that was determined for the scale factor that was closest to the 1/200 ACE event was assigned to the whole pattern.

Creating flow transforms: Once critical duration has been assigned to a pattern, one can create an unregulated to regulated flow transform. For a given pattern, a collection of flow pairs are generated by simulating scaled versions of the event in HEC-ResSim. Each scaling results in an unregulated volume corresponding to the critical duration, and a regulated peak flow. An event-specific unregulated to regulated transform is generated by connecting (interpolating between) flow pairs. An example transform in shown in **Figure E- 3**. Computed flow pairs resulting from simulations of scaled versions of the 1997 event are shown as diamonds. The interpolated transform is shown as a solid line.

Once a flow transform has been created, one can create a simplified event-specific frequency curve by replacing unregulated volume on the x-axis with the probability of the volume (based on the critical duration unregulated frequency curve).

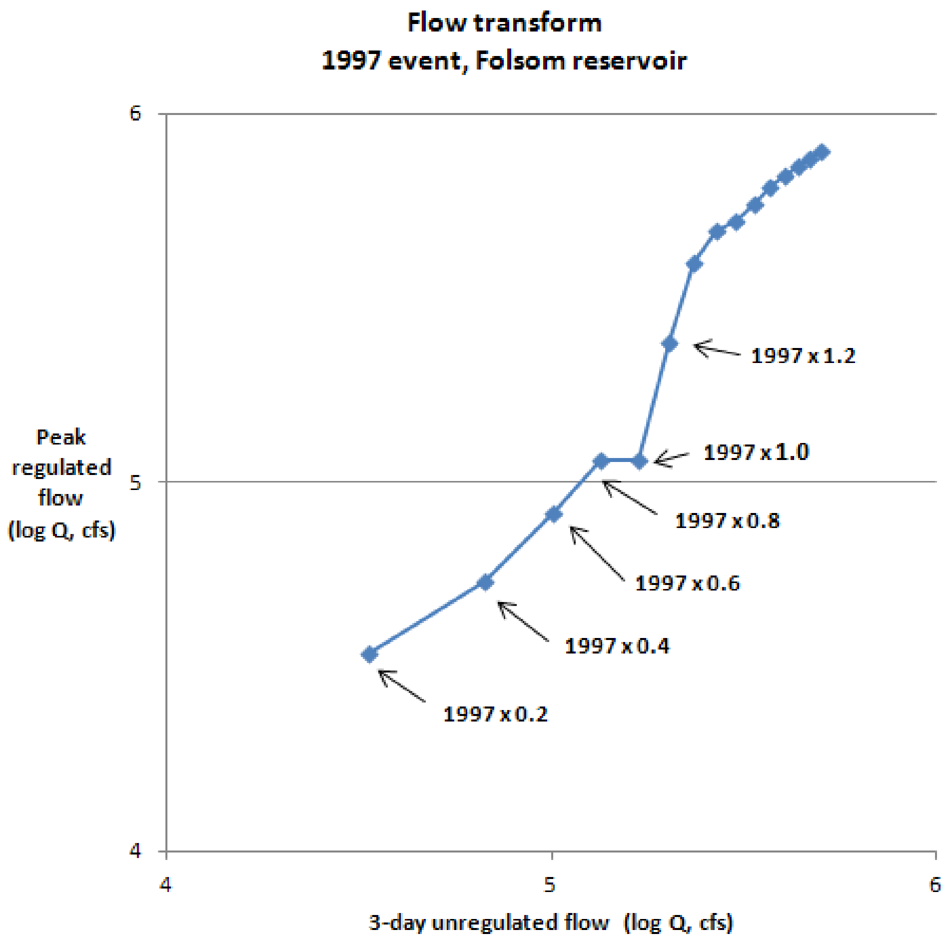


Figure E- 3: Example of Unregulated to Regulated Flow Transform

Event-specific (conditional to the pattern being used) probabilities are assigned to the unregulated and regulated peak flow pairs from the unregulated volume frequency curve corresponding to the event critical duration. The unregulated volume frequency curve will be the direct result of fitting a LP3 probability distribution to each duration.

In **Figure E- 3**, the 1997 x 1.0 flood routing resulted in the same regulated peak flow as the 1997 x 0.8. This is acceptable, especially if the reservoir is attempting to release objective flow or downstream channel capacity. There are instances where a larger scale factor results in a decrease in flow. This can be an anomaly caused by the reservoir operation rules. If the 1997 x 1.0 resulted in a lower peak outflow, the point can be removed and a line drawn from the 0.8 to the 1.2 scale factor values. This is called the “enveloping method” of producing a monotonic (never decreasing) transform function.

With an unregulated to regulated transform defined for each event, an event-specific regulated frequency curve is obtained by plotting the (log10) regulated peak flow values against probability of the volumes (based on the critical duration of each routed flood). The result is a shape-specific (conditional) regulated frequency curve for each event pattern. An example of a pattern specific regulated curve is shown in blue in **Figure E- 4**, with part of the curve that was non-monotonic removed (red).

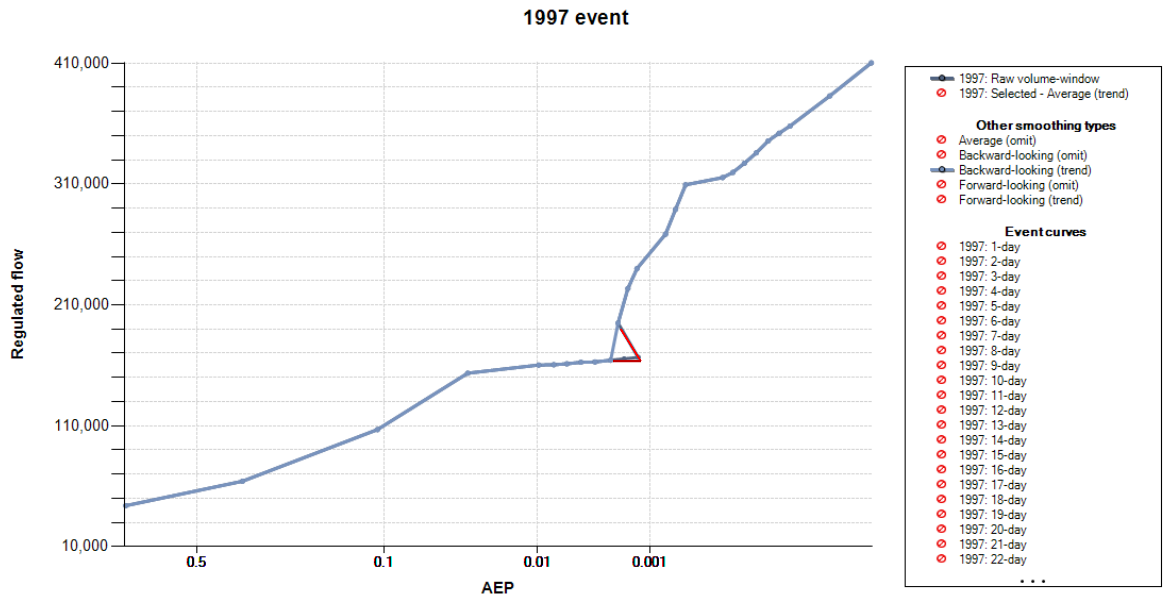


Figure E- 4: Example of Event-specific Regulated Frequency Curve (Blue)
Note: Non-monotonic points in curve removed (red)

To re-iterate, computation of simplified event-specific frequency curves such as is shown in **Figure E- 4** for each ROS, is a useful metric to compare and screen ROS sets.

E 5 Procedure to Compute Final Adopted Curves

To compute the final adopted curves, such as the one that will go in the Water Control Manual, the process of determining critical duration is performed for every one of the scaled and routed flood events. The process is a bit more complicated and the following steps are involved.

Step 1: Route various scaled versions of each event pattern flood through a reservoir model.

Step 2: Assess the critical duration for each scaled and routed flood.

Step 3: For each scaled event, assign a probability to the peak regulated flow based on the unregulated volume that was chosen as "critical". For each pattern, this results in a series of peak flow versus probability pairs.

Step 4: Create candidate unregulated to regulated transforms. A series of "candidate" unregulated to regulated transforms", much like the ones shown in **Figure E- 5** are generated, each one based on a different critical duration. **Figure E- 5** below, shows the candidate transforms (based on 1- to 30-day critical durations) for the 1986 pattern.

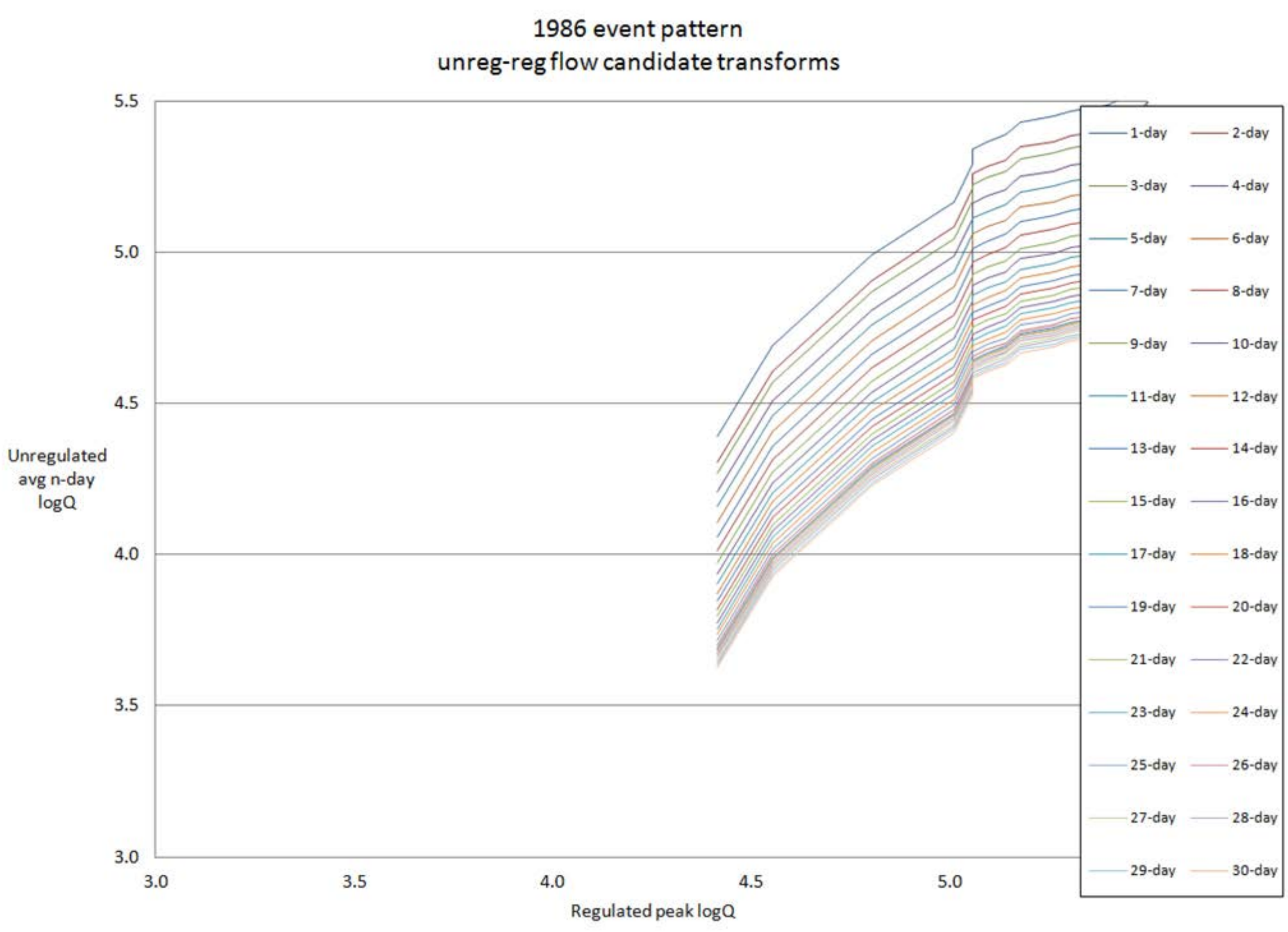


Figure E- 5: Candidate Transforms for the 1986 Event Pattern

Step 5: Convert each candidate transform into a candidate regulated flow frequency curve.

Step 6: Based on the results of Step 3, each scaled event falls on one of the candidate flow frequency curves (peak flow versus probability). A conditional event-based flow frequency curve is then produced as shown in **Figure E- 6**.

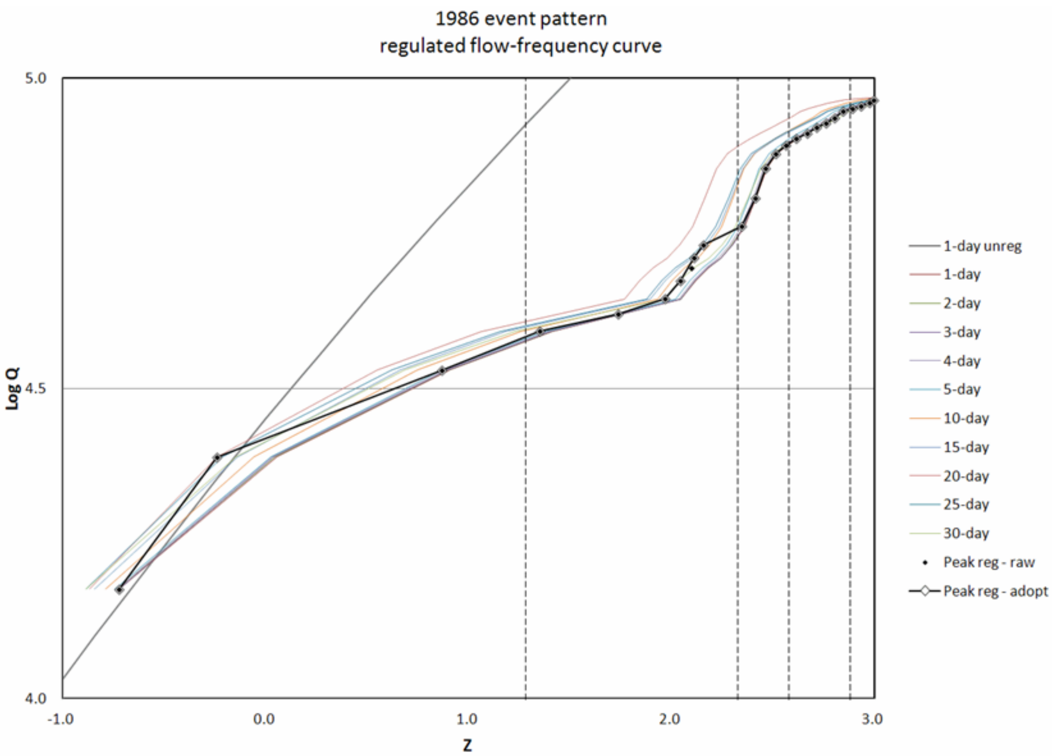


Figure E- 6: Candidate Flow Frequency Curves and Adopted Event Curve
Note: X-axis is equal to z (a surrogate symbol for probability). Based on the critical duration assigned to each scale factor, the probability of the peak flow can be determined, thus generating a conditional event-based frequency curve for the 1986 pattern.

Step 7: Using the process outlined in Step 6, create a conditional frequency curve for each event pattern (1955, 1964, 1986, and 1997).

Step 8: Combine and weight the conditional frequency curves to determine the final adopted regulated peak flow frequency curve. The combining and weighting process is described in the next section.

E 6 Weight and Combine Conditional Curves

Since there is a desire to produce one regulated flow frequency curve at a given location, the event-specific regulated frequency curves must somehow be combined to obtain a single regulated frequency curve. In the general case, if all patterns capable of producing a regulated flow value of interest are accounted for, and the relative likelihoods of these events producing that regulated flow are known, then the total probability theorem could be applied to compute the probability associated with the regulated flow value. This application of the total probability theorem can be written as:

$$P(Q) = P(Q|E1)P(E1) + P(Q|E2)*P(E2) + \ldots + P(Q|En)P(En)$$

where: Q = regulated flow value,
E1 = Event pattern #1,
E2 = Event pattern #2,

$P(E1)$ = Probability of event pattern 1 producing a large regulated flow,
$P(E2)$ = Probability of event pattern 2 producing a large regulated flow,
$P(Q|E1)$ = Probability of regulated flow Q being exceeded given that an event of pattern #1 has occurred,
$P(Q|E2)$ = Probability of regulated flow Q being exceeded given that an event of pattern #2 has occurred, and
n = number of event patterns in sample.

For the Water Control Manual update, four historical events were selected for scaling over the probability range of interest. Assumptions inherent in selecting these events which will define the regulated frequency curve are:

1. Patterns of large events that have occurred are representative of large events that will occur in the future.
2. Selection of a few (three to four) events for scaling is adequate to represent the spectrum of patterns which can occur.

The relative likelihood P(E) of (scaled) event E producing a large regulated flow can be estimated from the inverse of the exceedence probability of the historical (unscaled) event p(Eo). The exceedence probability p(Eo) of an event, historical or scaled, is obtained from the unregulated volume frequency curve corresponding to the critical duration of the event. If the three largest historical events had exceedence probabilities [p(E1o), p(E2o), p(E3o)], the inverses are first computed, then normalized so that the resulting relative likelihoods [P(E1), P(E2), P(E3)] sum to 1. If p(E1o) = 0.010, p(E2o)=0.015, p(E3o)=0.02, then 1/ p(E1o) = 100, 1/ p(E2o) = 66.7, 1/ p(E3o) = 50. Normalizing to the sum (216.7) gives. P(E1) = 100/216.7=0.46, P(E2) = 66.7/216.7=0.31, P(E3) = 50/216.7 = 0.23. The total probability theorem is then applied to give the probability associated with regulated flow Q:

$$\mathbf{P(Q) = P(Q|E1)\cdot P(E1) + P(Q|E2)\cdot P(E2) + P(Q|E3)\cdot P(E3)}$$
$$= P(Q|E1)\cdot 0.46 + P(Q|E2)\cdot 0.31 + P(Q|E3)\cdot 0.23$$

The event-specific probabilities [P(Q|E1), P(Q|E2), P(Q|E3)] corresponding to regulated flow Q are taken directly from the event-specific regulated frequency curves.

With the selected approach, a single continuous regulated frequency curve is defined only for the probability range for which all event-specific regulated frequency curves are defined. Care must be taken to consider the maximum and minimum regulated peak flow corresponding to these curves. This is because the upper-end of the combined curve will be limited by the probability corresponding to the minimum upper end regulated flow of the event-specific curves. Similarly, the lower-end of the combined curve will be limited by the probability corresponding to the maximum lower end regulated flow of the event-specific curves. As a result, care was taken in specifying the range of scaled events to be simulated to ensure that the resulting combined regulated frequency curve spans the probability range of interest. An example of a combined peak flow frequency curve is shown in **Figure E- 7**.

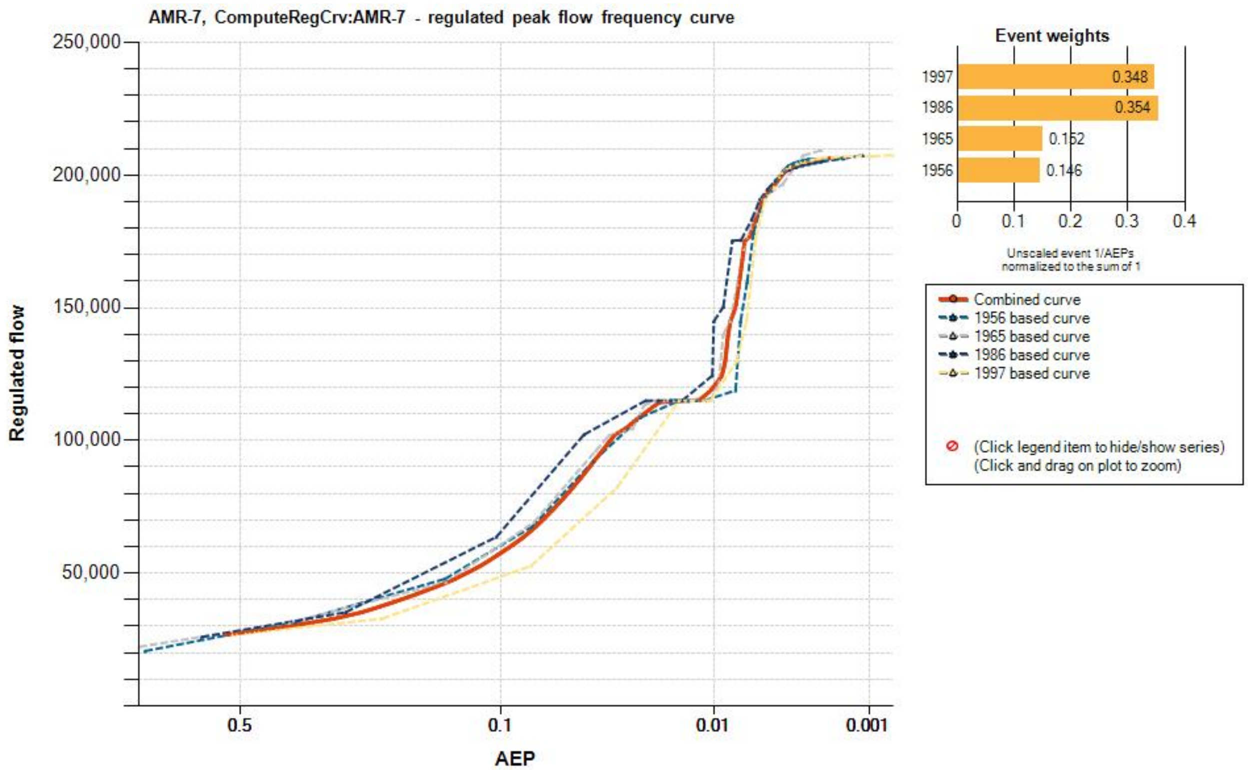


Figure E- 7: Example of Combining Event-specific Curves
Note: Adopted curve show in dark orange. Event weights shown in the upper right corner

Appendix F – Development of Emergency Spillway Release Diagram

This appendix documents development of the Emergency Spillway Release Diagram (ESRD), reflecting additional release capacity of the JFP auxiliary spillway. This ESRD is part of the Alternative 1 and Alternative 2 operations.

Reference

Engineering Manual EM 1110-2-3600, "Management of Water Control Systems," Department of the Army, U.S. Army Corps of Engineers, November 1987.

Introduction

The Folsom Dam Emergency Spillway Release Diagram (ESRD) must be updated to reflect additional release capacity provided by the JFP auxiliary spillway. The updated ESRD will be included in the updated Water Control Manual (WCM). The Folsom Dam ESRD is designed to ensure 3 feet of freeboard to the top of dam will be maintained during routing of the probable maximum flood (PMF).

The process for ESRD development is defined in EM 1110-2-3600. This process is implemented in the following steps, which are described in subsequent sections:

1. Define induced surcharge envelope (ISE).
2. Estimate recession constant (Ts) and construct inflow curves.
3. Test ESRD by routing PMF and other events.

Summary

The proposed ESRD is shown in Figure F-2, and as a plate for inclusion in the WCM in Figure F-3. Simulation of the all-season PMF event, both with and without antecedent events, resulted in top of dam freeboard of 3.2 feet (Figure F-3). This satisfies the established Folsom top of dam criterion of 3 feet freeboard. In addition to events listed in Table F-2, additional events were routed and are reported in the body of the Engineering Report. Those included seasonal versions of the PMF, which also were successfully routed with greater than 3 feet freeboard.

Two gate inoperability scenarios were considered to understand sensitivity of PMF routings to gate failure. These scenarios were:

1. PMF event with one non-functioning gate at main dam
2. PMF event with one non-functioning gate at JFP auxiliary spillway

In both scenarios, the non-functioning gate was modeled as zero release through the gate for the duration of the event. For these two scenarios, the resulting top of dam freeboard was 1.3 feet and 1.7 feet, respectively. These simulations were executed again, using the same ESRD, but with releases allowed to exceed the maximum controlled release envelope (curve D in Figure F-

2) up to the maximum uncontrolled release curve (green curves in Figure F-2) once the pool elevation exceeded 474.3 feet NAVD88 (472.0 feet NGVD29). This reflects allowing the lifting of functional main dam gates out of the water to achieve the ESRD-specified release. With this operation, the top of dam freeboard was found to be 3.1 and 3.2 feet, respectively for scenarios 1 and 2. Based on this result, should a single gate become inoperative, and releases above 600 kcfs are required by the ESRD, the operator should be prepared to lift the main dam gates out of the water to achieve the specified release.

Induced Surcharge Envelope

[The reader is referred to Figure F-2 to support the following discussion of key pool elevations, controlled release capacity, and ISE development.]

The ISE defines the minimum required release as a function of pool elevation to ensure passage of the PMF event. Ideally, the combined spillway release capacity at the top of the surcharge pool will be equal to or greater than the PMF peak inflow. The ISE for this scenario would typically be curved in shape, reflecting the trace of pool elevation versus total release as main dam tainter gates are lifted to prevent gate overtopping (curve A). The curved ISE would intersect the maximum controlled release capacity curve at the top of the surcharge pool (curve D). The resulting ESRD would allow increased used of surcharge storage and require increased releases as the magnitude of events increase, up to and including the PMF event.

At Folsom Dam the all-season PMF peak inflow is 905 kcfs. The combined controlled release capacity of all spillways (including the JPF auxiliary spillway) at the top of the surcharge pool (3 feet below top of dam) is 846 kcfs. Successful routing of the PMF event will require an ESRD that does not allow use of storage high in the surcharge pool unless the maximum controlled release is already being made. This ensures that space remains available in the surcharge pool for routing the PMF peak inflow within the surcharge pool. This operation is enforced by locating the ISE lower in the surcharge pool, as indicated in Figure F-2.

A consequence of the Folsom Dam ESRD will be that rare events, but smaller than the PMF event (ACE=1/1000 event for example), will be routed without using physically available surcharge space. This is necessary if the ESRD is to allow an operator to route the PMF event with knowledge limited to the current inflow and pool elevation. In this “lights out” situation, there will be no indication as to whether the current event is a 1/1000, or PMF event.

Simulations with preliminary ESRDs revealed the surcharge pool elevation at which the maximum controlled release must be made to pass the PMF event is 475.3 feet NAVD88 (473.0 feet NGVD29). At this elevation the maximum controlled release is 733 kcfs. This release value is achieved with all six (submerged) auxiliary spillway gates 100 percent open, and five service and three emergency gates on the main dam at hoist chain travel (HCT) 38 feet. During the PMF event, as inflows increase above 733 kcfs and the pool elevation increases above 475.3 feet NAVD88, the eight gates on the main dam are further raised to maintain the maximum controlled release condition. Passing the PMF event with 3 feet freeboard requires maintaining maximum controlled release up to pool elevation 479.8 feet NAVD88 (477.5 feet NGVD29). At this peak pool elevation, the maximum controlled release of 846 kcfs is achieved with eight main dam gates open to HCT 42. The change in storage between pool elevations of 475.3 feet and 479.8 feet NAVD88 is 52.7 KAF.

Along the ISE, the specified release is equal to inflow. An event that is routed such that plotted elevation-release pairs follow the top of the ISE reflects accurate inflow estimates and gate adjustments to release current inflow. If inflows are increasing, the pool elevation will increase until the next time inflow is estimated and the gates are opened further to again release inflow. The slope of the ISE should therefore reflect:

- the maximum rate of increase in inflow
- the maximum rate of increase of outflow
- how often inflow will be estimated and gate settings adjusted (.t)

Constraints on maximum rate of increase are listed in Error! Reference source not found.. Simulations reflected water released only through the five service and three emergency tainter gates at the main dam, and the six submerged tainter gates at the JFP auxiliary spillway. Releases through lower river outlets and power house were set to zero.

Table F-1: Maximum Rate of Release Increase

Pertinent Diagram	Releases between:	Will not be *increased* by more than this amount
WCD	8 kcfs to 30 kcfs	5 kcfs per 2 hours
WCD	30 kcfs to 115 kcfs	30 kcfs per 2 hours
ESRD	30 kcfs to 160 kcfs	30 kcfs per 2 hours
ESRD	160 kcfs to 360 kcfs	100 kcfs per hour
ESRD	360 kcfs and greater	200 kcfs per hour

At Folsom, the maximum rate of increase of inflow which could be expected to occur is 47 kcfs per hour. This has not been observed in any historical events, but is a characteristic of the PMF inflow hydrograph. For the release range 160 kcfs and greater, the maximum rate of increase in releases is 100 kcfs per hour. Therefore, for this release range, releases are not restricted by maximum rate of increase, and the pool will rise as a result of the time between updating gate settings alone. For the condition in which releases are adjusted once per DT hours to match current inflow, and the current inflow increases steadily while outflow is held constant, the ratio of change in storage to change in outflow is given by Equation 1. This is the required slope of the ISE in terms of storage to accommodate the maximum rate of inflow increase.

Equation 1 ISE slope (releases greater than 160 kcfs) = DT / 2

where DT = Time (hrs) between gate changes.

Reclamation has stated that gate changes will be made once per 30 minutes for releases 160 kcfs and greater (Reclamation, 2017). While the Corps supports the 30-minute update interval, it is considered overly optimistic for the purpose of ESRD development. The frequency at which pool elevation will be measured, inflow estimated, and gate settings adjusted, is therefore assumed to be 1.5 hours. This value of DT is intended to keep the operation "on the diagram" (pool elevation does not exceed ISE) under stressful or less than ideal operational conditions. The ISE slope can be written in units of time or change in storage (ac-ft) per change in flow (cfs), as shown in Equation 2.

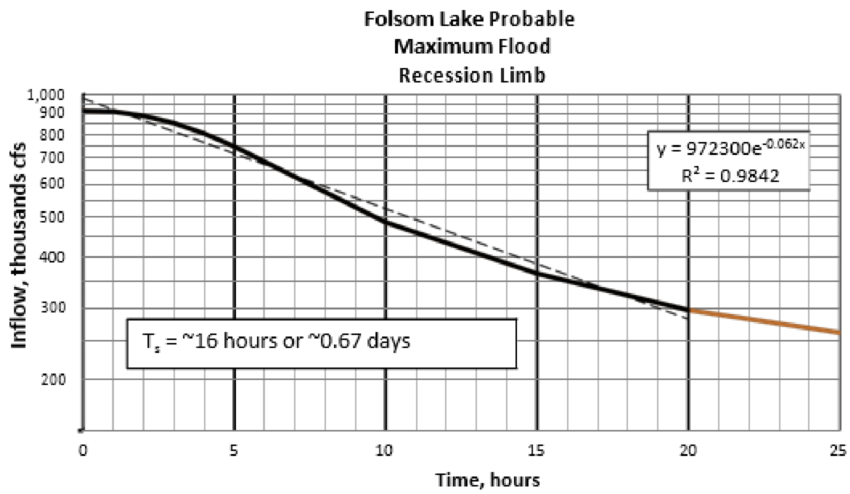
Equation 2 ISE slope = DT / 2 = 1.5 hrs / 2 = 0.75 hrs = 0.062 ac-ft/cfs = 5.4 $x10^{-6}$ ft/cfs

The final term in Equation 2 is obtained by substituting the ratio of elevation change to storage change, which is $8.7x10^{-5}$ feet/acre-foot in elevation range of interest. With a 1.5-hour update interval, should the maximum rate of increase of inflow of 47 kcfs per hour occur, the pool would rise by 0.4 feet/hour. The ISE for releases between 160 kcfs and 733 kcfs is therefore defined by the line starting at the point having ESRD coordinates (733 kcfs release, 475.3 feet NAVD88 elevation), and extended back to release 160 kcfs with slope 5.4 x 10^{-6} feet/cfs. This computation yields an ISE elevation of 475.3 feet NAVD88– 3.1 feet = 472.2 feet NAVD88. This value was rounded up to 472.3 feet NAVD88 to correspond with the pool elevation above which the emergency gates must be opened to maintain 1 foot top-of-gate freeboard. As a result, the adopted ISE slope was 5.2 x 10^{-6} feet/cfs. The resulting ISE for release range 160 kcfs to 733 kcfs is therefore a straight line spanning a change in 3.0 feet pool elevation or 33 KAF storage.

Releases in the range of 115 kcfs to 160 kcfs are restricted to 30 kcfs per 2 hours. This reflects an operation in which gate changes will be implemented once per 2 hours (verbal communication from Reclamation). For this case, in which the maximum rate of increase in release cannot keep pace with the possible maximum rate of increase in inflow, there is no simple analytical solution for defining the ISE envelope. The slope must be steeper in this release range than in the 160 kcfs and greater range, effectively pushing inflow curves down to require releases in the 115 kcfs to 160 kcfs range at lower pool elevations. The approach taken was simply to implement a straight line ISE to the top of flood pool (466.0 NGVD29, 468.3 feet NAVD88) at release 115 kcfs. Simulations of a wide range of event magnitudes temporal distributions were performed to ensure that the ISE was not exceeded.

Recession Constant (Ts)

EM 1110-2-3600 defines Ts as the time for inflow Q to recede to a value of Q/2.718. Ts was estimated from the recession limb of the all-season PMF inflow hydrograph. A linear computation of Ts is shown in Error! Reference source not found., and an average Ts value of 0.67 days, or 16 hours, was computed. Ts was also estimated as 15, 13, and 19 hours for the historical events of water years 1956, 1986, and 1997. Based on this assessment, the value of 16 hours was considered reasonable and was adopted.



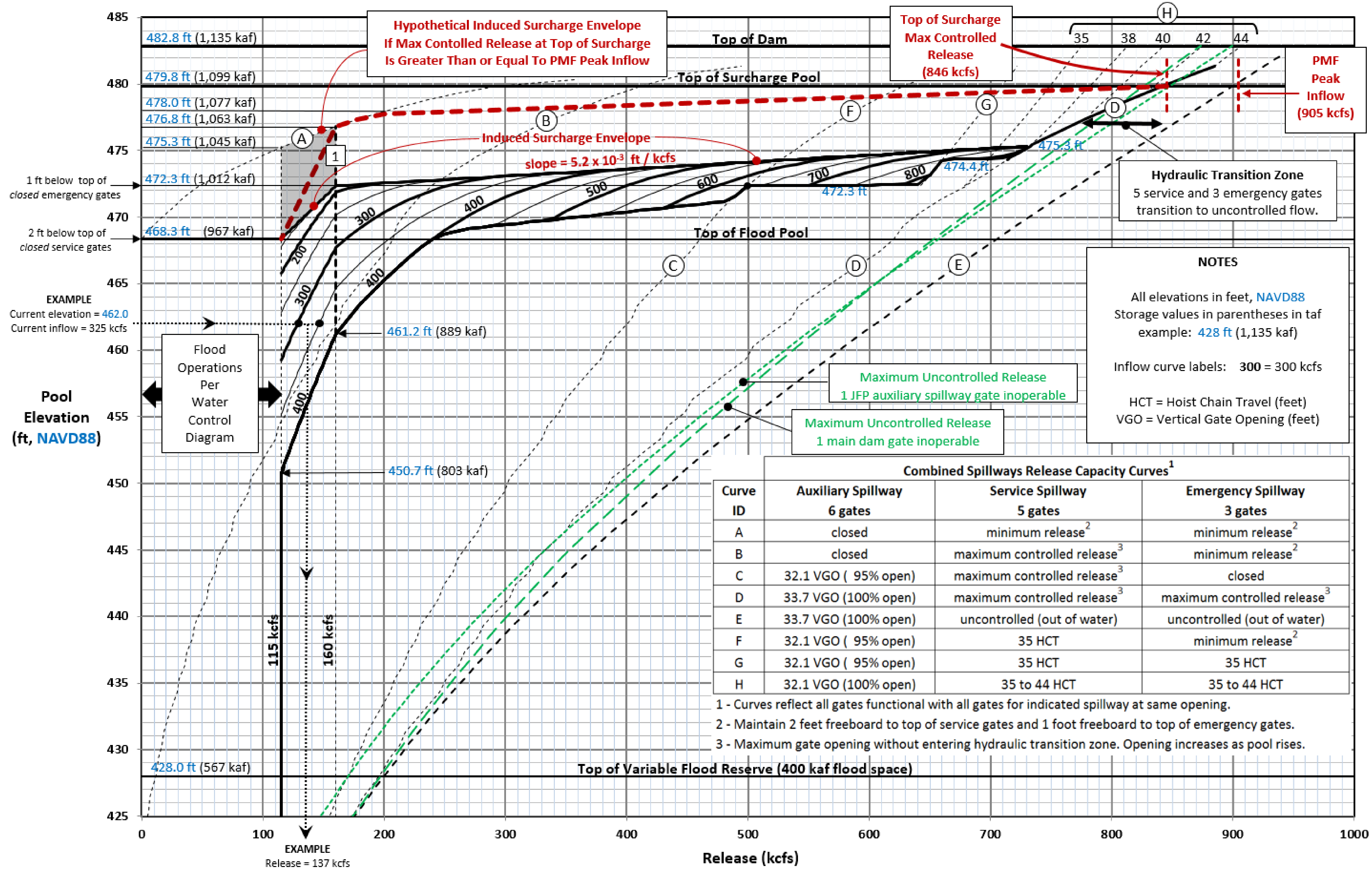
Q_1 (cfs)	$Q_2 = Q_1/2.718$ (cfs)	T_1 (hr)	T_2 (hr)	$T_2 - T_1$ (hr)	T_s (days)
905770	333249	50	64.9	16.3	0.68
902630	332093	51	65.9	16.3	0.68
881560	324341	52	66.8	16.2	0.68
845950	311240	53	67.8	16.2	0.67
845950	293712	54	68.7	16.1	0.67
798310	272800	55	69.6	16.0	0.67

Figure F-1: PMF Ts Estimate

With the ISE defined and Ts specified, the resulting family of inflow curves were computed following the EM 1110-2-3600 procedure. The resulting ESRD is shown in Figure F-2. Release capacity curves for indicated spillway gate configurations were added to provide understanding of operational options for achieving required releases. The lower portion of the diagram was truncated to remove curves not needed to support event routings. The diagram, as it will appear in the Water Control Manual, is provided in Figure F-3.

Informed Use of Surcharge

Two features were added to the diagram to recognize the step increase in risk to lives that occurs when releases exceed 160 kcfs (downstream channel capacity). A shaded area was included on the diagram to indicate surcharge which may be used only when the determination can be made with high-confidence that the event is in final recession. A dashed vertical line corresponding to 160 kcfs release is also included and should not be exceeded without consultation between Corps and Reclamation while communication lines are functional. Both of these features are described in item 4 of the Operating Instructions portion of the plate..



Combined Spillways Release Capacity Curves[1]

Curve ID	Auxiliary Spillway 6 gates	Service Spillway 5 gates	Emergency Spillway 3 gates
A	closed	minimum release[2]	minimum release[2]
B	closed	maximum controlled release[3]	minimum release[2]
C	32.1 VGO (95% open)	maximum controlled release[3]	closed
D	33.7 VGO (100% open)	maximum controlled release[3]	maximum controlled release[3]
E	33.7 VGO (100% open)	uncontrolled (out of water)	uncontrolled (out of water)
F	32.1 VGO (95% open)	35 HCT	minimum release[2]
G	32.1 VGO (95% open)	35 HCT	35 HCT
H	32.1 VGO (100% open)	35 to 44 HCT	35 to 44 HCT

1 - Curves reflect all gates functional with all gates for indicated spillway at same opening.
2 - Maintain 2 feet freeboard to top of service gates and 1 foot freeboard to top of emergency gates.
3 - Maximum gate opening without entering hydraulic transition zone. Opening increases as pool rises.

Figure F-2: ESRD with additional labeled curves

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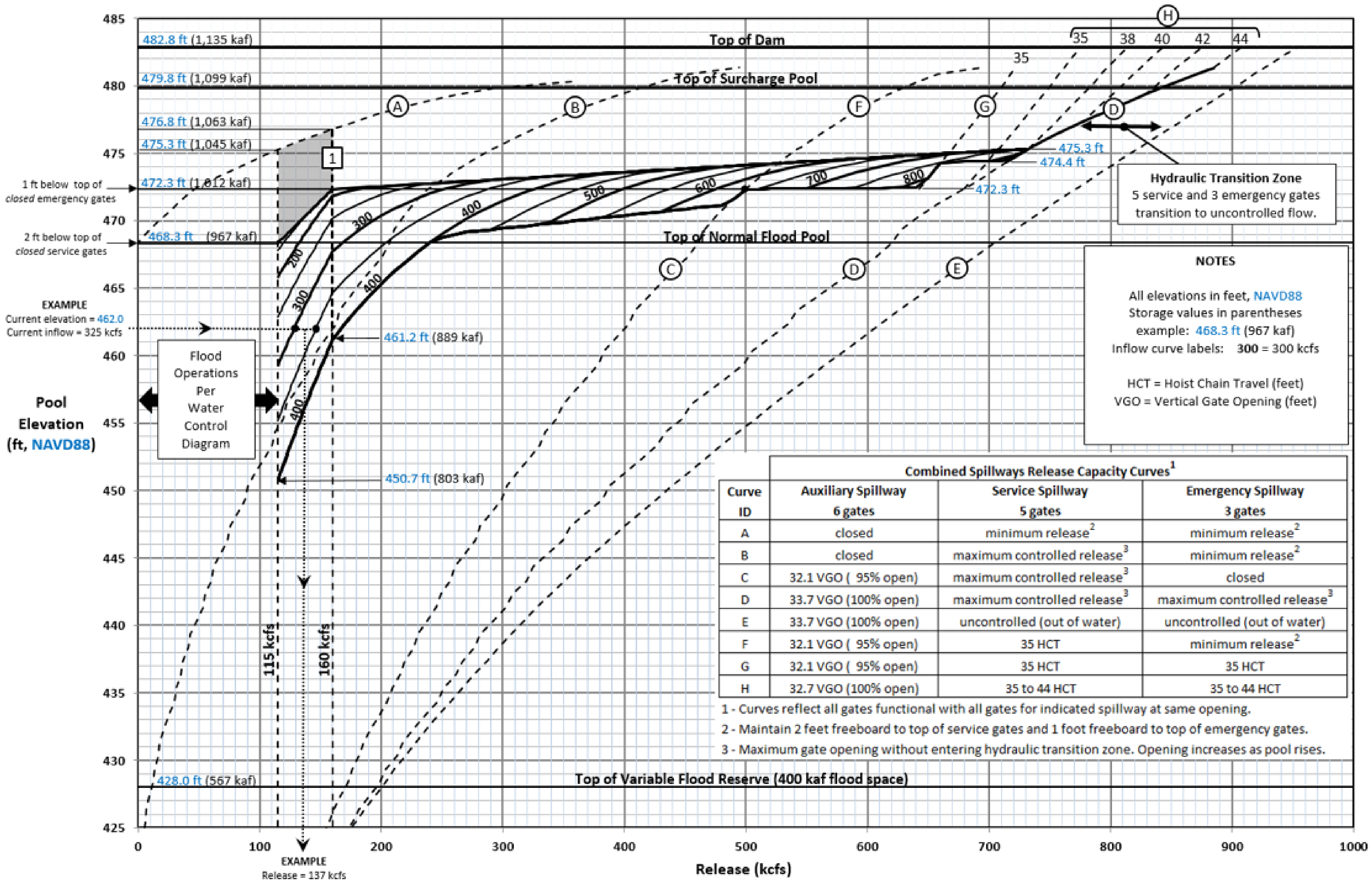
EMERGENCY SPILLWAY RELEASE DIAGRAM



Curve ID	Combined Spillways Release Capacity Curves[1] Auxiliary Spillway 6 gates	Service Spillway 5 gates	Emergency Spillway 3 gates
A	closed	minimum release[2]	minimum release[2]
B	closed	maximum controlled release[3]	minimum release[2]
C	32.1 VGO (95% open)	maximum controlled release[3]	closed
D	33.7 VGO (100% open)	maximum controlled release[3]	maximum controlled release[3]
E	33.7 VGO (100% open)	uncontrolled (out of water)	uncontrolled (out of water)
F	32.1 VGO (95% open)	35 HCT	minimum release[2]
G	32.1 VGO (95% open)	35 HCT	35 HCT
H	32.7 VGO (100% open)	35 to 44 HCT	35 to 44 HCT

1 - Curves reflect all gates functional with all gates for indicated spillway at same opening.
2 - Maintain 2 feet freeboard to top of service gates and 1 foot freeboard to top of emergency gates.
3 - Maximum gate opening without entering hydraulic transition zone. Opening increases as pool rises.

OPERATING INSTRUCTIONS

The Emergency Spillway Release Diagram (ESRD) is used to determine if releases greater than 115 kcfs are required, and if so, to specify the minimum required release. The steps below should be initiated whenever water is stored above elevation 450.7 ft NAVD88.

1) Estimate current reservoir inflow in kcfs.

   Estimate inflow, based on change in storage and release made over previous 2 hours, or greater period if necessary to obtain reliable measurements.

2) Enter the ESRD with current inflow (in kcfs) and current pool elevation (ft, NAVD88), to compute the minimum required release value.

   EXAMPLE: For a current inflow estimate of 325 kcfs and current pool elevation of 462.0 ft NAVD88, the emergency release value is found as follows. The bounding inflow curves on the ESRD are the 300 kcfs and 325 kcfs. The points at which both curves intersect the horizontal line corresponding to elevation 462.0 ft are identified. On the horizontal line between these points, the location of the point corresponding to inflow 325 kcfs is estimated. For this point, the ESRD minimum release value of 137 kcfs is read on the X axis.

3) Once releases based on the ESRD are initiated, gate changes shall be made in accordance with the criteria found herein until the required outflow drops to 115 kcfs. Use the Water Control Diagram to determine release of 115 kcfs or less.

4) While communication systems are functional, Reclamation and Corps shall consult before releases greater than 160 kcfs are made. This is indicated by **line 1** on diagram. The **shaded area** indicates surcharge which may be used during final recession of the event.

SCHEDULE FOR EMERGENCY SPILLWAY RELEASES

POOL RISE/FALL	POOL ELEVATION	DS LEVEES	INDICATED RELEASE	ACTION
n/a	Less than 450.7	Intact	0 to 115 kcfs	Follow Water Control Diagram
Rising	450.7 – 476.8	Intact	115 kcfs to 160 kcfs	Increase outflows to indicated release at a rate of 30 kcfs per two hours. Notify local authorities that evacuation of areas adjacent to downstream levees should be initiated. Do not reduce outflow while pool is rising.
Rising	Greater than 461.2	Intact	Greater than 160 kcfs	Increase outflow to indicated release but not greater than 160 kcfs until 6 hours has elapsed since flows greater than 115 kcfs were initiated. Maximum rate of increase is 50 kcfs per 30 minutes (100 kcfs per hour) for releases ranging between 160 kcfs to 360 kcfs. Maximum rate of increase is 200 kcfs per hour for releases greater than 360 kcfs.
Falling	Greater than 450.7	Intact	The lesser of 125% of inflow or maximum event release	Make indicated release but do not reduce outflows below 115 kcfs until the reservoir pool has dropped below elevation 450.7.
Falling	Greater than 450.7	Inoperative	The lesser of 125% of inflow or maximum event release	Make indicated release but do not reduce outflows below 50 kcfs until the reservoir pool has dropped below elevation 450.7.

FOLSOM DAM AND LAKE
American River, California

EMERGENCY SPILLWAY RELEASE DIAGRAM

APPROVED ______________________________

APPROVED ______________________________

Effective Date ____________ File No. ____________

FINAL DRAFT 06/06/2017 PLATE A-10

Figure F-3: ESRD as Plate in Water Control Manual

Event Simulations

Event simulations were performed using HEC-ResSim with hourly computation time step. Table F-2 summarizes results of the PMF routings. In Table F-2, Alternatives 1 and 2 reflect the proposed ESRD. All Alternative 1 and 2 PMF events were successfully routed with at least 3.2 feet freeboard to top of dam. Hydrograph plots of the all-season PMF routings for Alternatives 1 and 2 are provided in Figures F-4 and F-5. Trace plots, overlaying the hourly elevation-release series onto the ESRD, are provided in Figure F-6 for Alternative 2. Trace plots show the elevation-release series up to the time of peak pool elevation and help visualize use of storage space as prescribed by the ESRD.

Additional events, including the Standard Project Flood, and ACE = 1/200 and 1/1000 events using six temporal patterns, were also simulated. These events were simulated to test the ESRD for events more frequent than the PMF. Results for these simulations are shown in Figures F-7 through F-9. In Figure F-10, one maximum release-maximum elevation data point for each scaled event is plotted. Events were scaled to ACE values ranging from 1/2 to 1/1000.

Table F-2: Summary of PMF Event Simulations

Flood operation (1)	Season (2)	Antecedent event (3)	Folsom Initial Storage (ac-ft) (4)	Folsom Initial Release (cfs) (5)	HEC-ResSim Simulation (6)	Peak Inflow (cfs) (7)	Peak Outflow (cfs) (8)	Peak Storage (ac-ft) (9)	Peak Pool Elevation (ft, ngvd29) (10)	Top of Dam Freeboard (ft) (11)
Existing Interim (no JFP) (E504)	All-season (winter)	Without	966,934	115,000	SYNT_PMF_SET	904,579	914,869	1,165,923	483.09	overtopped
		With	566,934	6,480	SYNT_PMF_SET	904,579	910,811	1,165,218	483.03	overtopped
Alternative 1 Credit-based (J602P)	All-season (winter)	Without	966,934	115,000	P-PMF-winter-wo	904,579	828,688	1,096,238	477.27	3.23
		With	497,254	3,004	P-PMF-winter-w	904,579	828,686	1,096,237	477.27	3.23
Alternative 2 Forecast-based (J602F)	All-season (winter)	Without	966,934	115,000	F-PMF-winter-wo	904,579	828,688	1,096,238	477.27	3.23
		With	566,934	3,004	F-PMF-winter-w	904,579	828,618	1,096,213	477.27	3.23
Seasonal Guide Curve Alternatives 1 and 2	March	Without	966,934	115,000	F-PMF-Mar-wo	904,579	828,655	1,096,223	477.27	3.23
		With	566,934	3,004	F-PMF-Mar-w	904,579	828,655	1,096,223	477.27	3.23
	April	Without	966,934	50,000	F-PMF-Apr-wo	696,258	691,465	1,044,056	472.81	7.69
		With	771,108	2,000	F-PMF-Apr-w	696,258	696,170	1,050,380	473.35	7.15
	May	Without	966,934	25,000	F-PMF-May-wo	534,505	529,041	1,034,325	471.96	8.54
		With	908,058	2,000	F-PMF-May-w	534,505	534,376	1,075,450	475.51	4.99
	June	Without	966,934	2,000	F-PMF-Jun-wo	388,836	385,797	1,024,709	471.13	9.37
		With	966,934	2,000	F-PMF-Jun-w	388,836	385,799	1,024,709	471.13	9.37
	October	Without	966,934	115,000	F-PMF-Oct-wo	702,080	701,959	1,044,413	472.84	7.66
		With	954,434	1,150	F-PMF-Oct-w	702,080	693,428	1,044,384	472.84	7.66
	November	Without	966,934	115,000	F-PMF-Nov-wo	826,826	776,775	1,070,246	475.06	5.44
		With	695,753	1,400	F-PMF-Nov-w	826,826	776,472	382,430	475.05	5.45

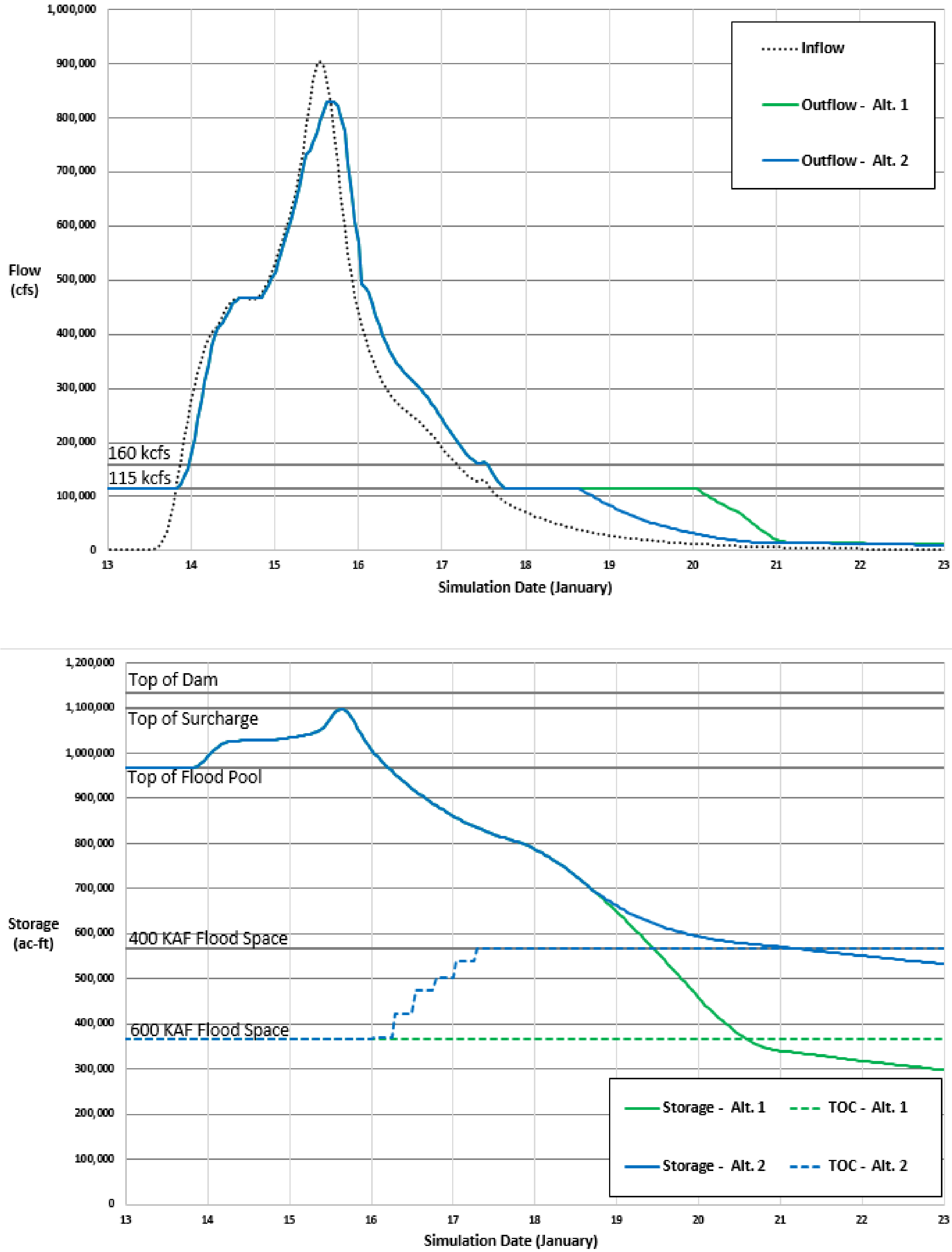


Figure F-4: Winter (All-Season) PMF Hydrographs

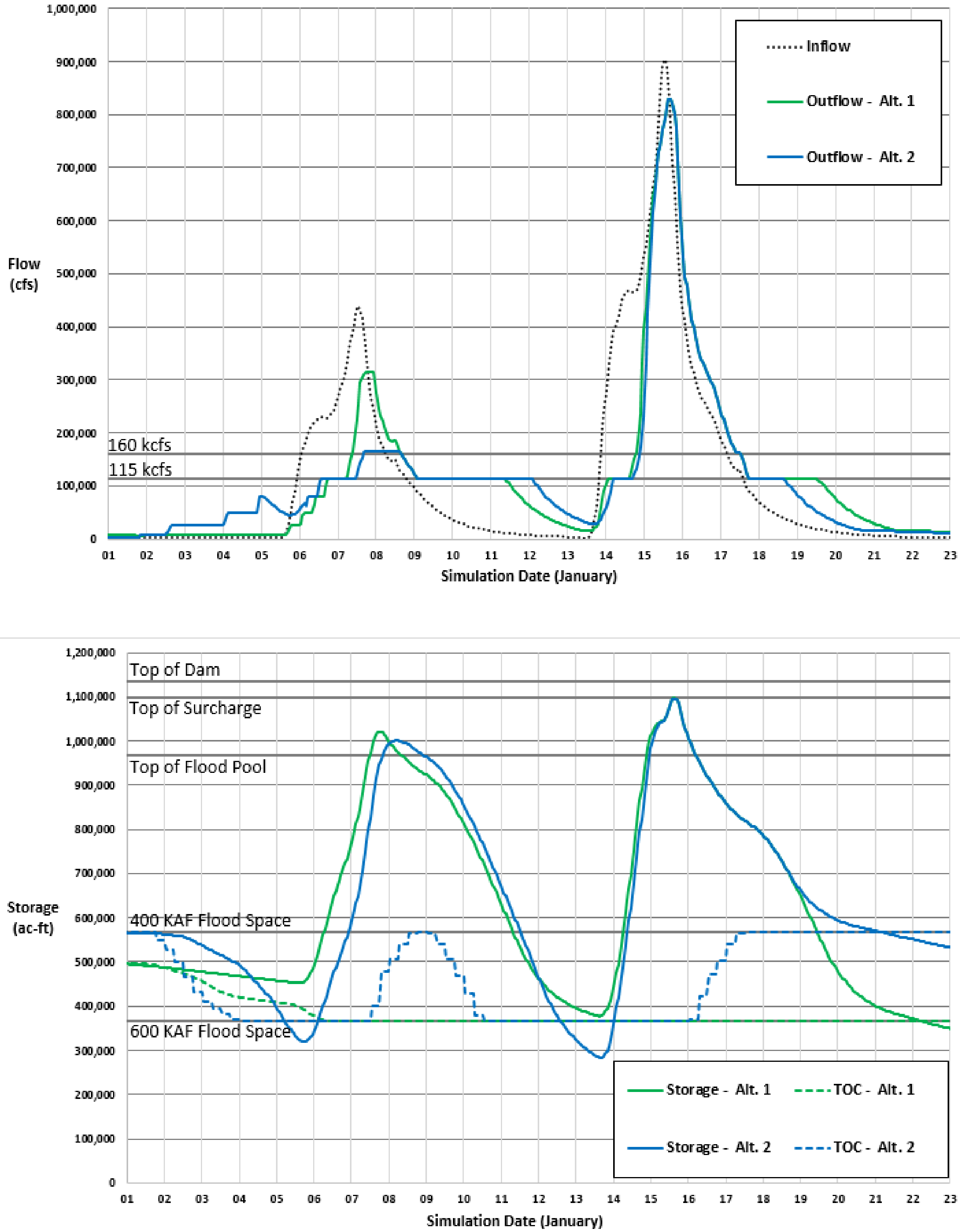


Figure F-5: Winter (All-Season) PMF Hydrographs – with Antecedent Event

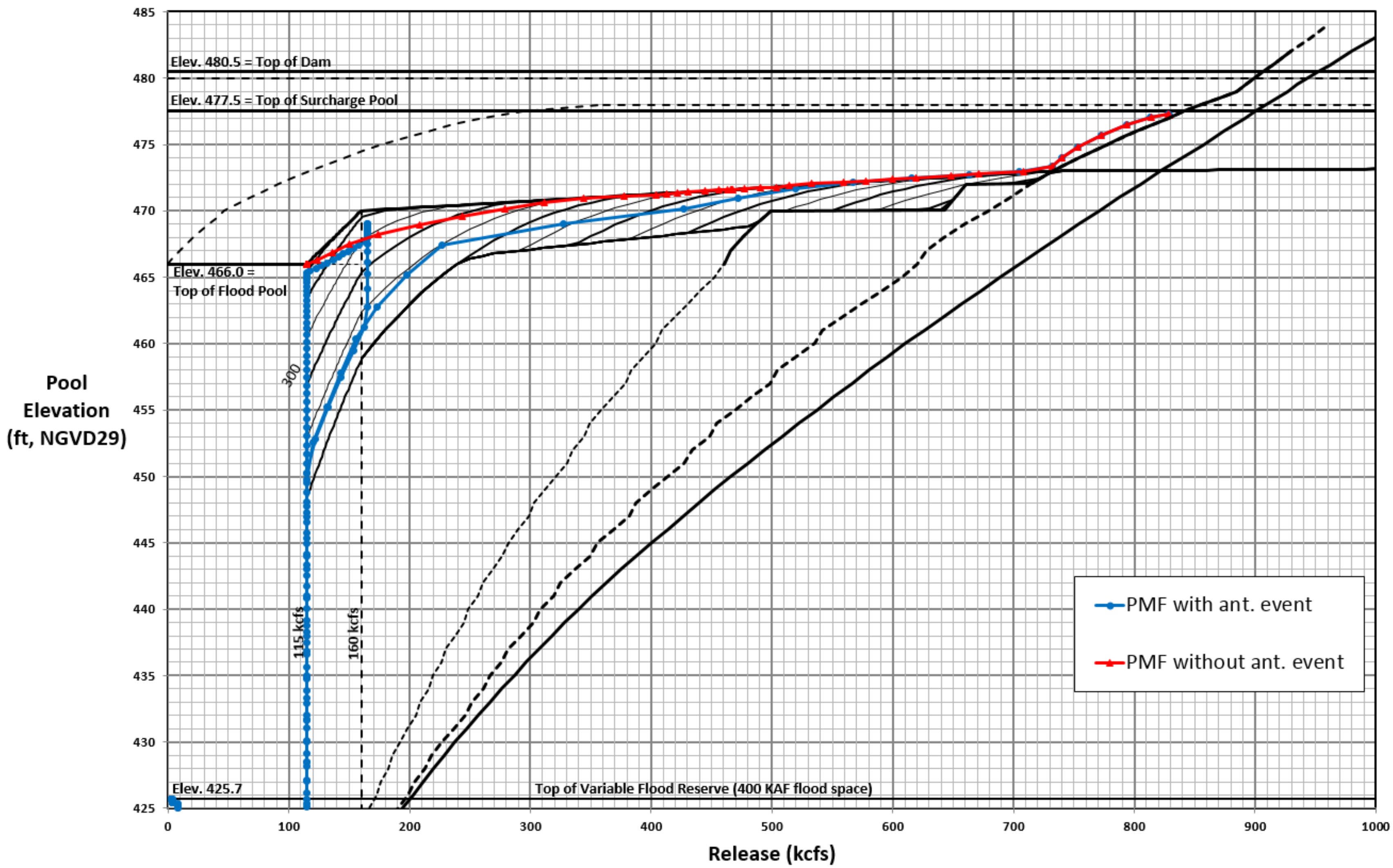


Figure F-6: Winter (All-Season) PMF Traces – Alternative 2

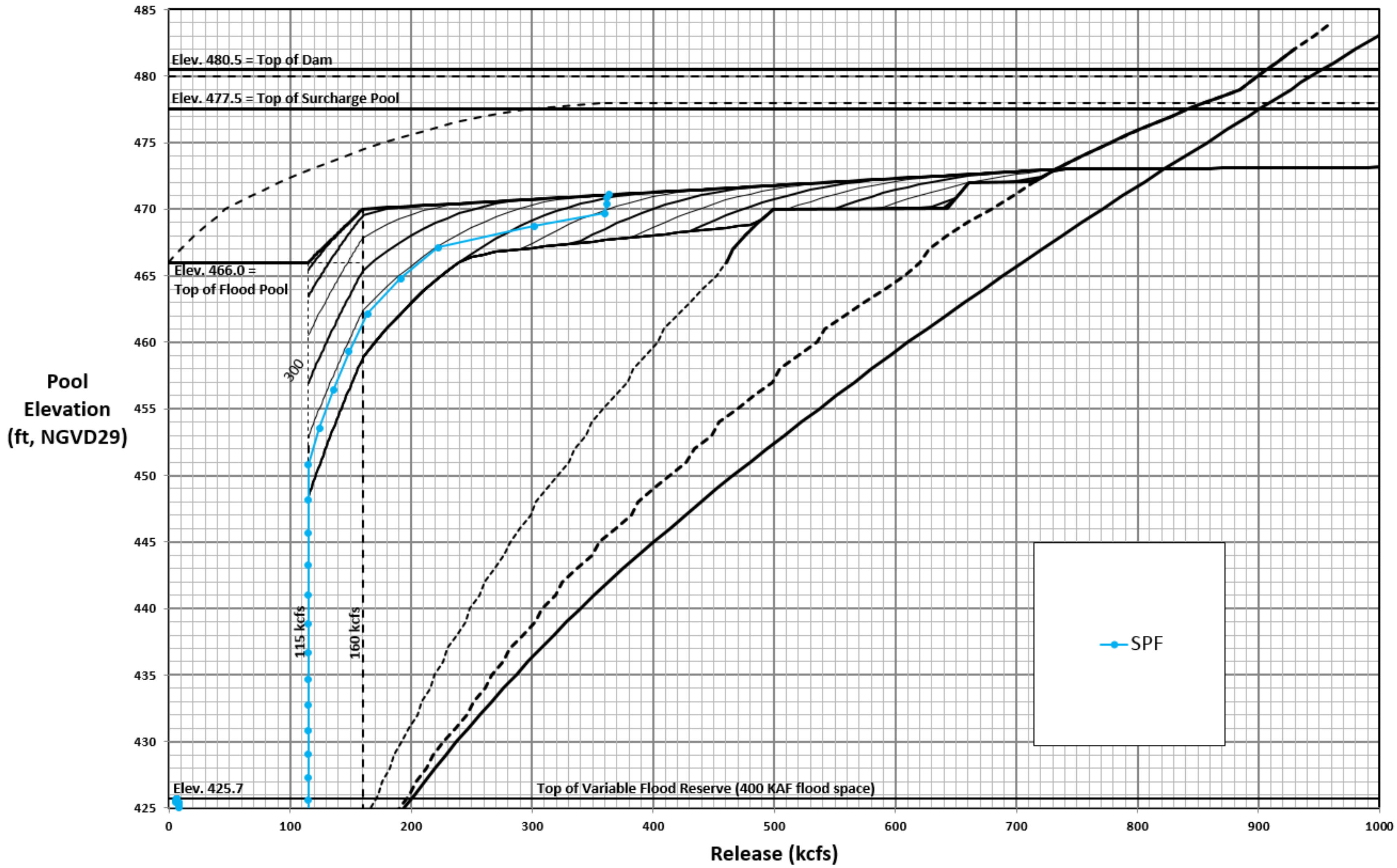


Figure F-7: SPF Event Trace – Alternative 2

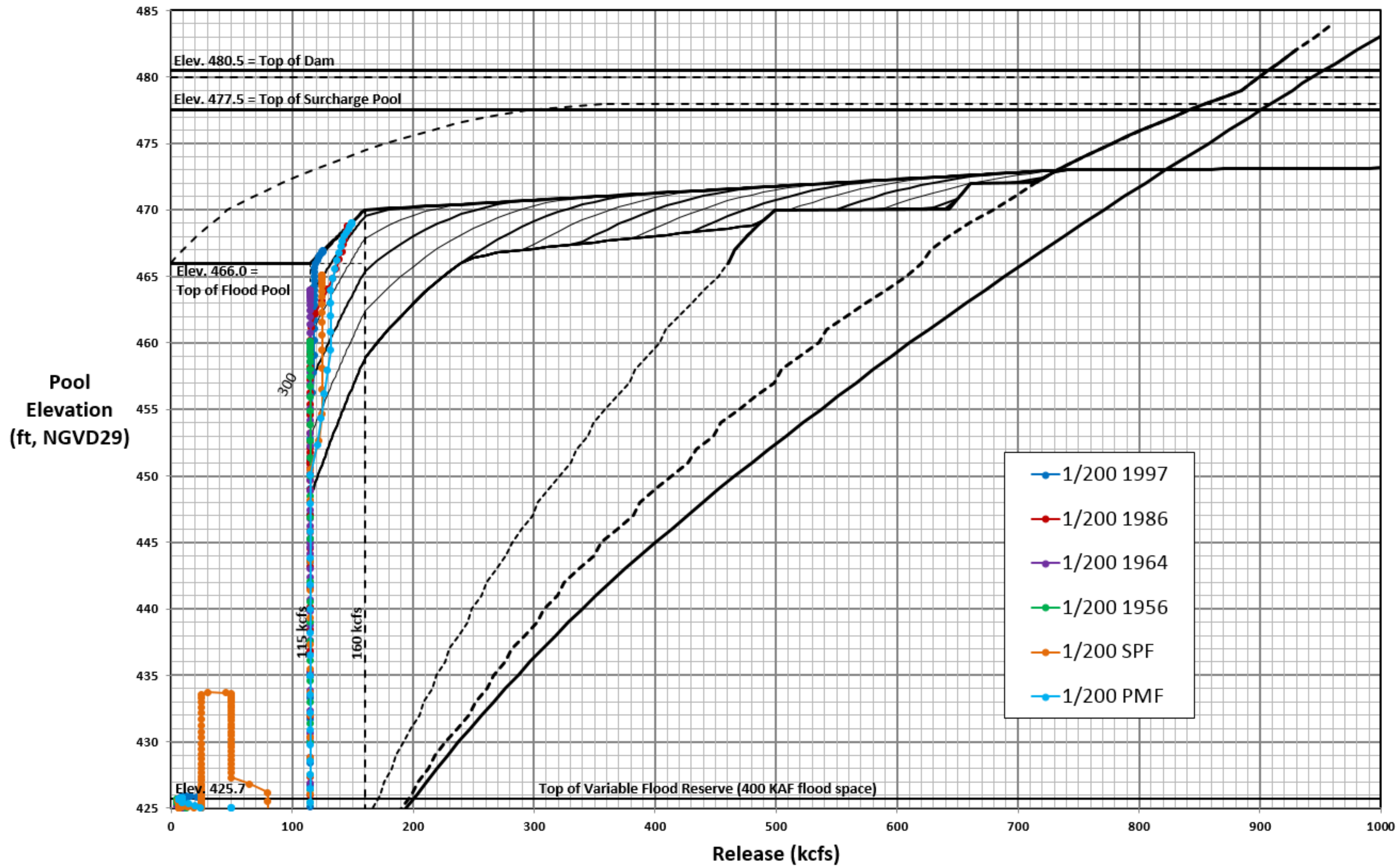


Figure F-8: 1/200 ACE Event Traces – Alternative 2

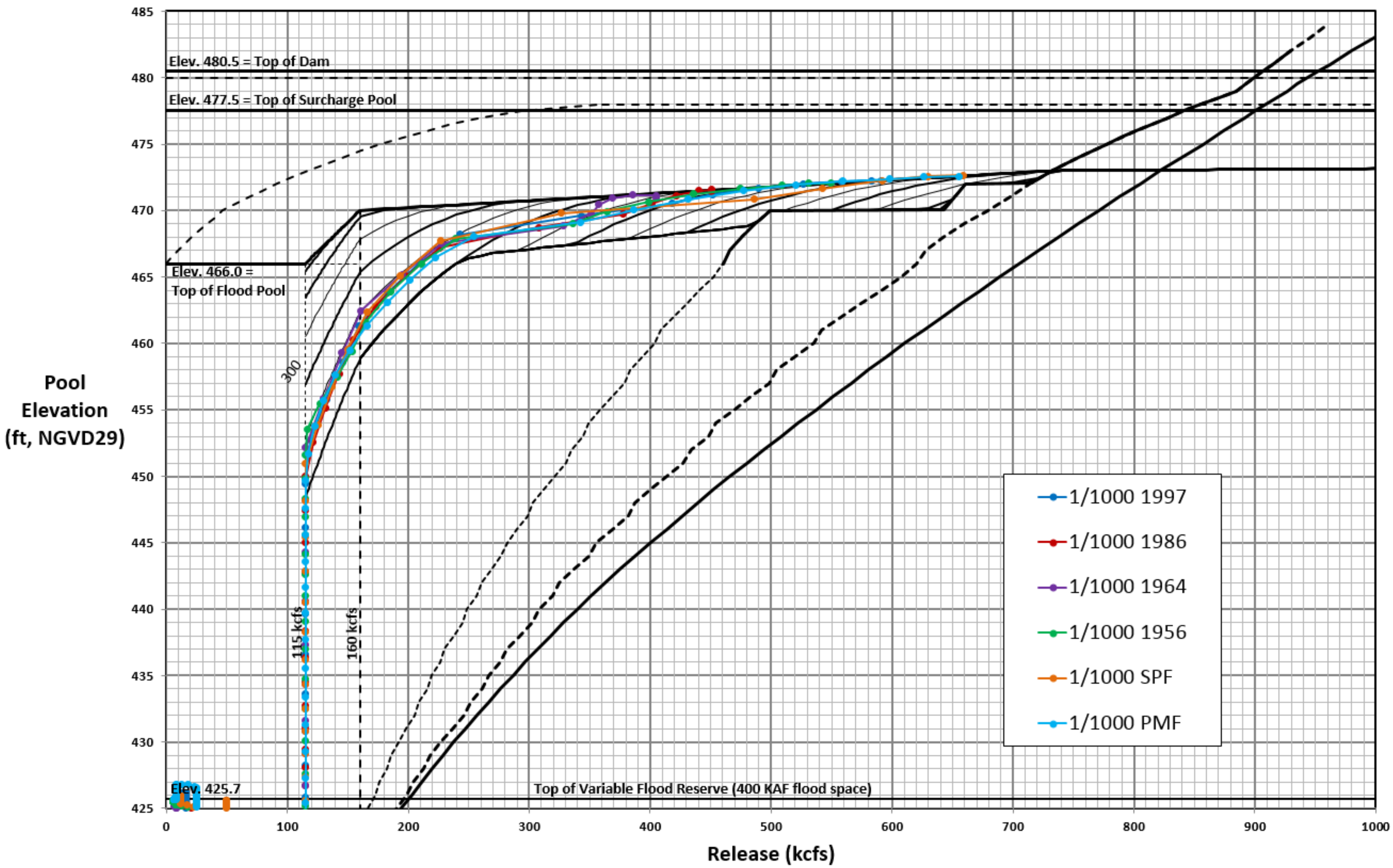


Figure F-9: 1/1000 ACE Event Traces – Alternative 2

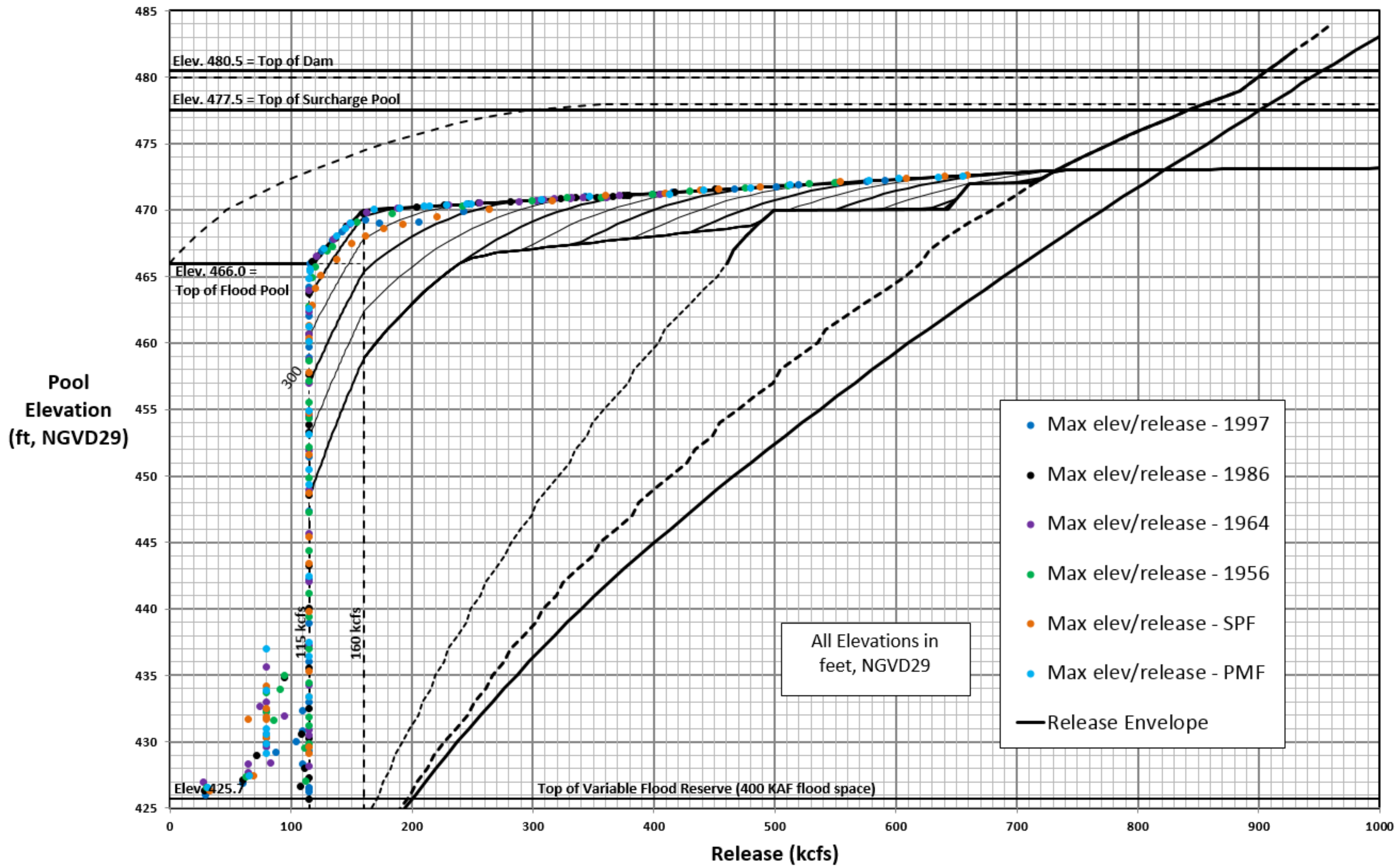


Figure F-10: 1/2 to 1/1000 ACE Events, Max Elevation-Max Release Event Pairs – Alternative 2

Appendix G – Selected Figures

This appendix contains figures that are referenced and discussed in various sections of the main report but which were moved to this appendix to make the report easier to read.

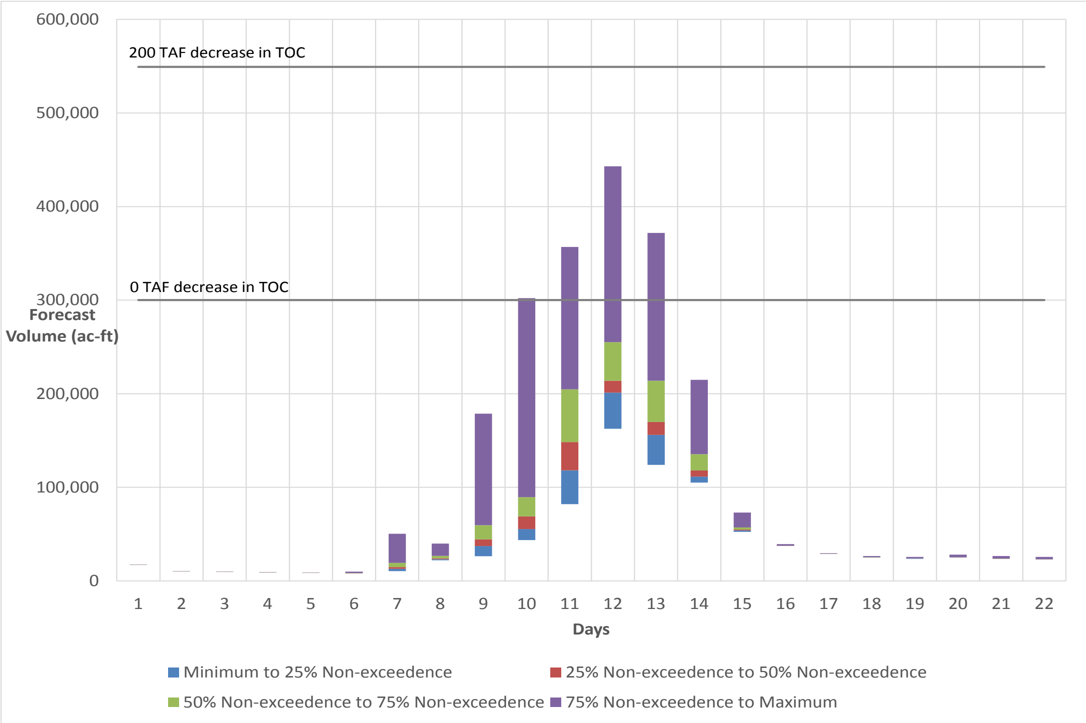


Figure G- 1: Hindcast 1-day Volume Quartiles of the WY 1986 ACE 1=1/100 Event

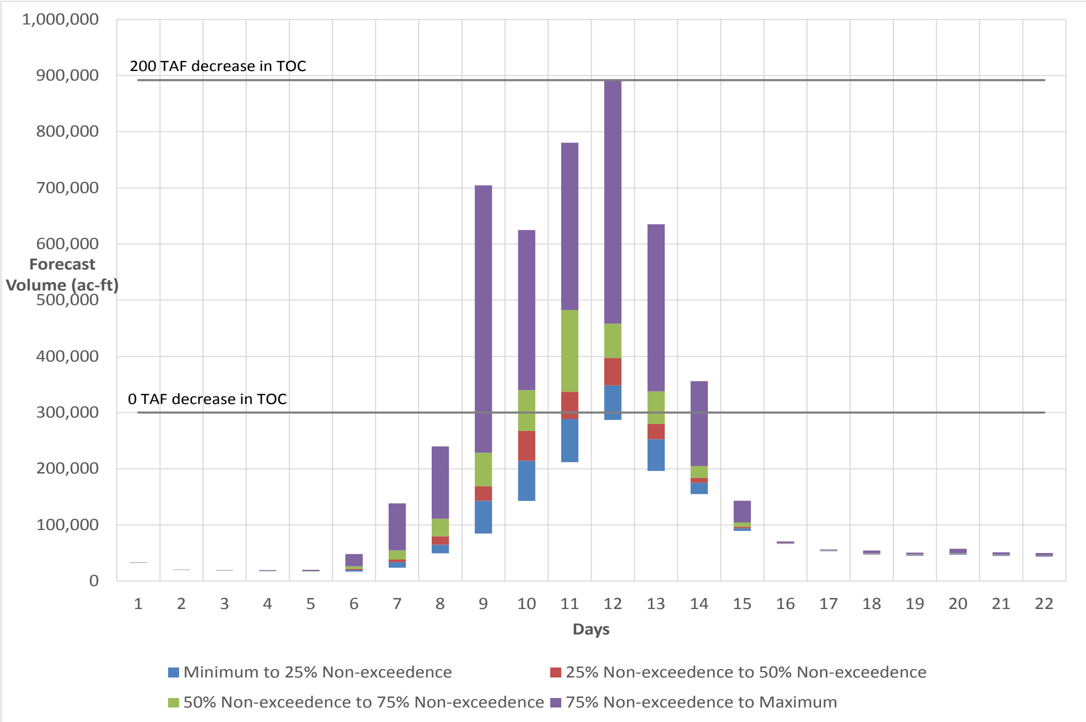


Figure G- 2: Hindcast 2-day Volume Quartiles of the WY 1986 ACE=1/100 Event

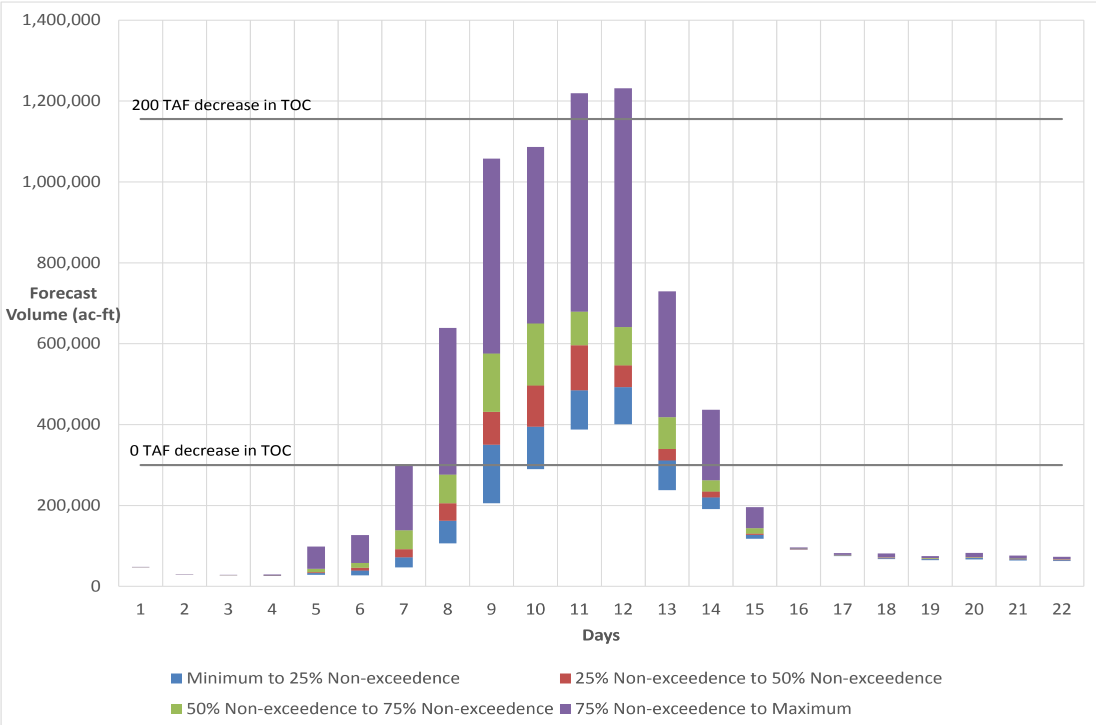


Figure G- 3: Hindcast 3-day Volume Quartiles of the WY 1986 ACE=1/100 Event

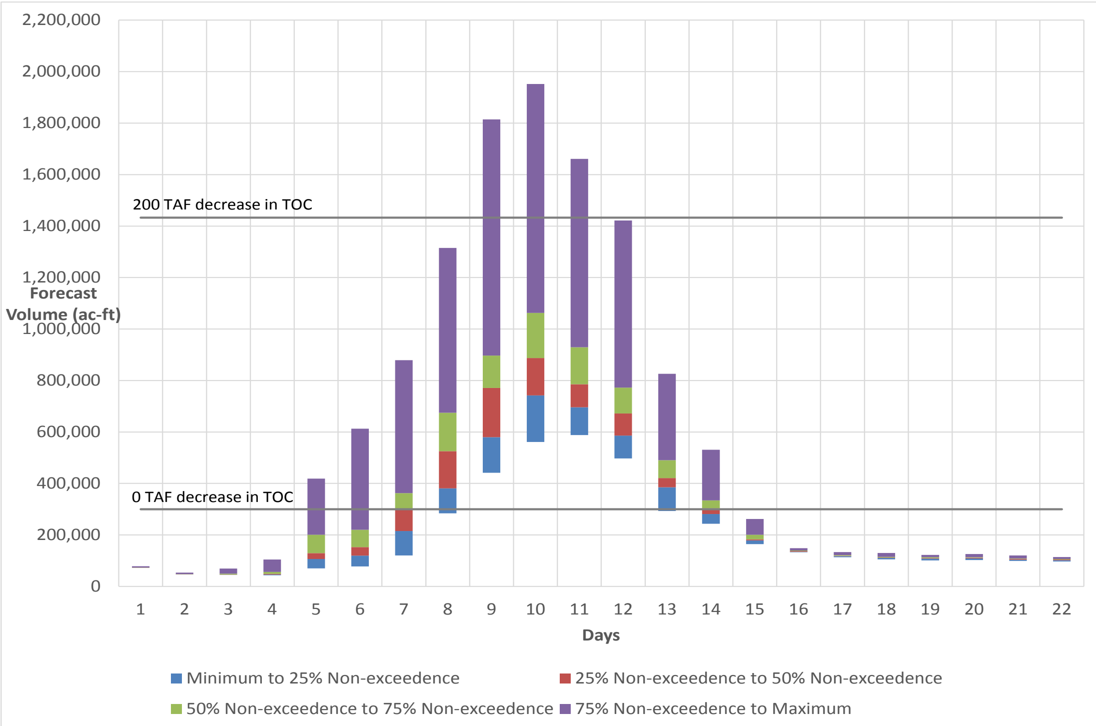


Figure G- 4: Hindcast 5-day Volume Quartiles of the WY 1986 ACE=1/100 Event

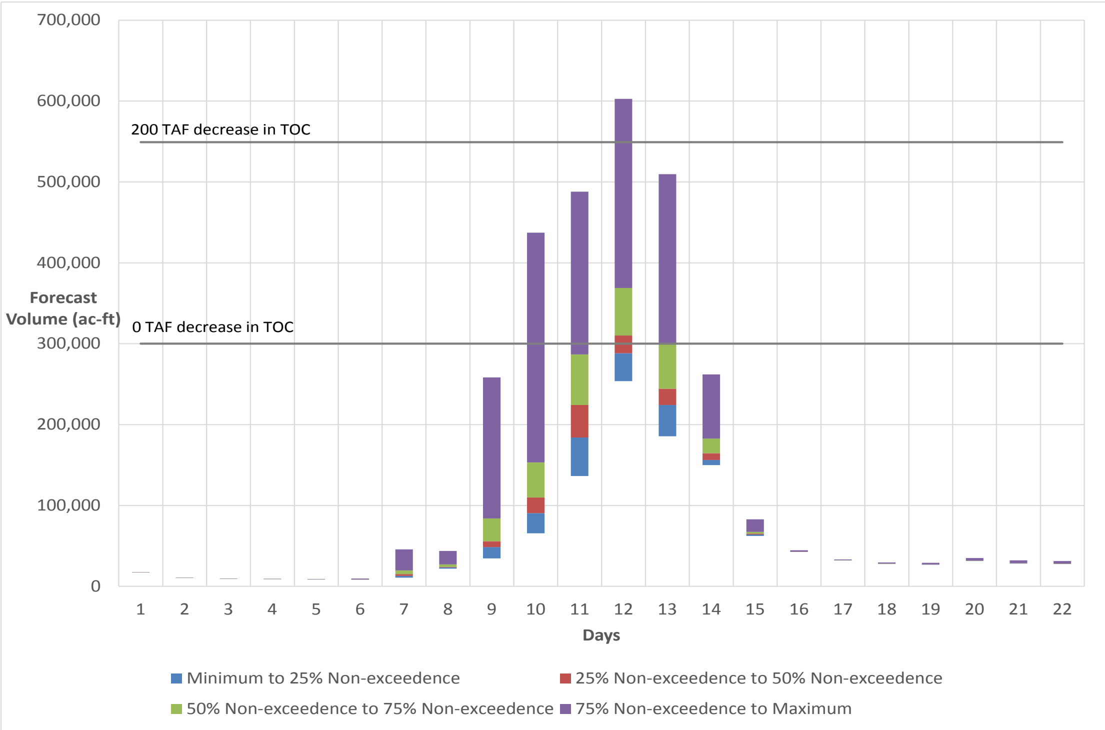


Figure G- 5: Hindcast 1-day Volume Quartiles of the WY 1986 ACE=1/200 Event

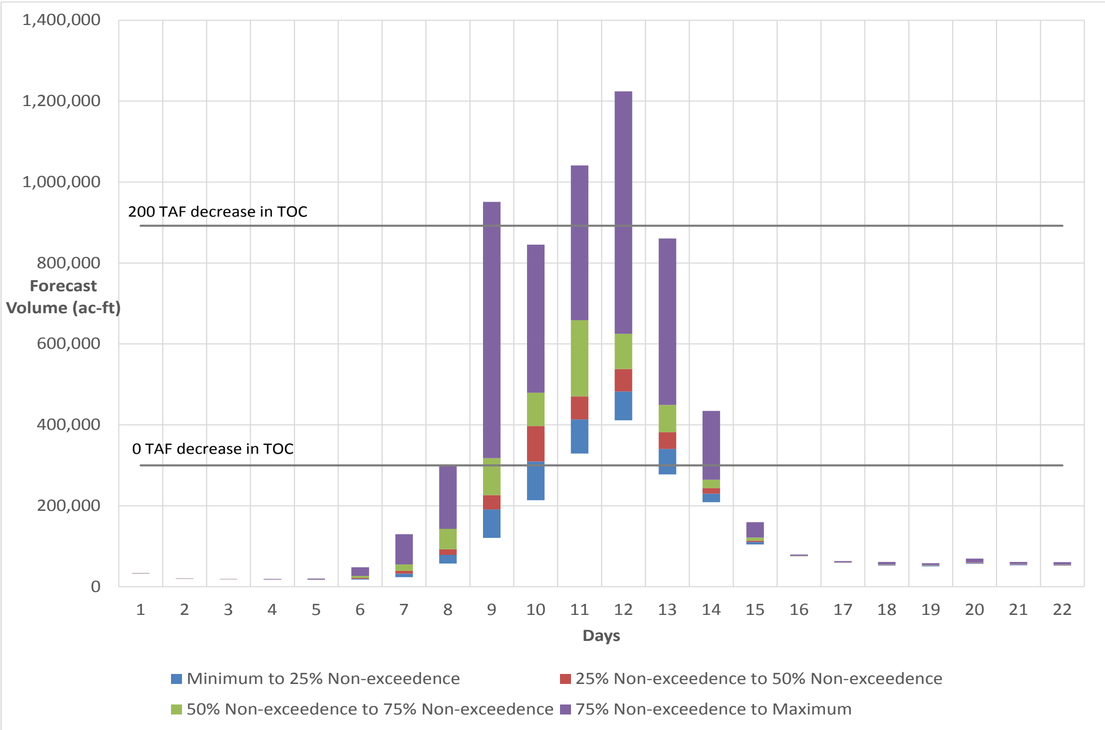


Figure G- 6: Hindcast 2-day Volume Quartiles of the WY 1986 ACE=1/200 Event

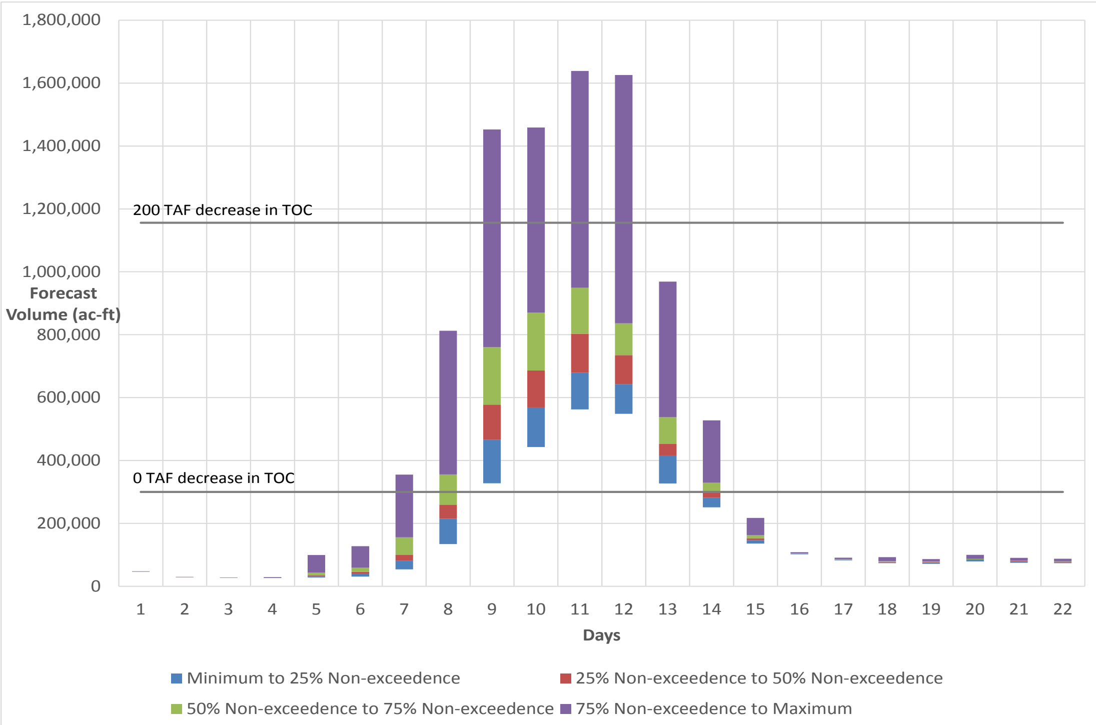


Figure G- 7: Hindcast 3-day Volume Quartiles of the WY 1986 ACE=1/200 Event

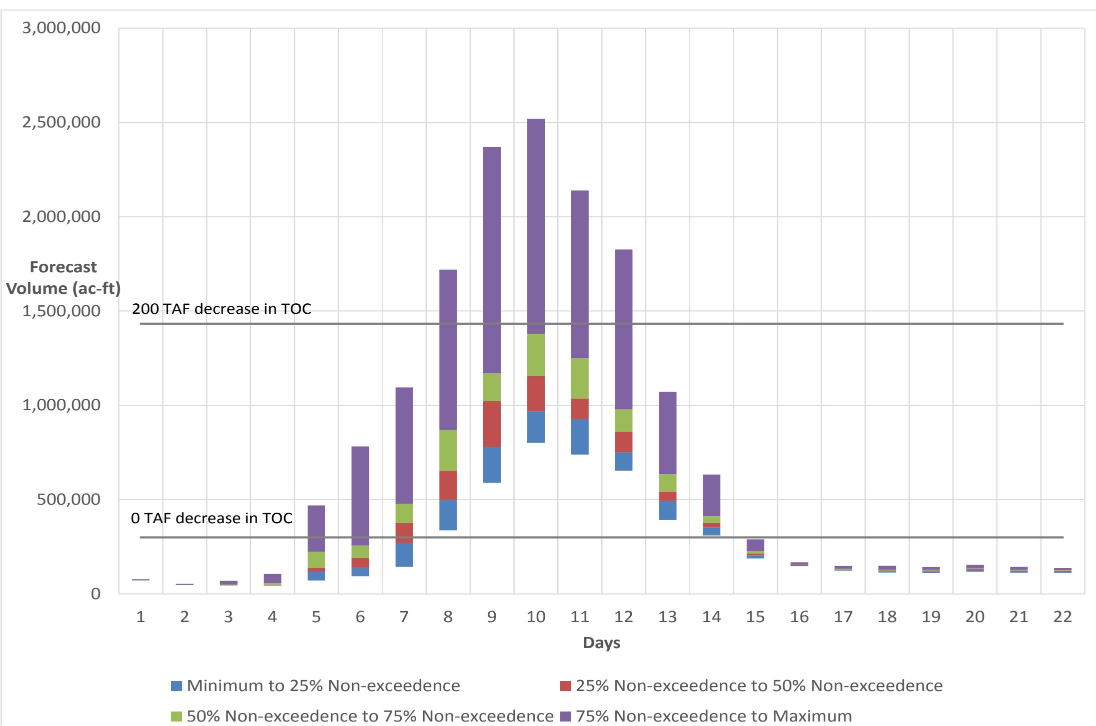


Figure G- 8: Hindcast 5-day Volume Quartiles of the WY 1986 ACE=1/200 Event

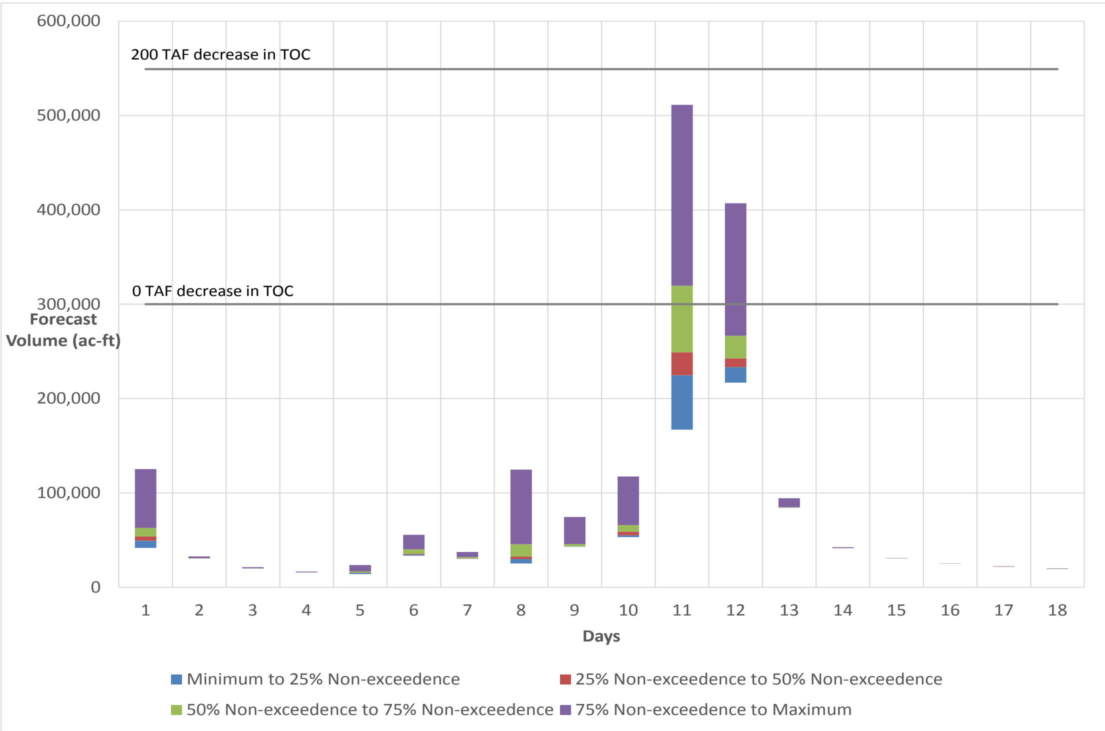


Figure G- 9: Hindcast 1-day Volume Quartiles of the WY 1997 ACE=1/100 Event

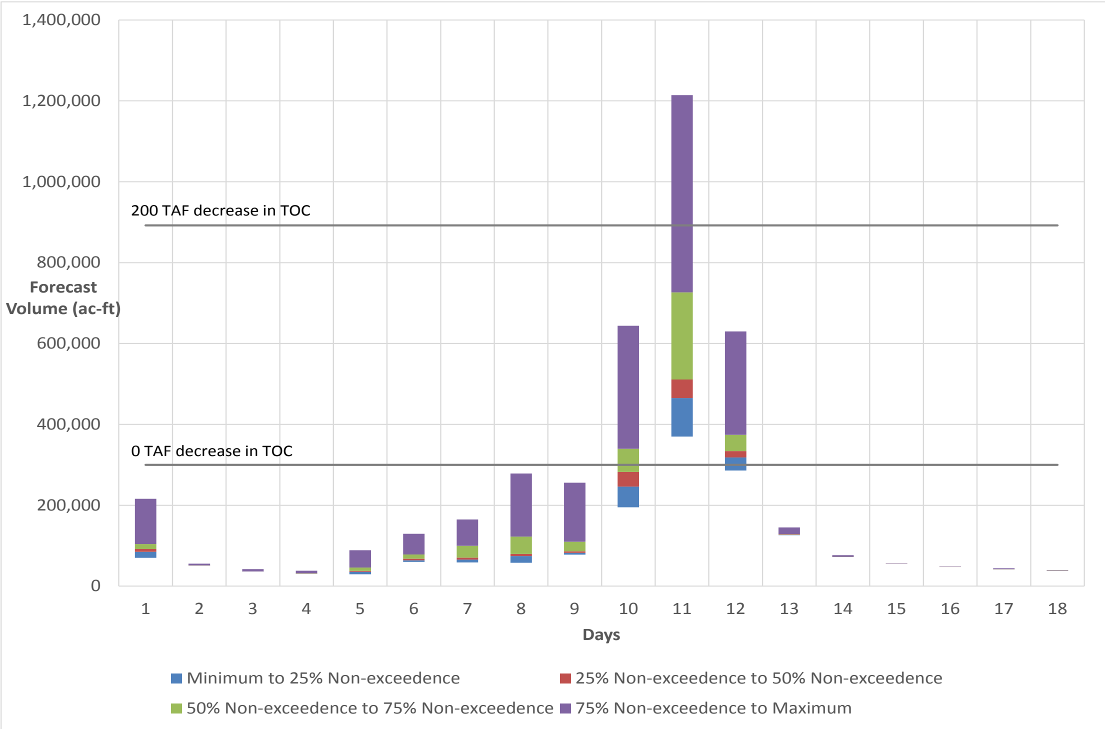


Figure G- 10: Hindcast 2-day Volume Quartiles of the WY 1997 ACE=1/100 Event

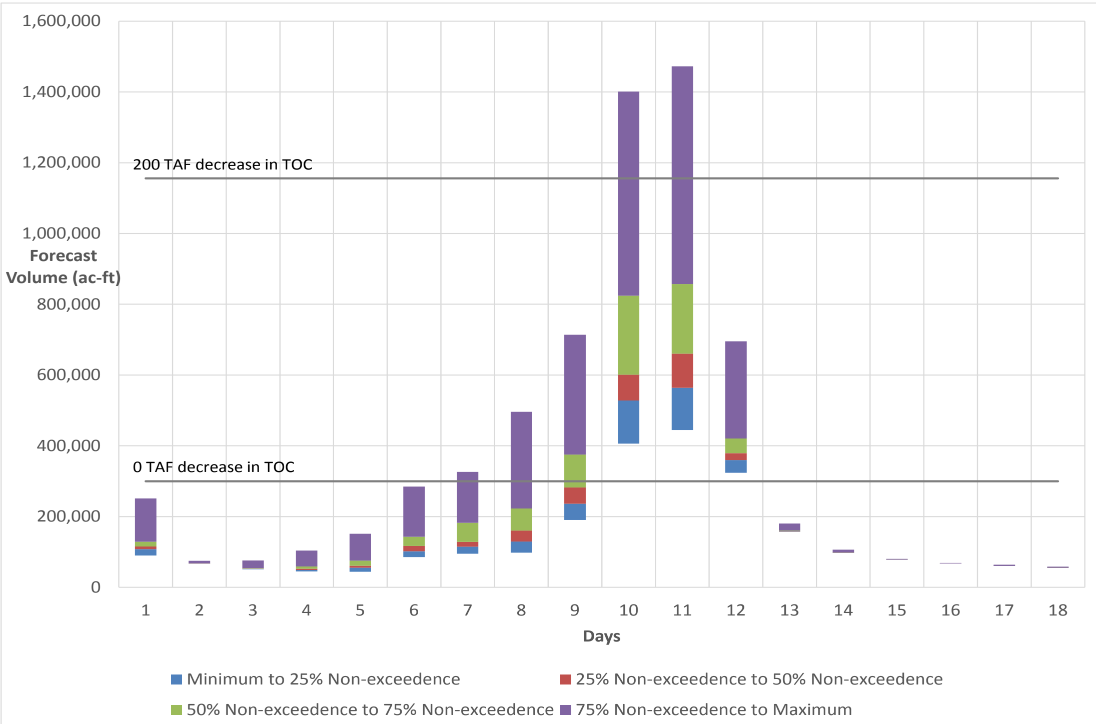


Figure G- 11: Hindcast 3-day Volume Quartiles of the WY 1997 ACE=1/100 Event

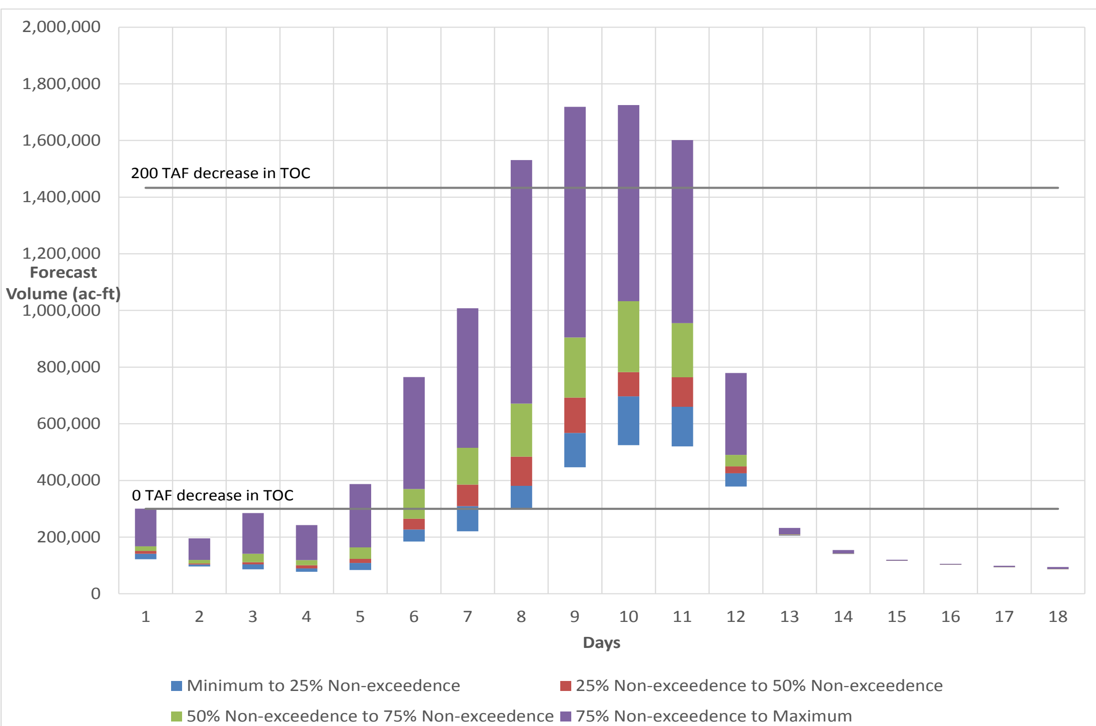


Figure G- 12: Hindcast 5-day Volume Quartiles of the WY 1997 ACE=1/100 Event

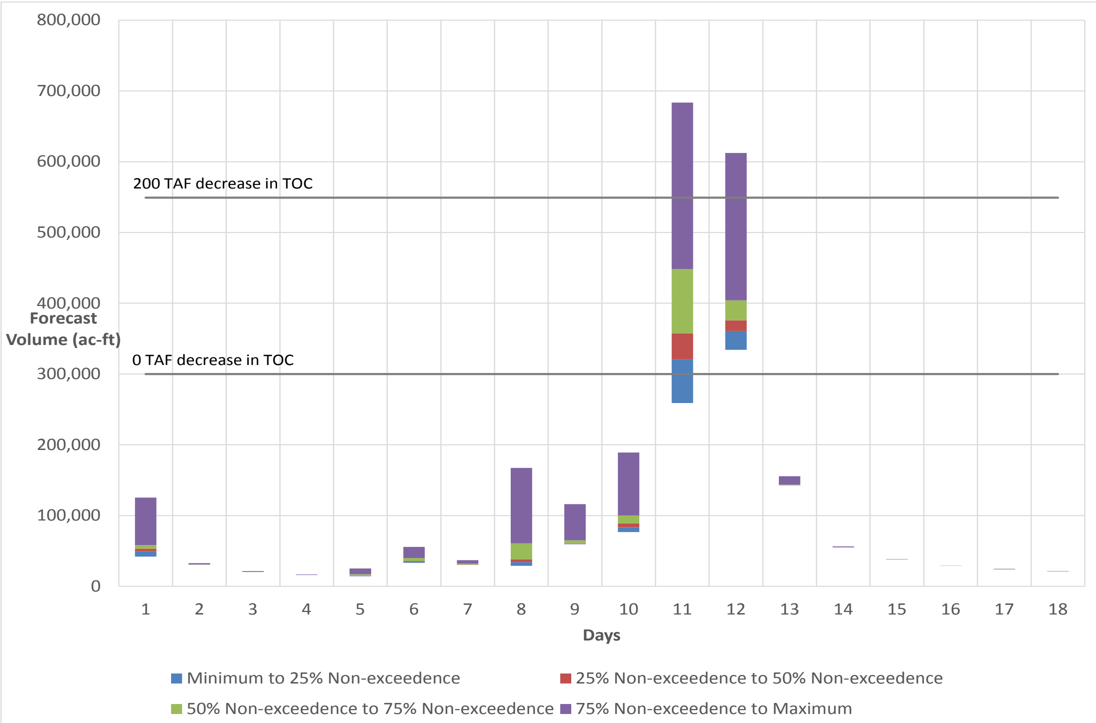


Figure G- 13: Hindcast 1-day Volume Quartiles of the WY 1997 ACE=1/200 Event

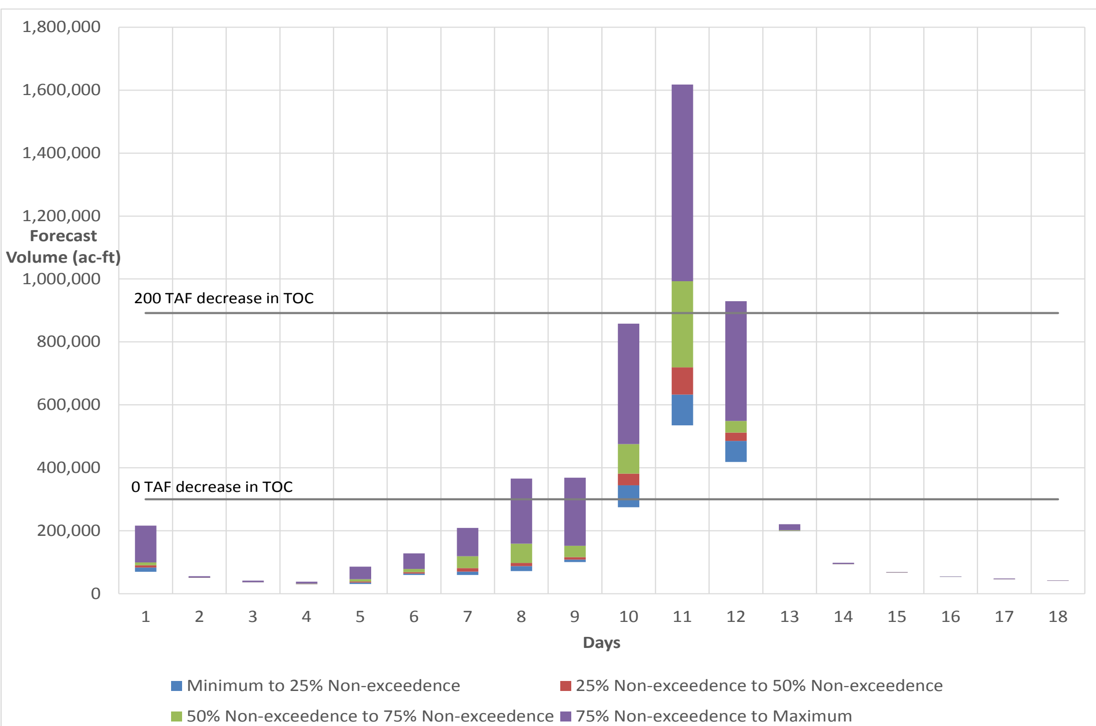


Figure G- 14: Hindcast 2-day Volume Quartiles of the WY 1997 ACE=1/200 Event

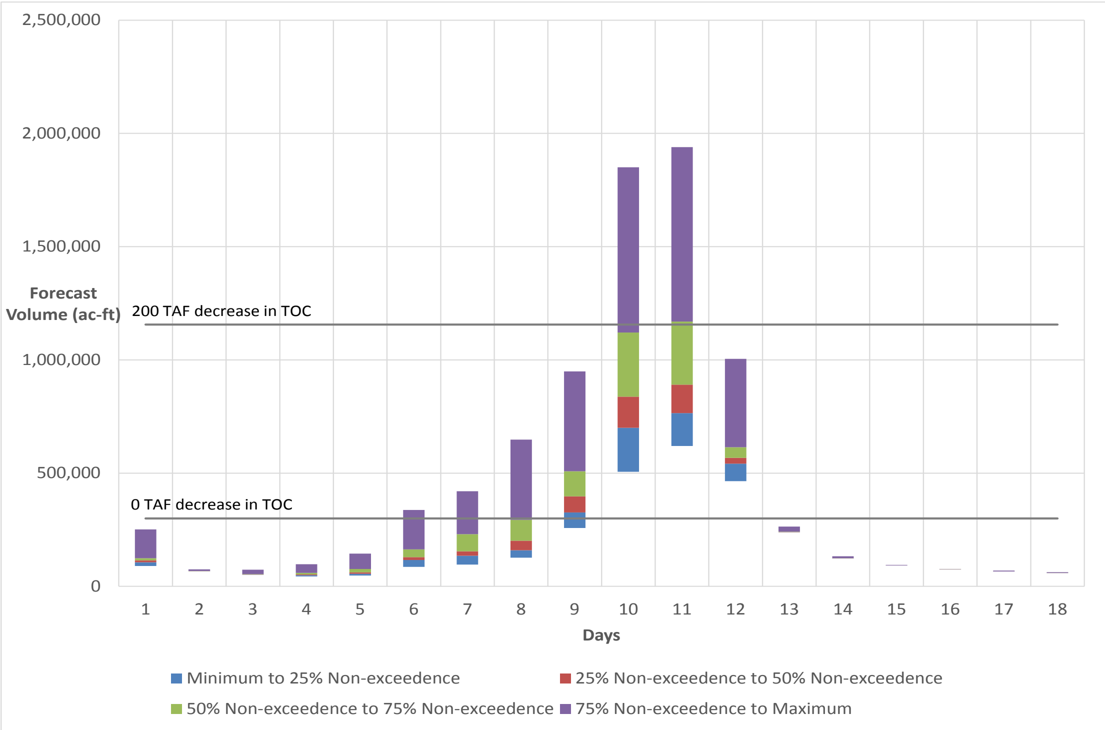


Figure G- 15: Hindcast 3-day Volume Quartiles of the WY 1997 ACE=1/200 Event

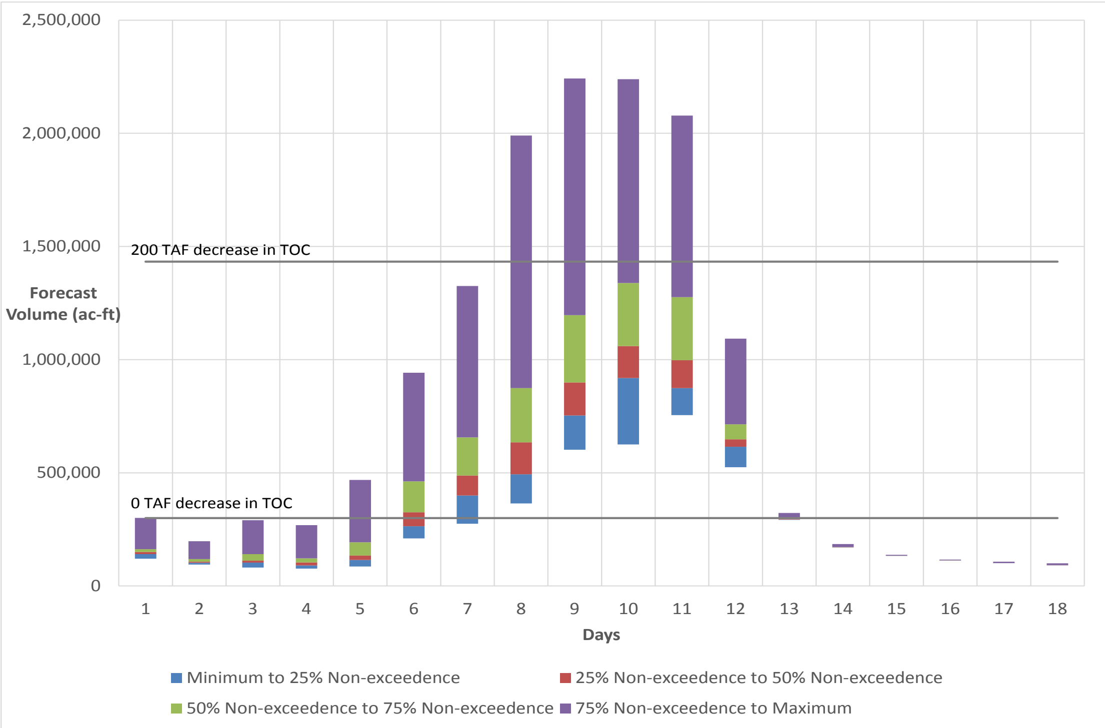


Figure G- 16: Hindcast 5-day Volume Quartiles of the WY 1997 ACE=1/200 Event

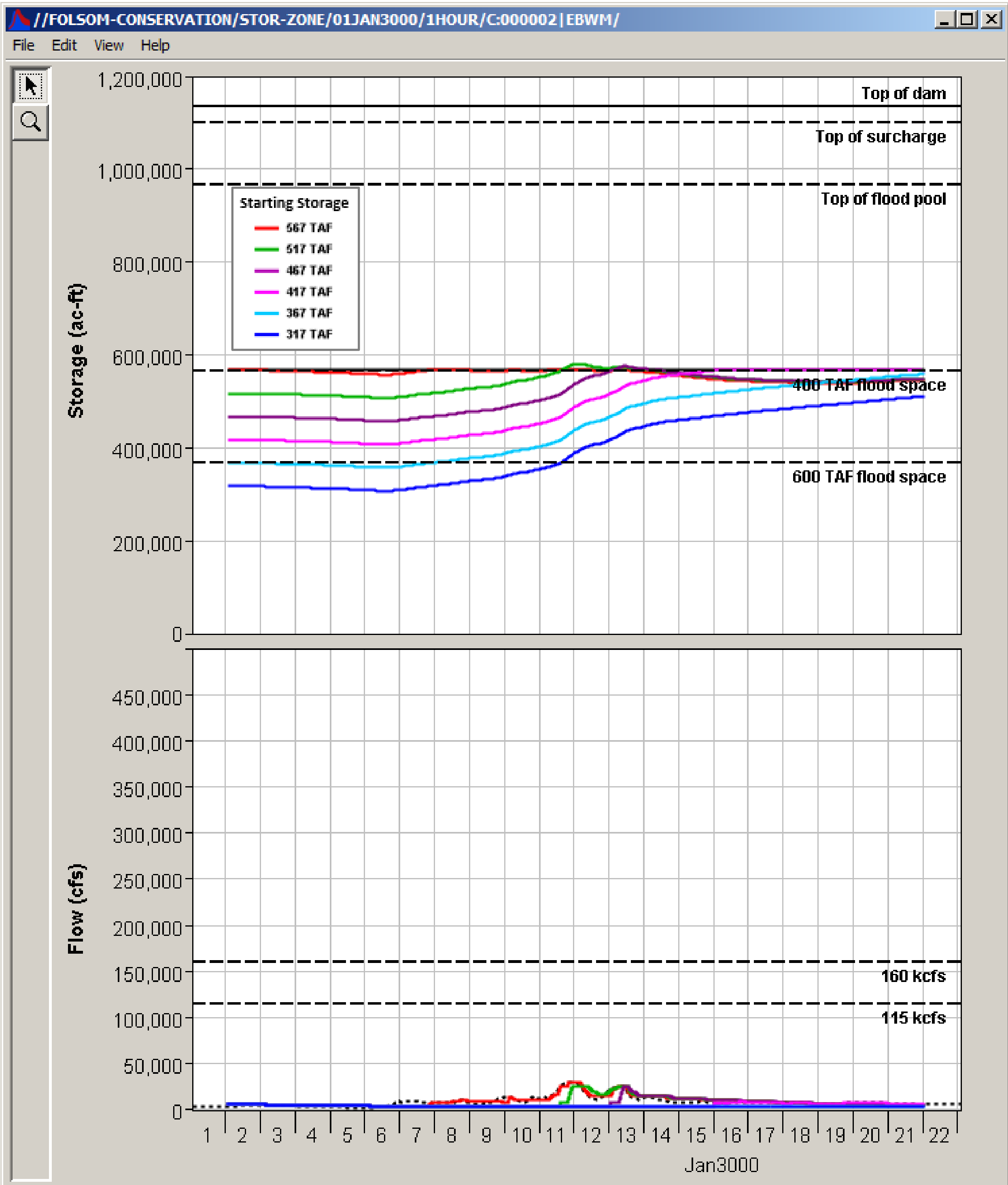


Figure G- 17: Comparison of Storage and Flow Results for the WY 1986 Event Pattern Balanced to ACE=1/2 for Selected Starting Storage Conditions

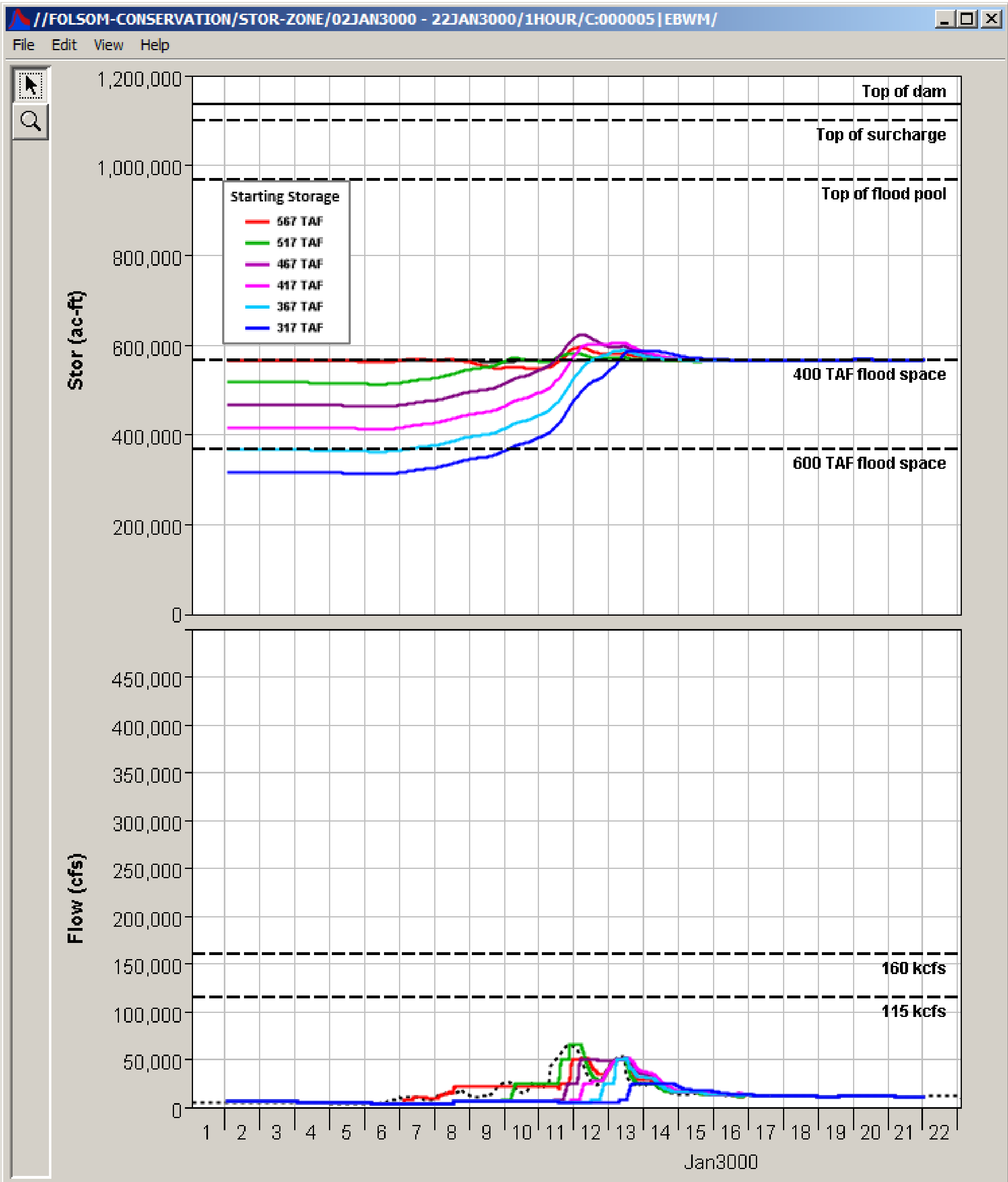


Figure G- 18: Comparison of Storage and Flow Results for the WY 1986 Event Pattern Balanced to ACE=1/5 for Selected Starting Storage Conditions

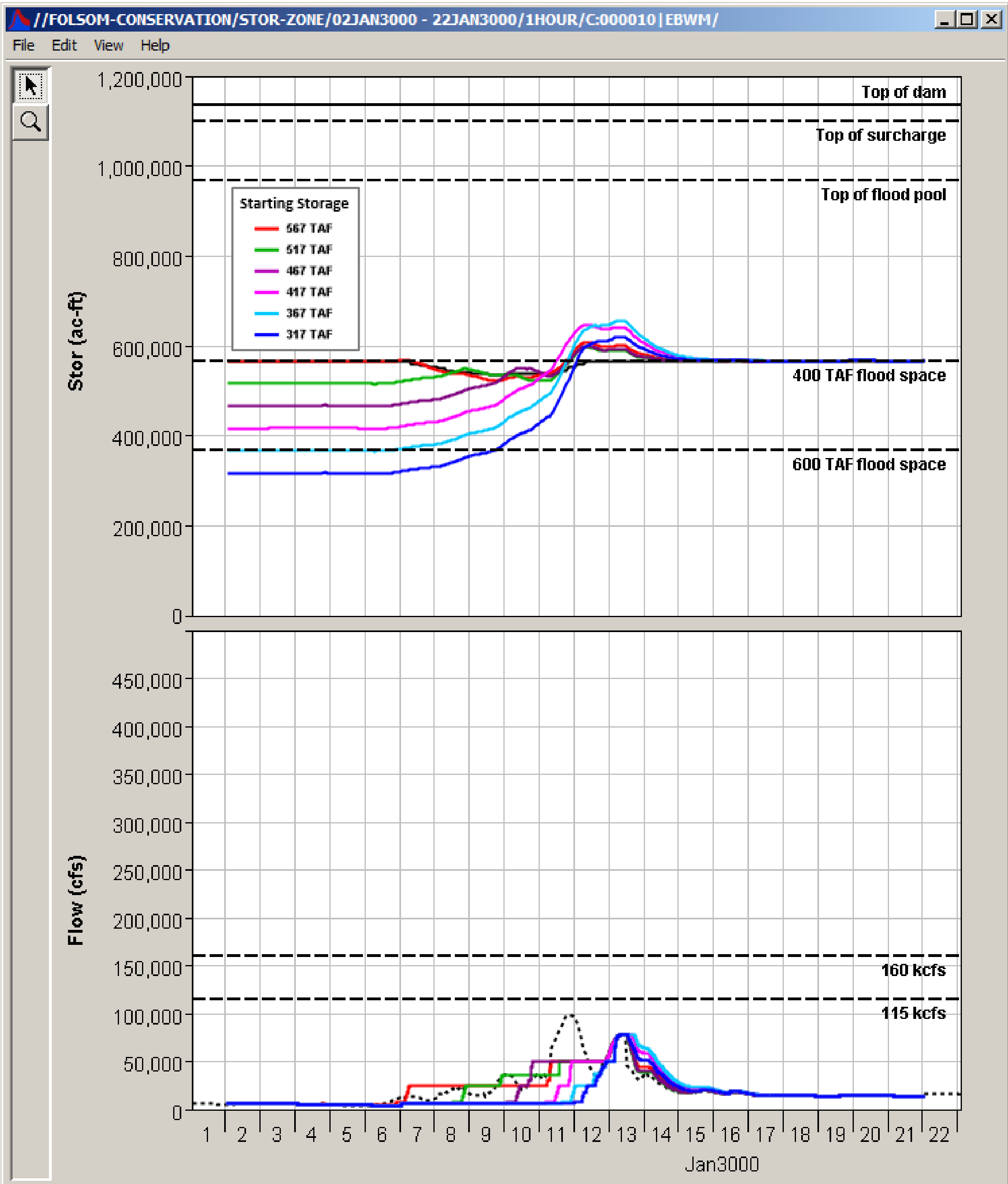


Figure G- 19: Comparison of Storage and Flow Results for the WY 1986 Event Pattern Balanced to ACE=1/10 for Selected Starting Storage Conditions

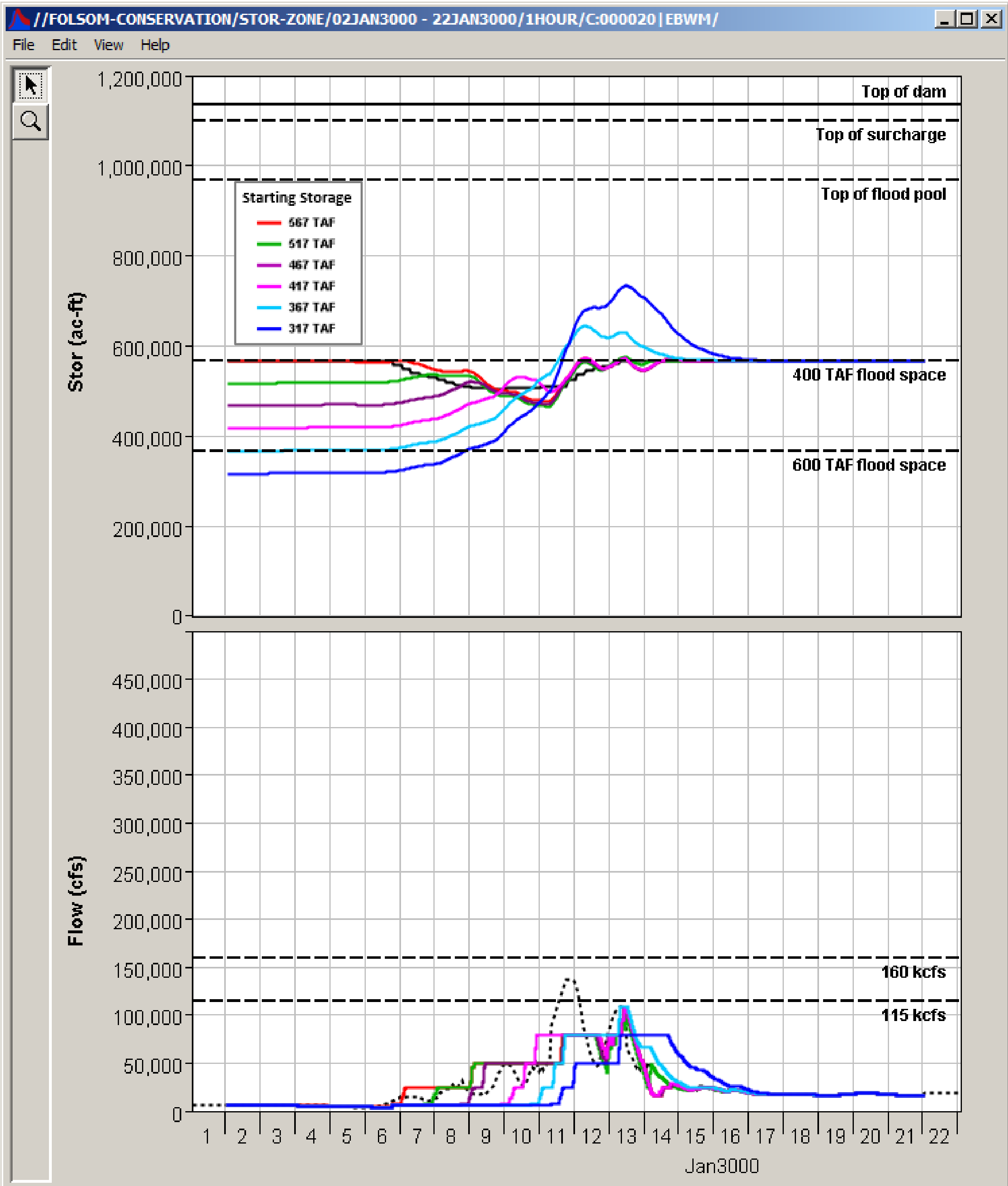


Figure G- 20: Comparison of Storage and Flow Results for the WY 1986 Event Pattern Balanced to ACE=1/20 for Selected Starting Storage Conditions

- DRAFT -

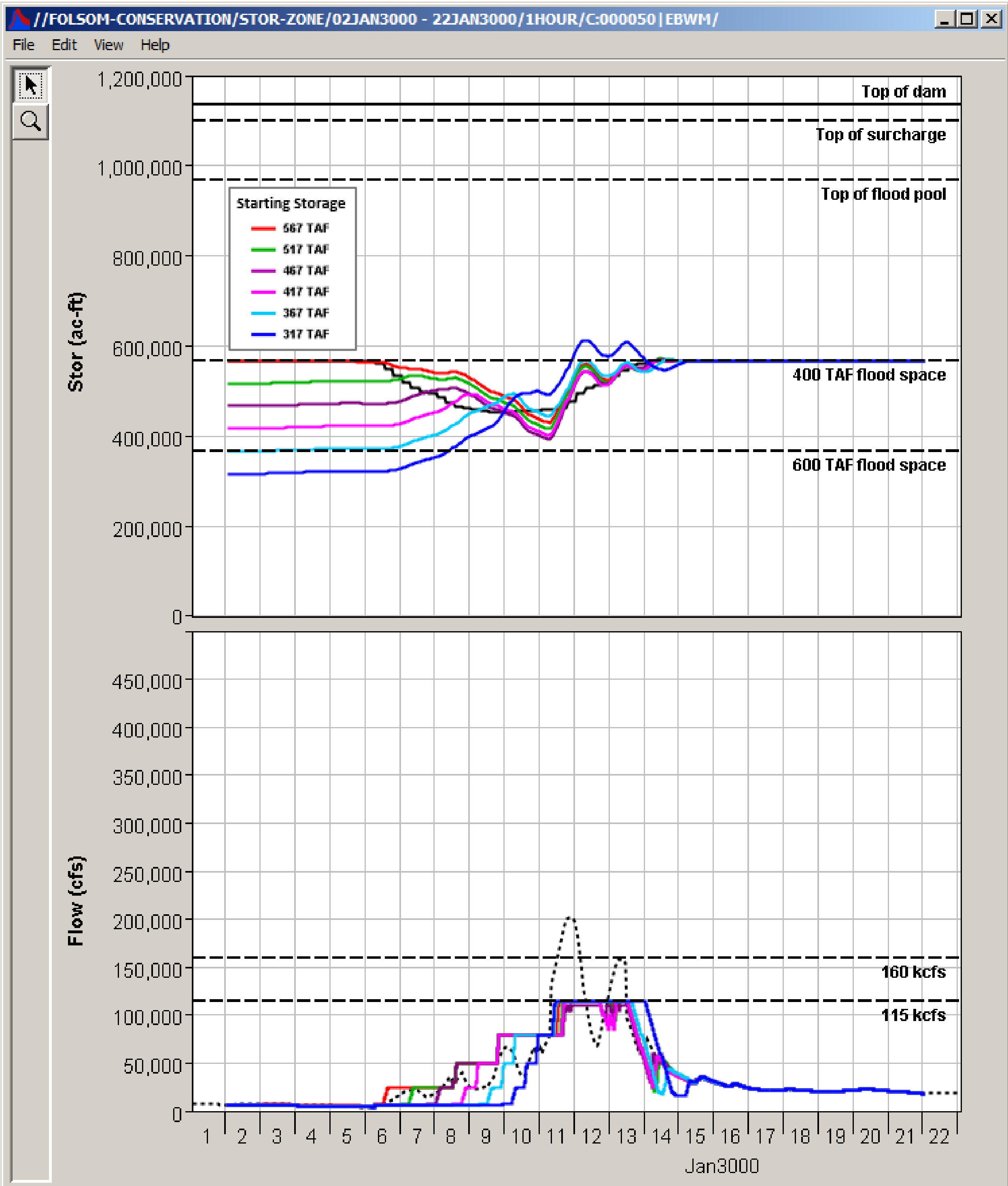


Figure G- 21: Comparison of Storage and Flow Results for the WY 1986 Event Pattern Balanced to ACE=1/50 for Selected Starting Storage Conditions

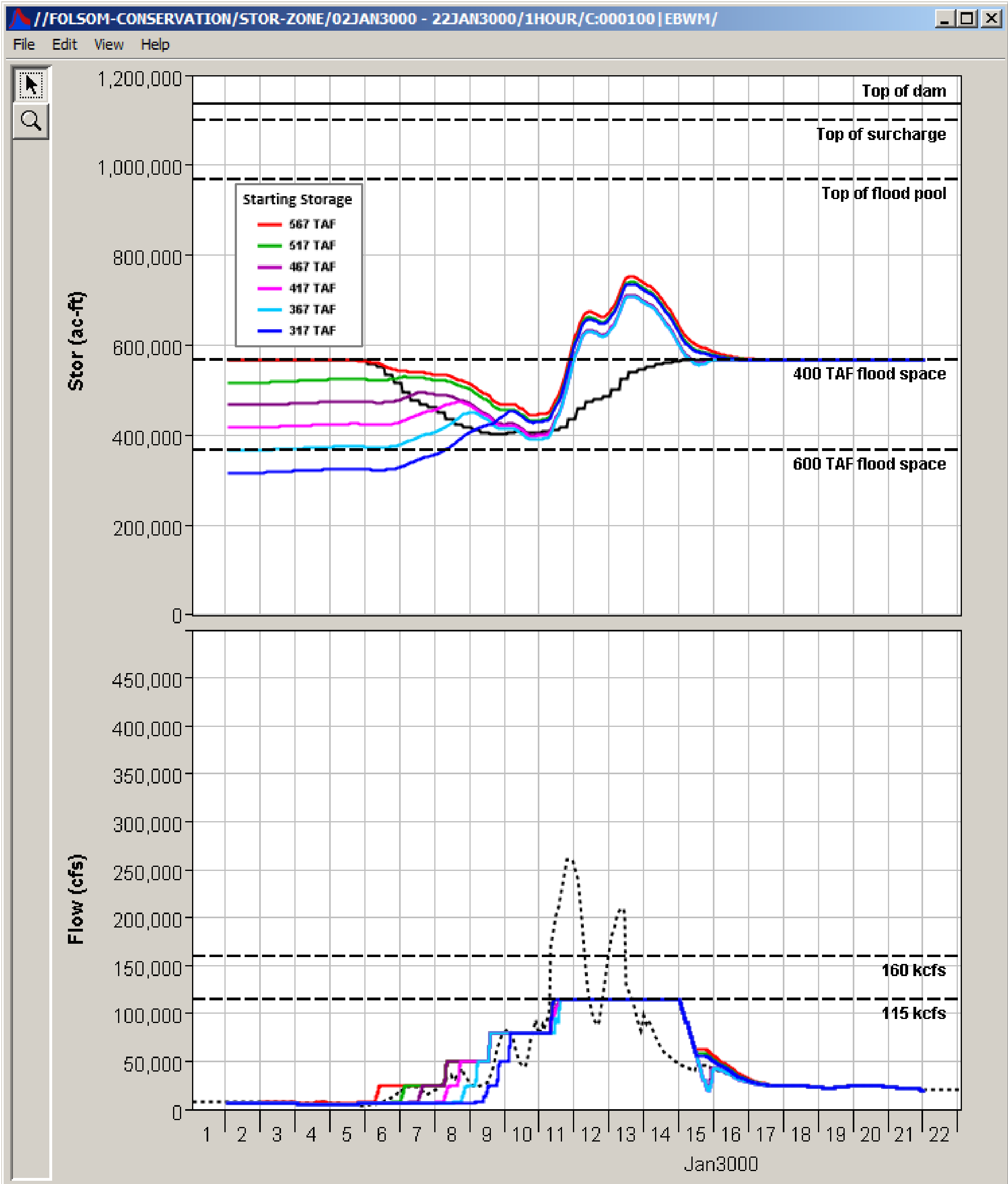


Figure G- 22: Comparison of Storage and Flow Results for the WY 1986 Event Pattern Balanced to ACE=1/100 for Selected Starting Storage Conditions

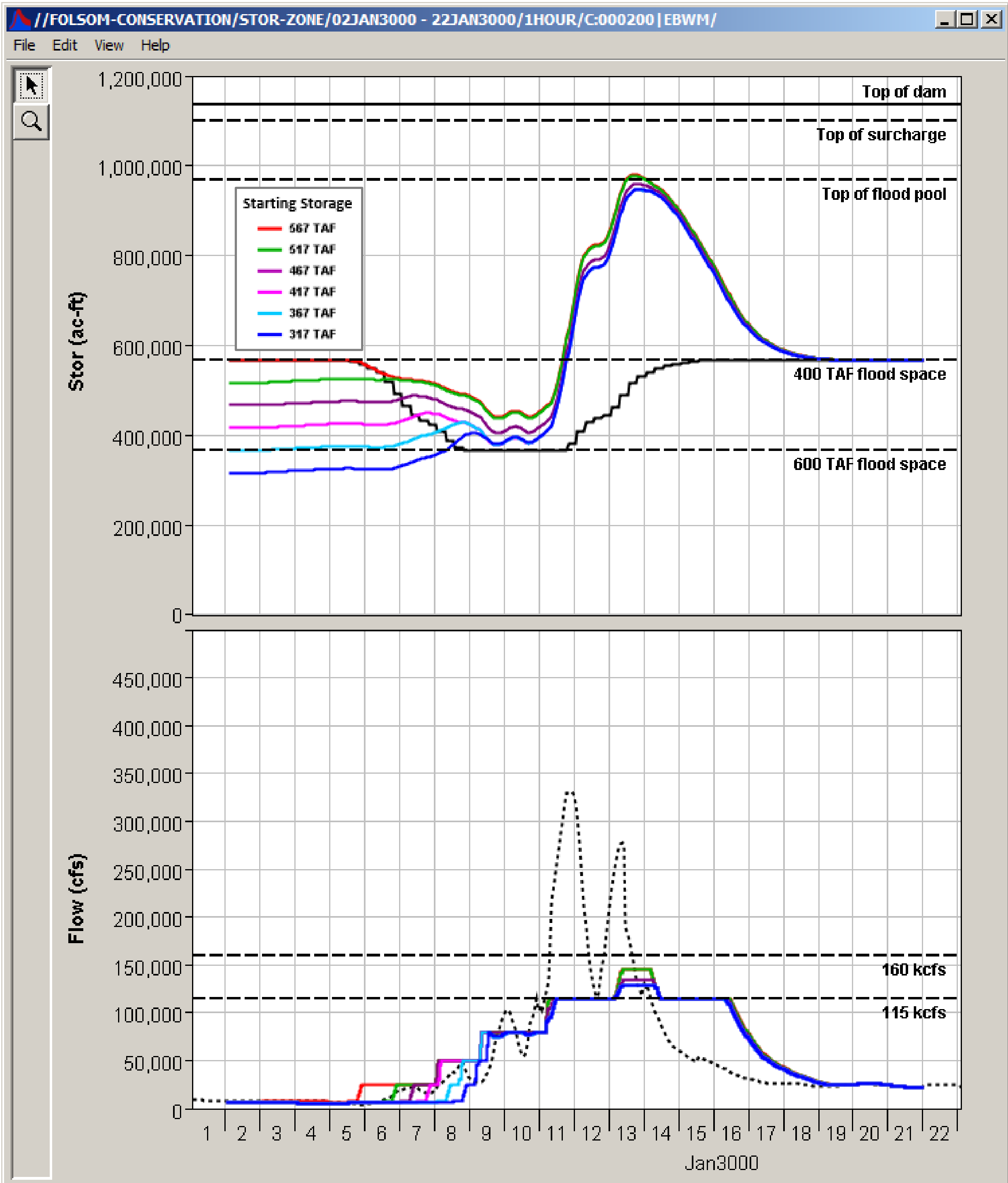


Figure G- 23: Comparison of Storage and Flow Results for the WY 1986 Event Pattern Balanced to ACE=1/200 for Selected Starting Storage Conditions

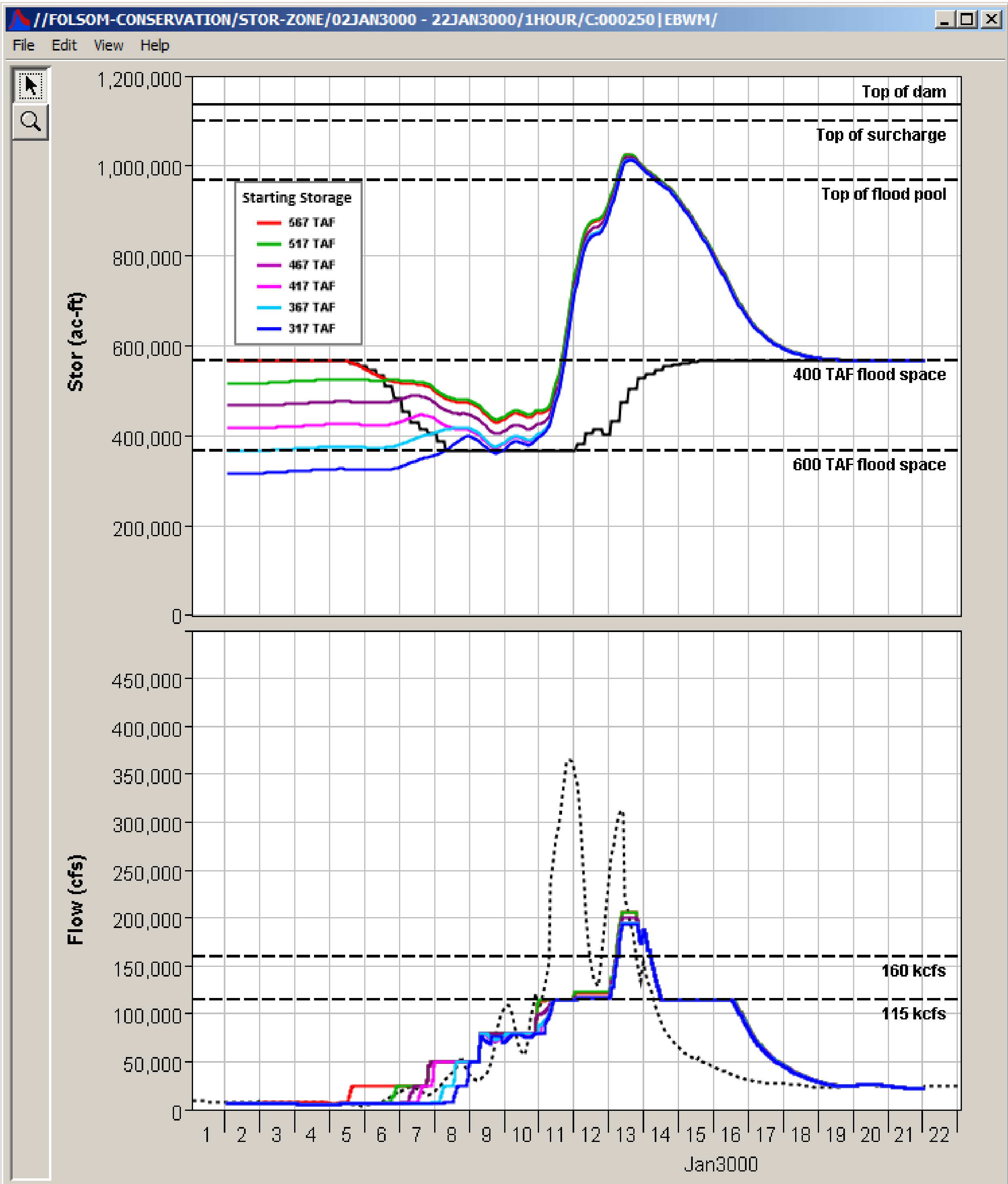


Figure G- 24: Comparison of Storage and Flow Results for the WY 1986 Event Pattern Balanced to ACE=1/250 for Selected Starting Storage Conditions

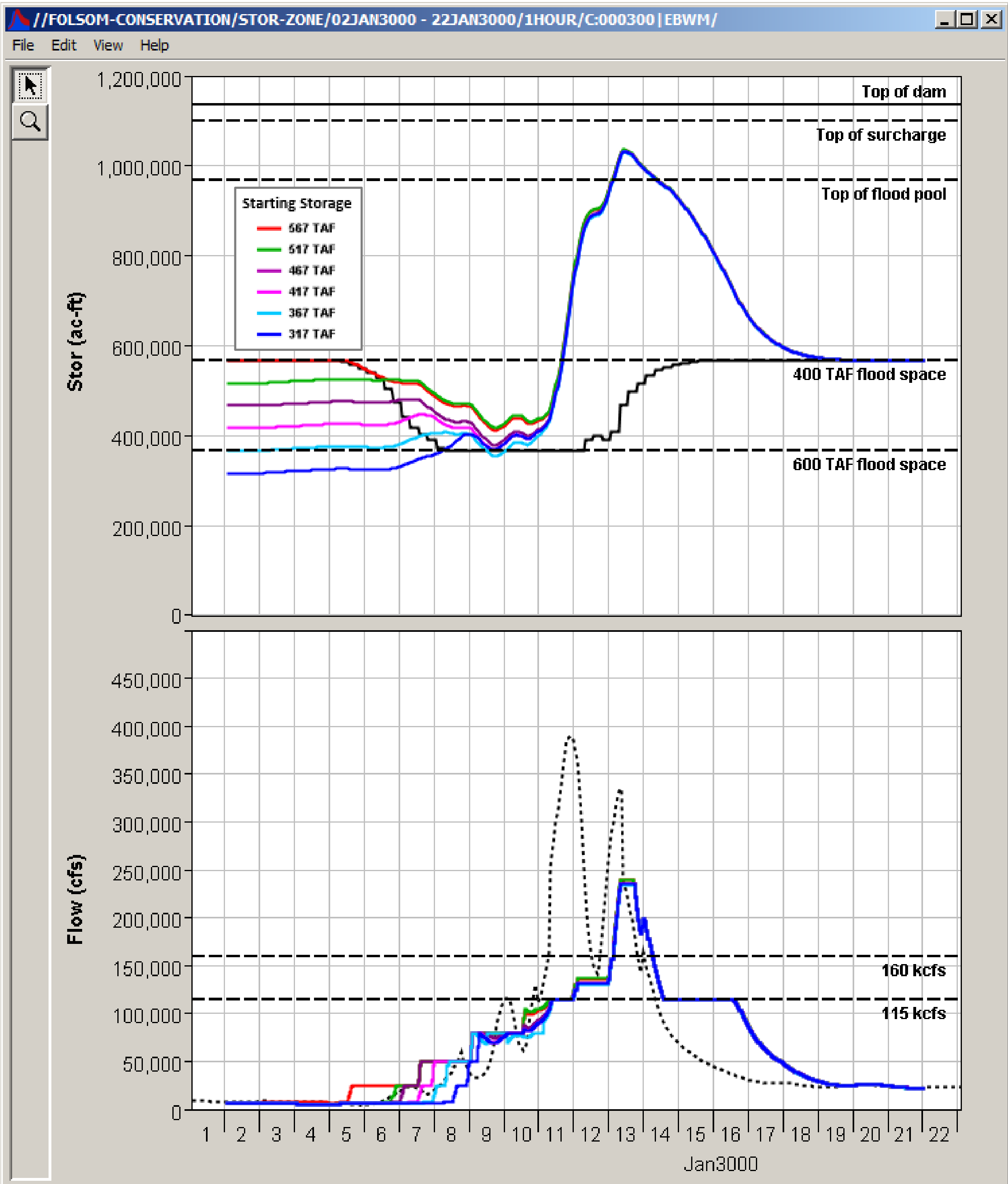


Figure G- 25: Comparison of Storage and Flow Results for the WY 1986 Event Pattern Balanced to ACE=1/300 for Selected Starting Storage Conditions

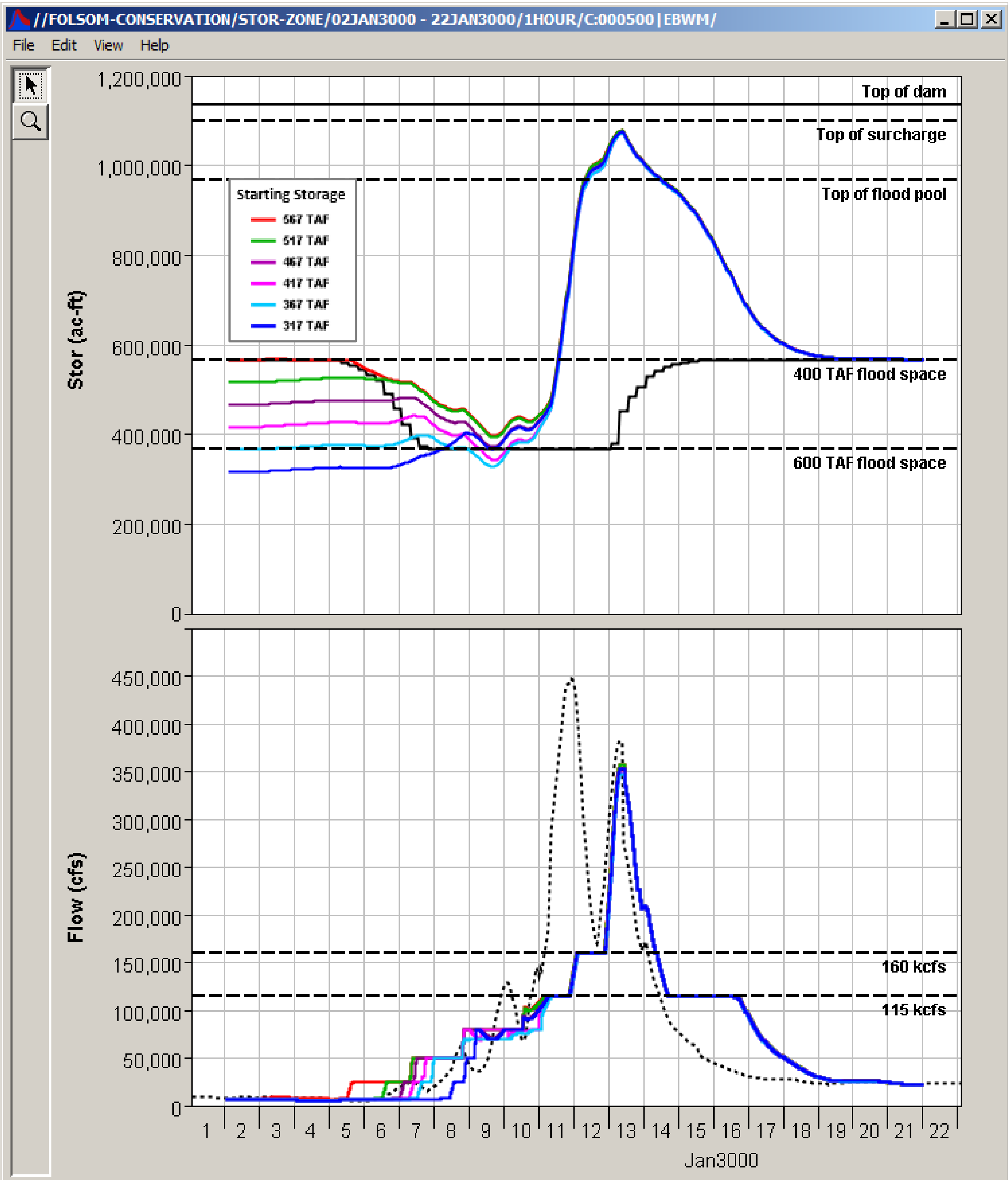


Figure G- 26: Comparison of Storage and Flow Results for the WY 1986 Event Pattern Balanced to ACE=1/500 for Selected Starting Storage Conditions

- DRAFT -

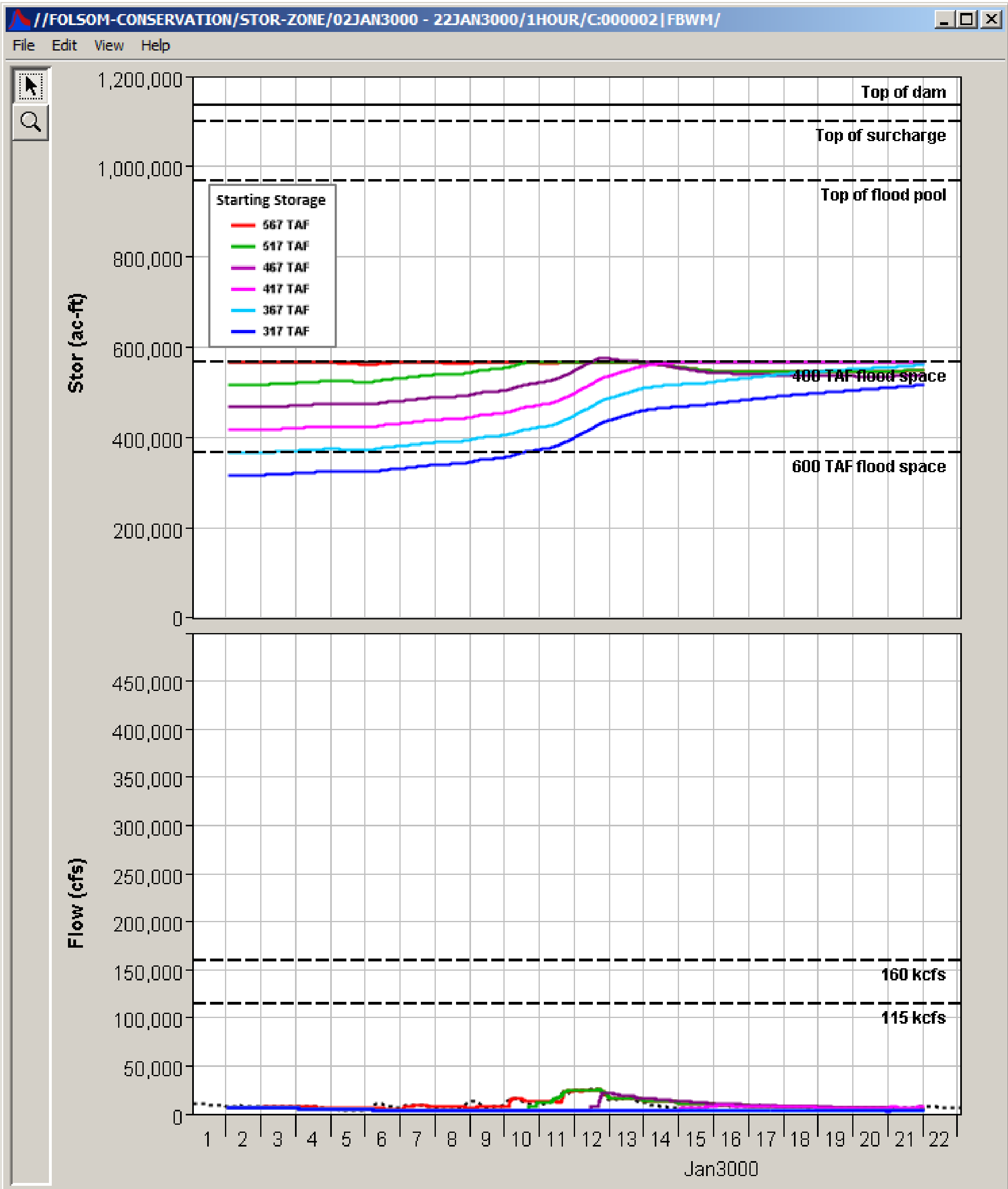


Figure G- 27: Comparison of Storage and Flow Results for the WY 1997 Event Pattern Balanced to ACE=1/2 for Selected Starting Storage Conditions

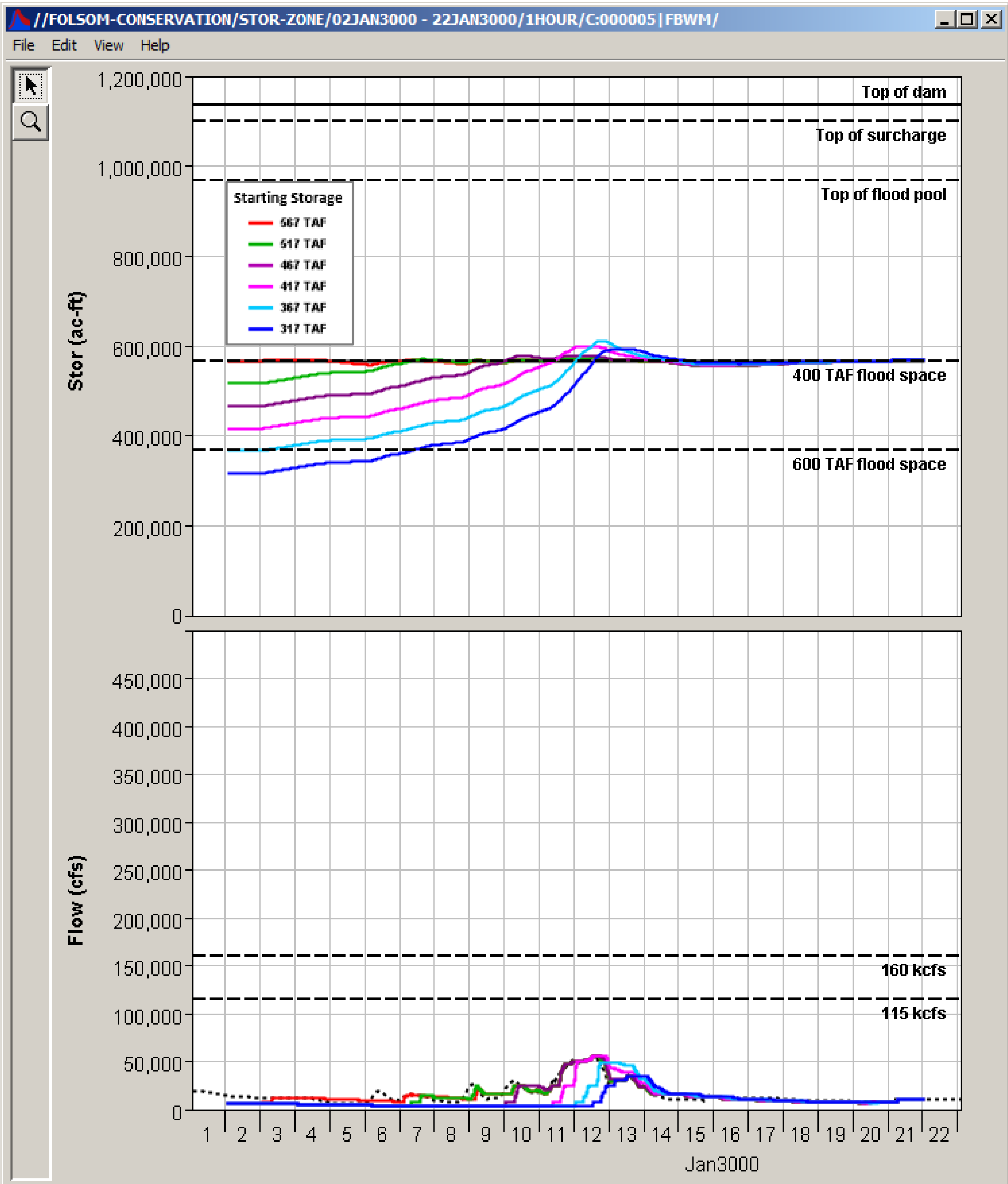


Figure G- 28: Comparison of Storage and Flow Results for the WY 1997 Event Pattern Balanced to ACE=1/5 for Selected Starting Storage Conditions

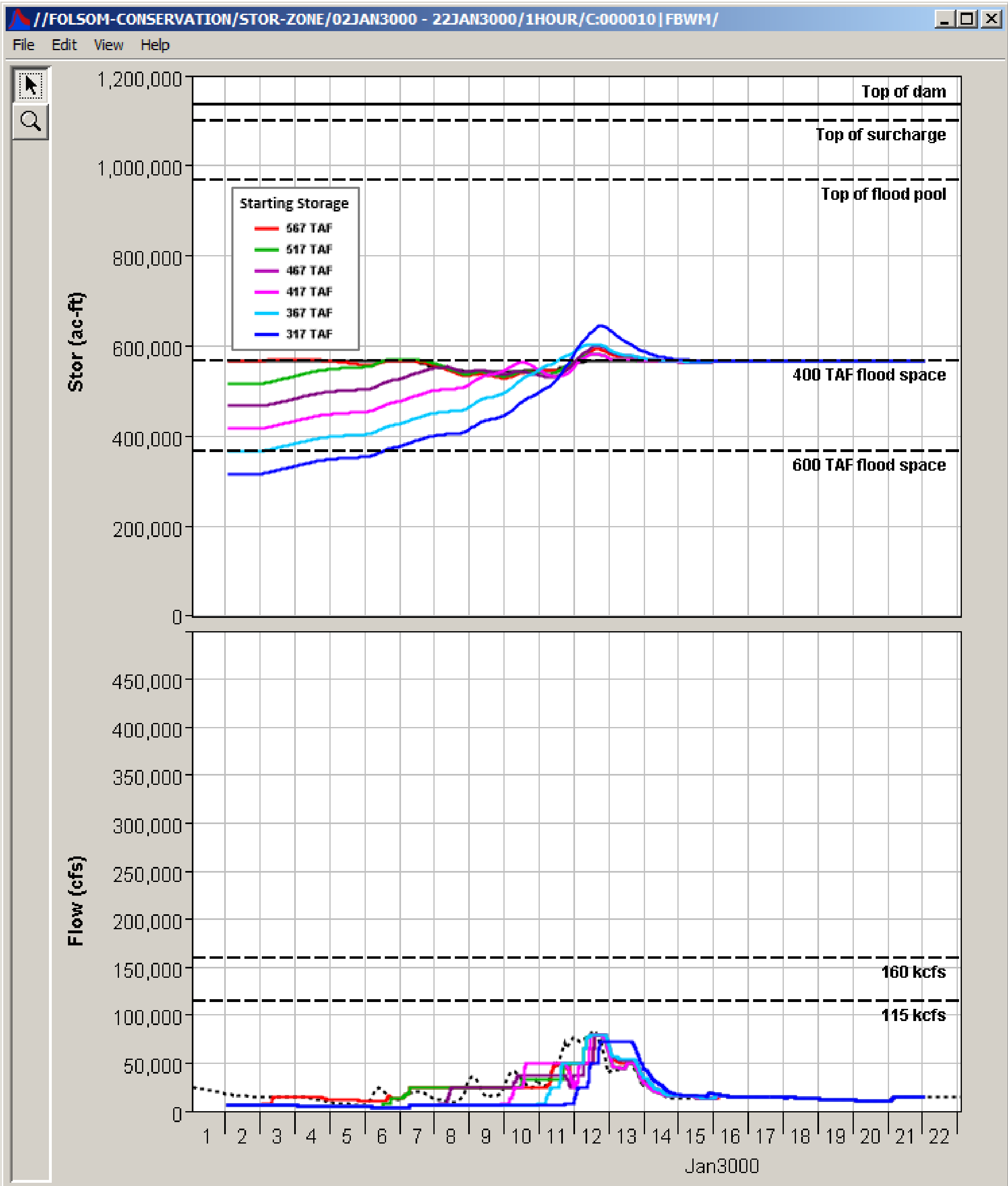


Figure G- 29: Comparison of Storage and Flow Results for the WY 1997 Event Pattern Balanced to ACE=1/10 for Selected Starting Storage Conditions

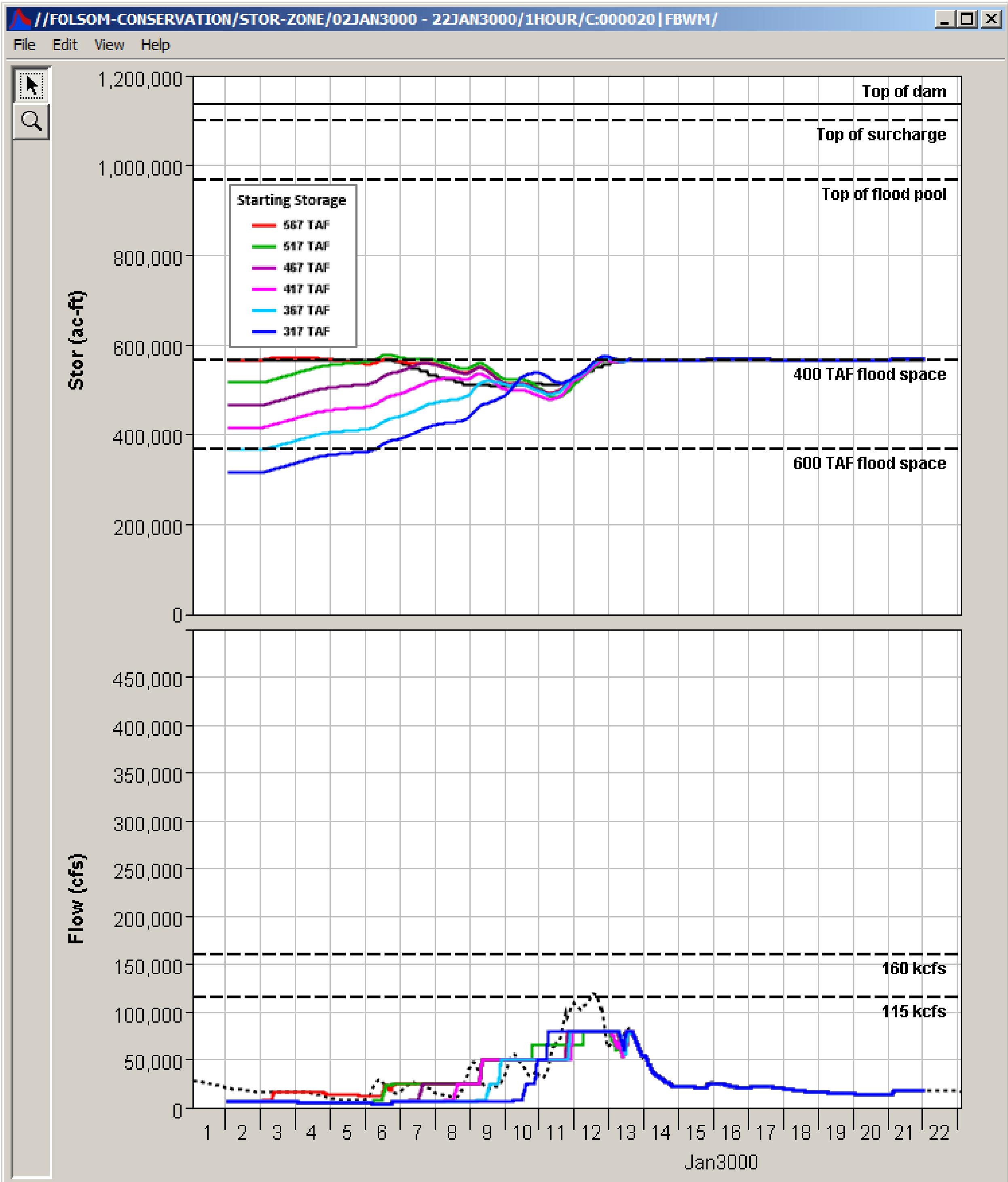


Figure G- 30: Comparison of Storage and Flow Results for the WY 1997 Event Pattern Balanced to ACE=1/20 for Selected Starting Storage Conditions

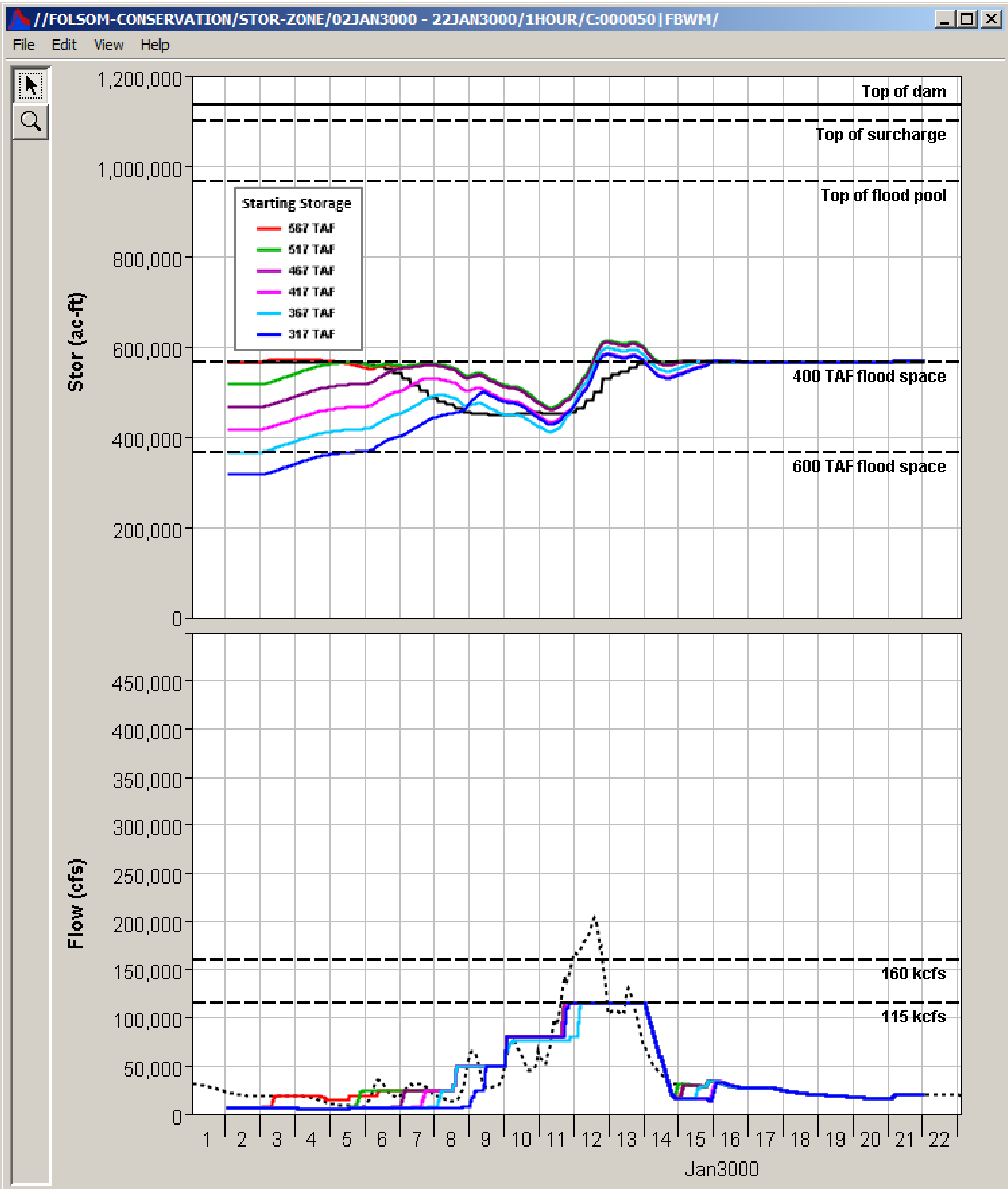


Figure G- 31: Comparison of Storage and Flow Results for the WY 1997 Event Pattern Balanced to ACE=1/50 for Selected Starting Storage Conditions

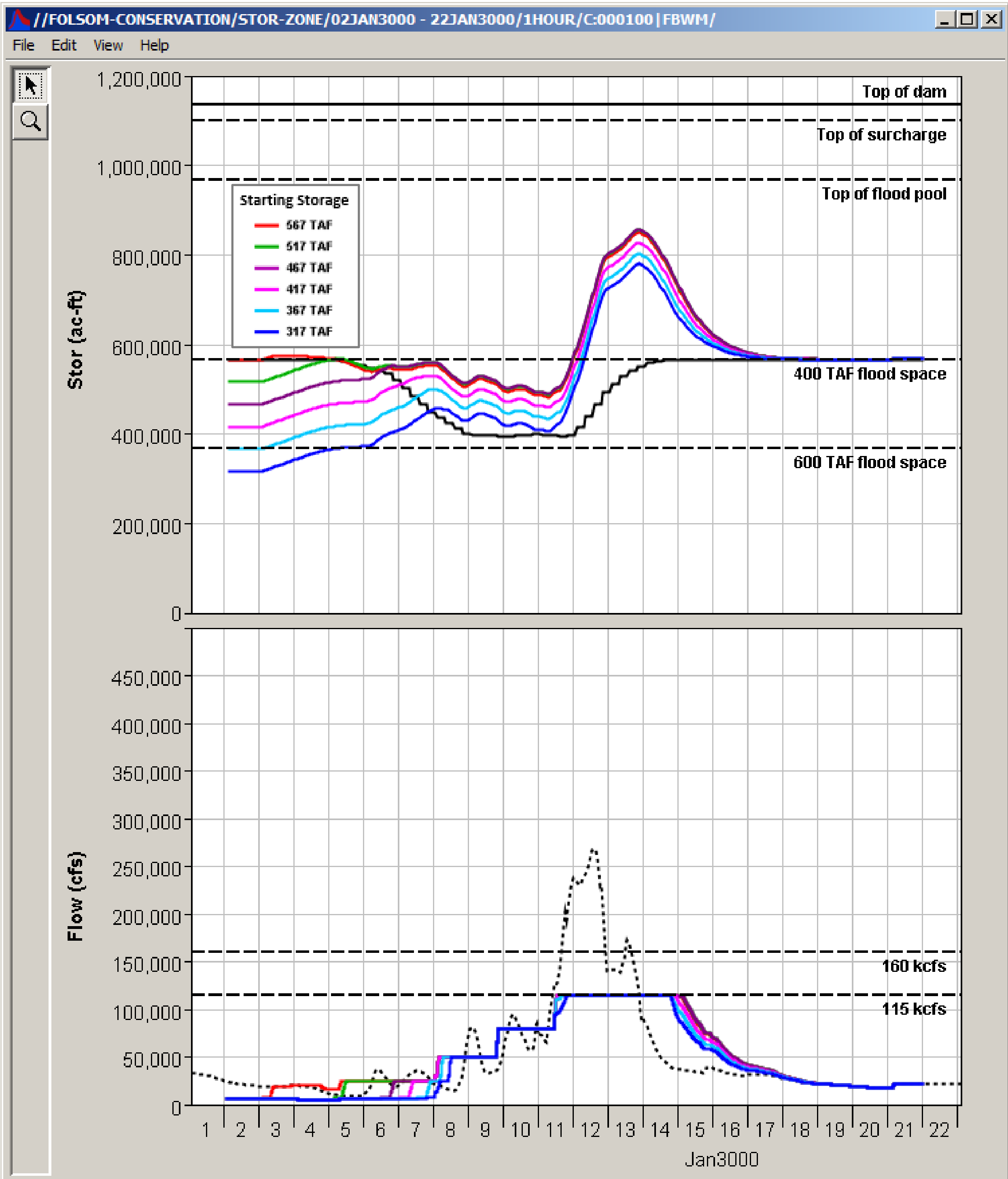


Figure G- 32: Comparison of Storage and Flow Results for the WY 1997 Event Pattern Balanced to ACE=1/100 for Selected Starting Storage Conditions

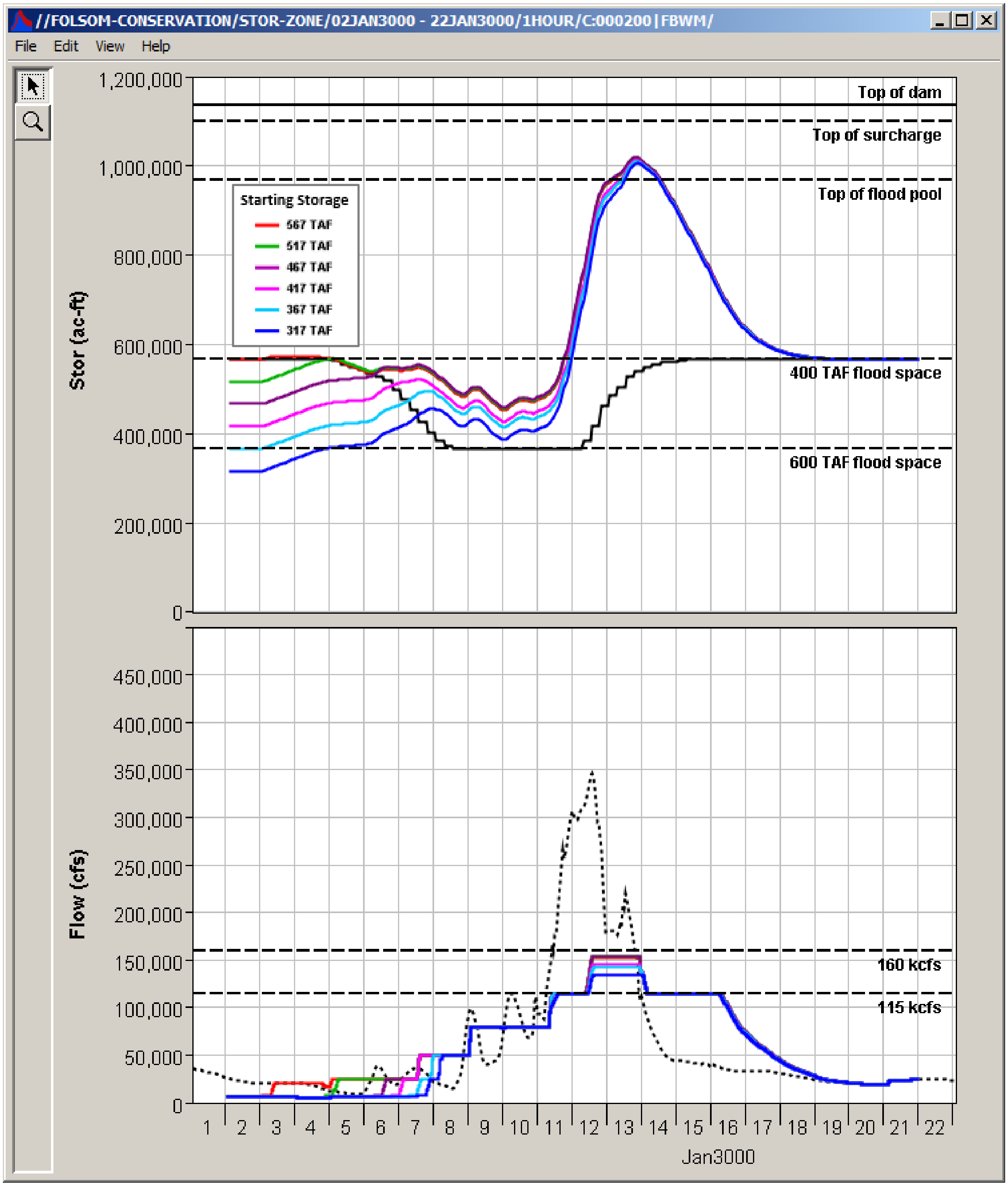


Figure G- 33: Comparison of Storage and Flow Results for the WY 1997 Event Pattern Balanced to ACE=1/200 for Selected Starting Storage Conditions

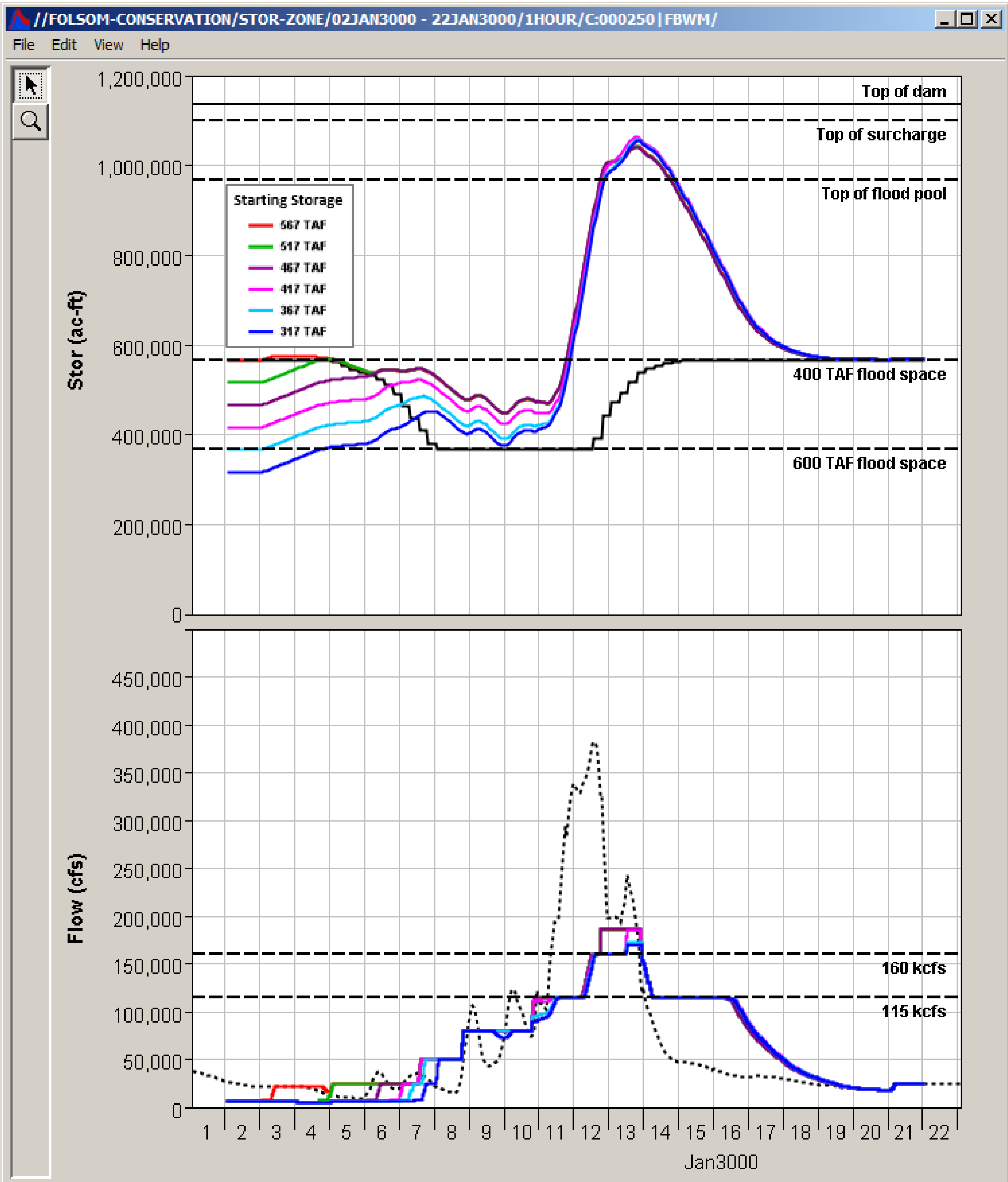


Figure G- 34: Comparison of Storage and Flow Results for the WY 1997 Event Pattern Balanced to ACE=1/250 for Selected Starting Storage Conditions

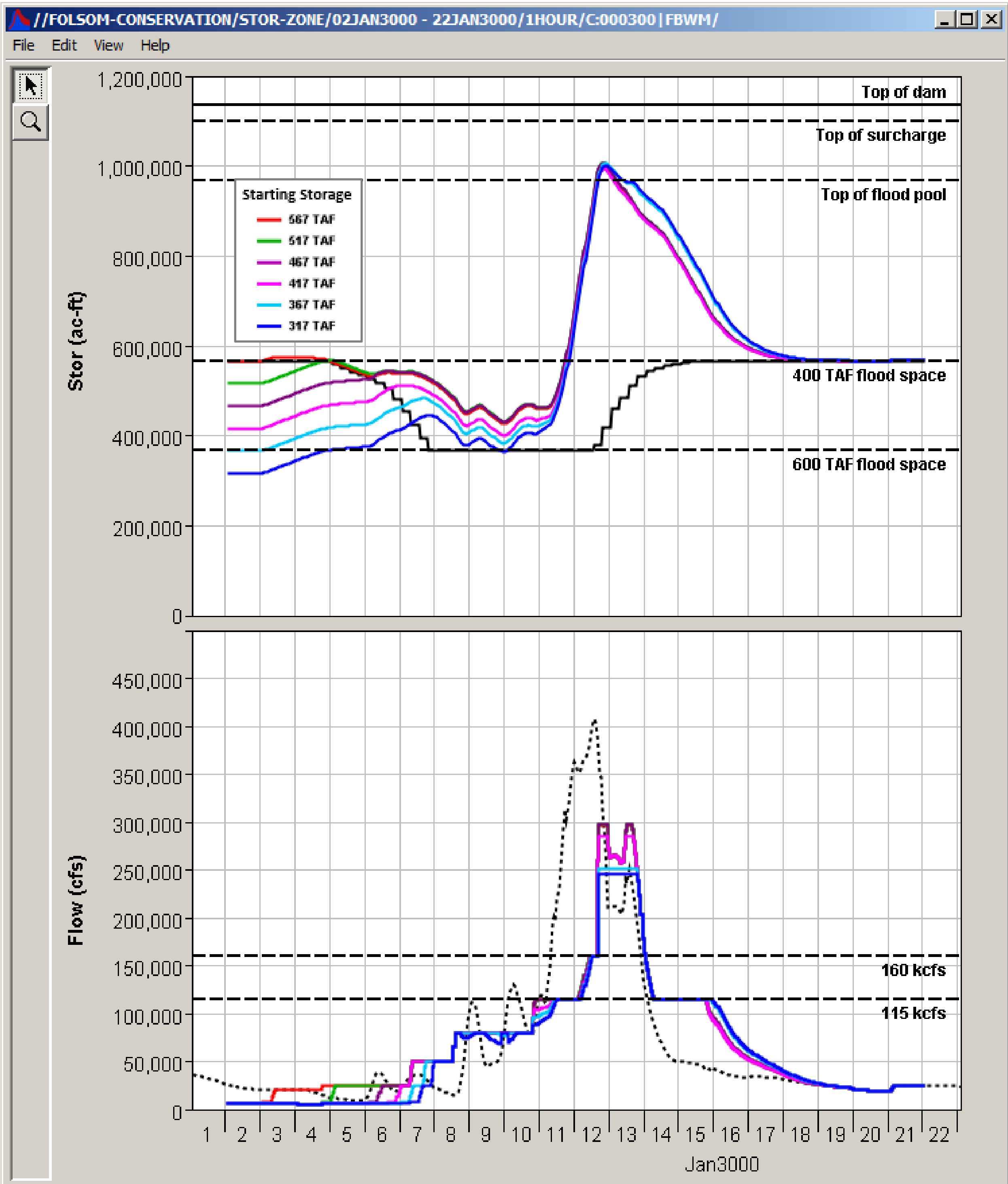


Figure G- 35: Comparison of Storage and Flow Results for the WY 1997 Event Pattern Balanced to ACE=1/300 for Selected Starting Storage Conditions

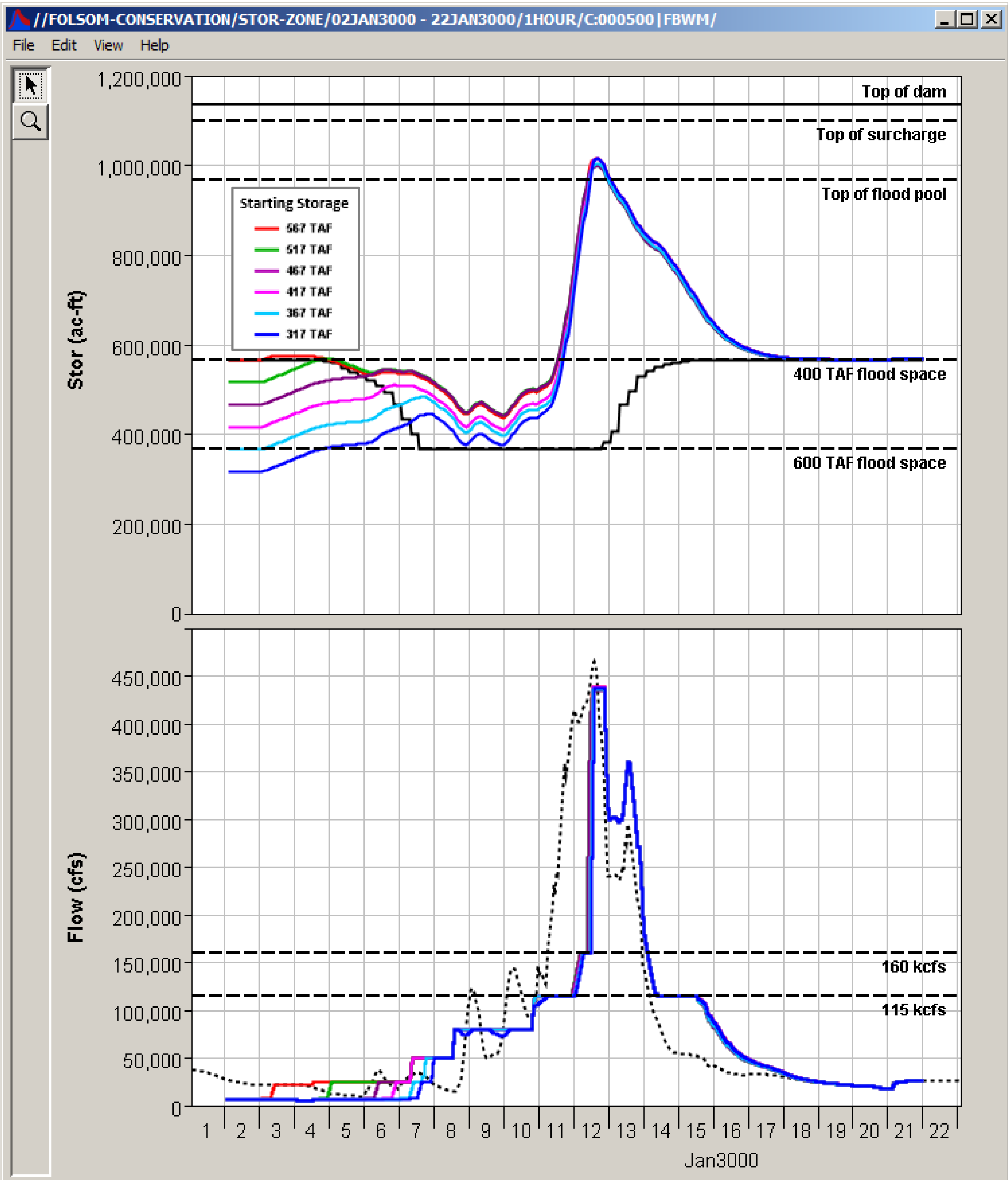


Figure G- 36: Comparison of Storage and Flow Results for the WY 1997 Event Pattern Balanced to ACE=1/500 for Selected Starting Storage Conditions

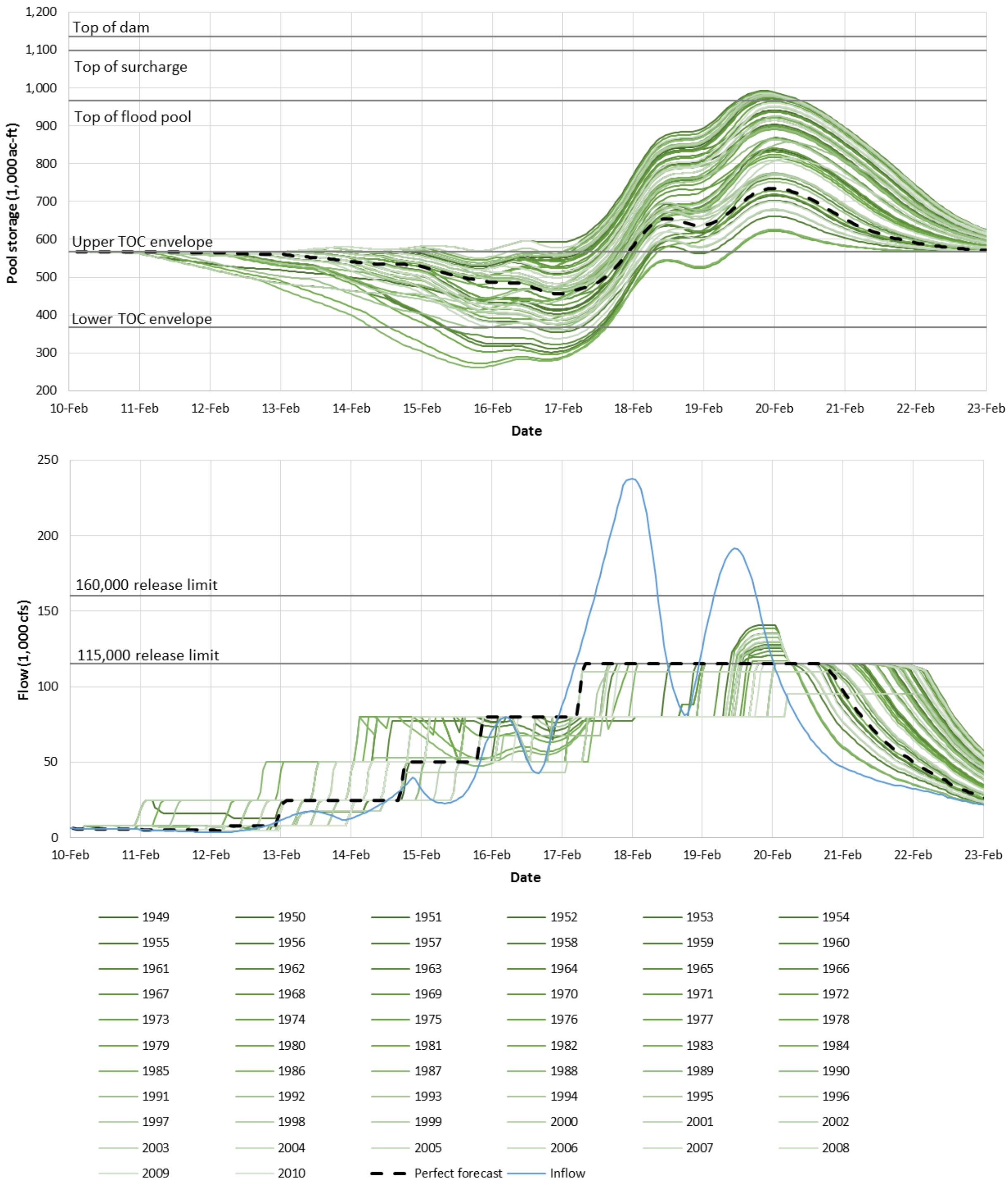


Figure G- 37: EMS Hydrographs WY 1986 pattern ACE=1/100

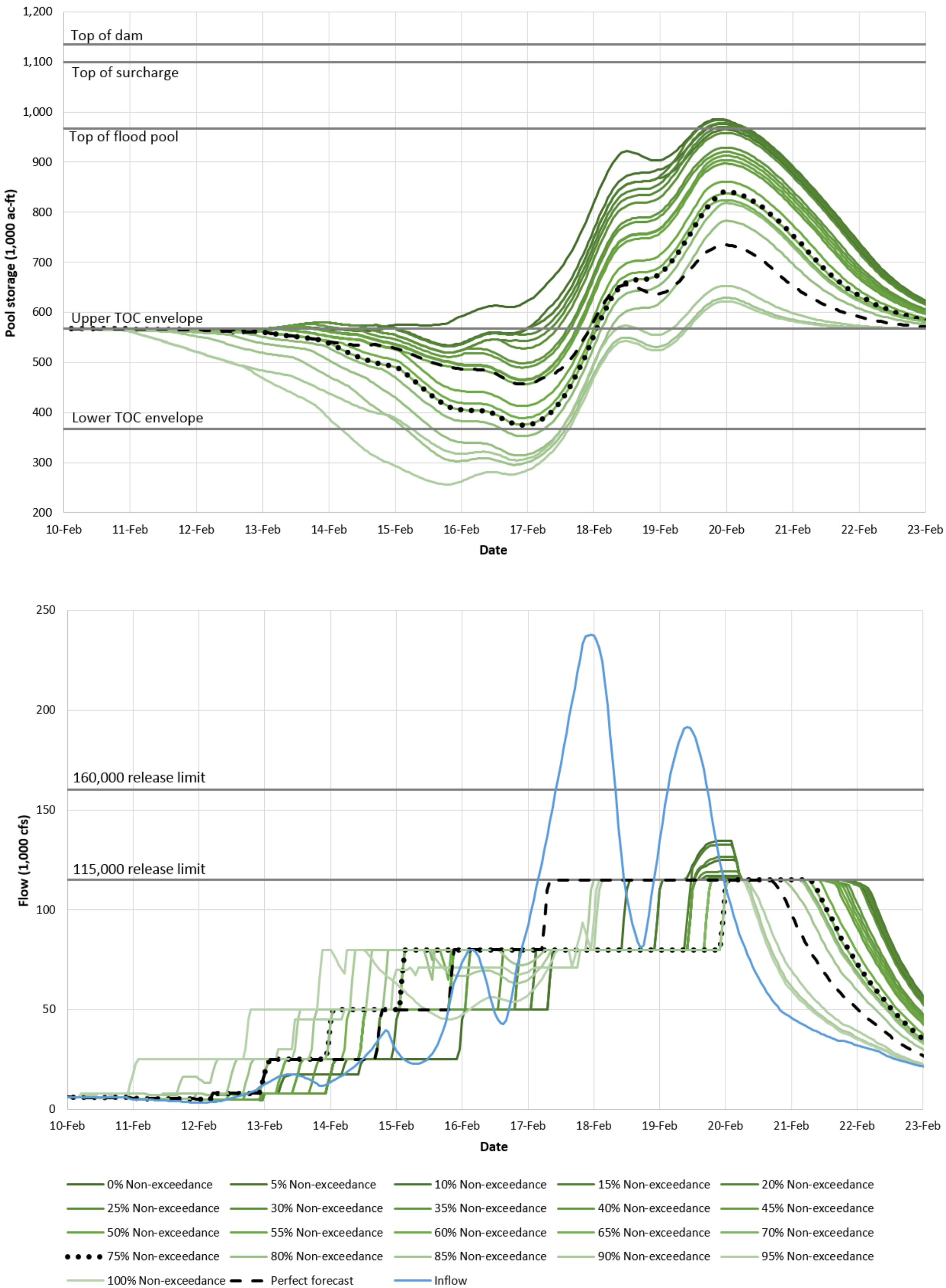


Figure G- 38: EST Hydrographs WY 1986 Pattern ACE=1/100

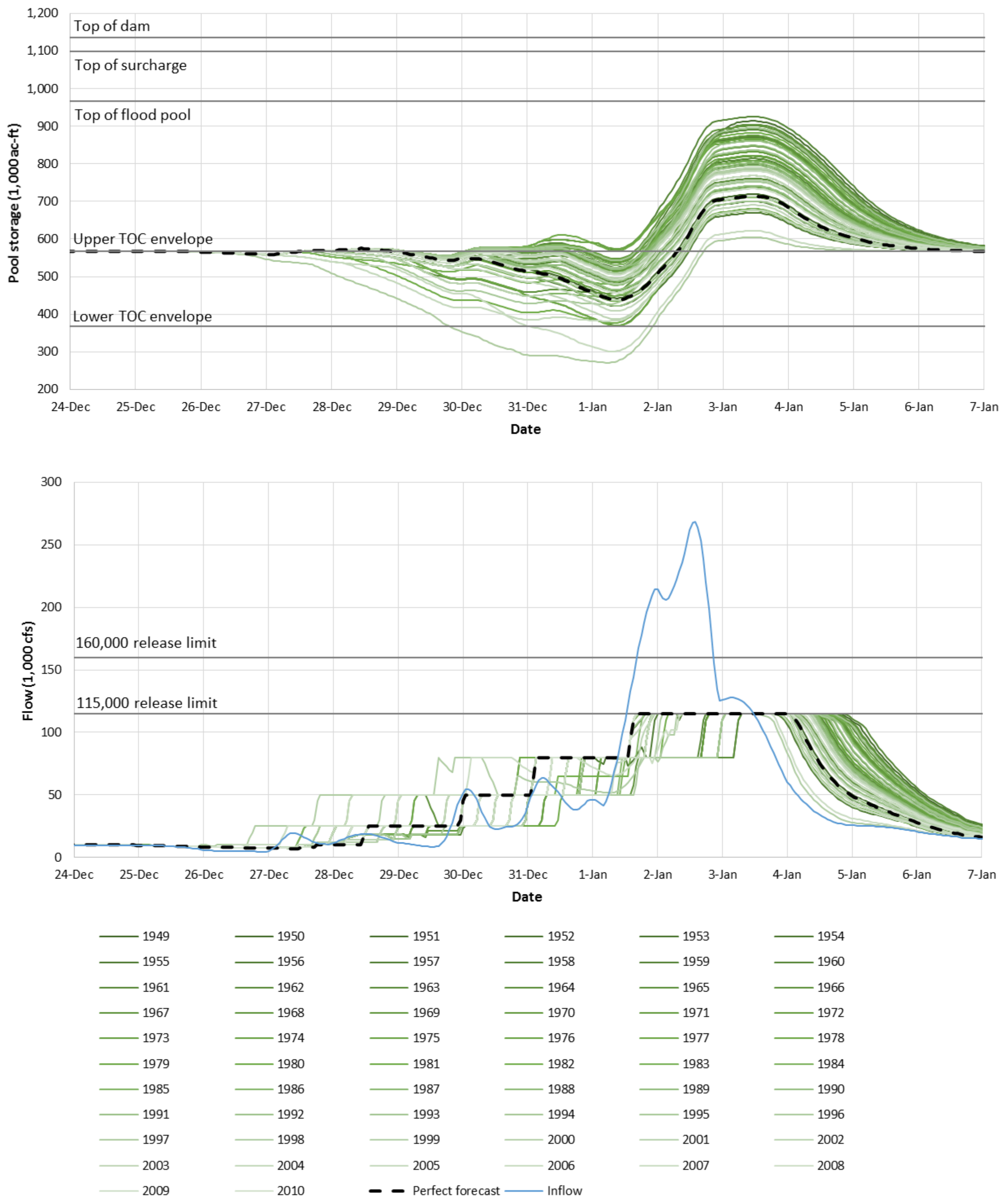


Figure G- 39: EMS Hydrographs WY 1997 Pattern ACE=1/100

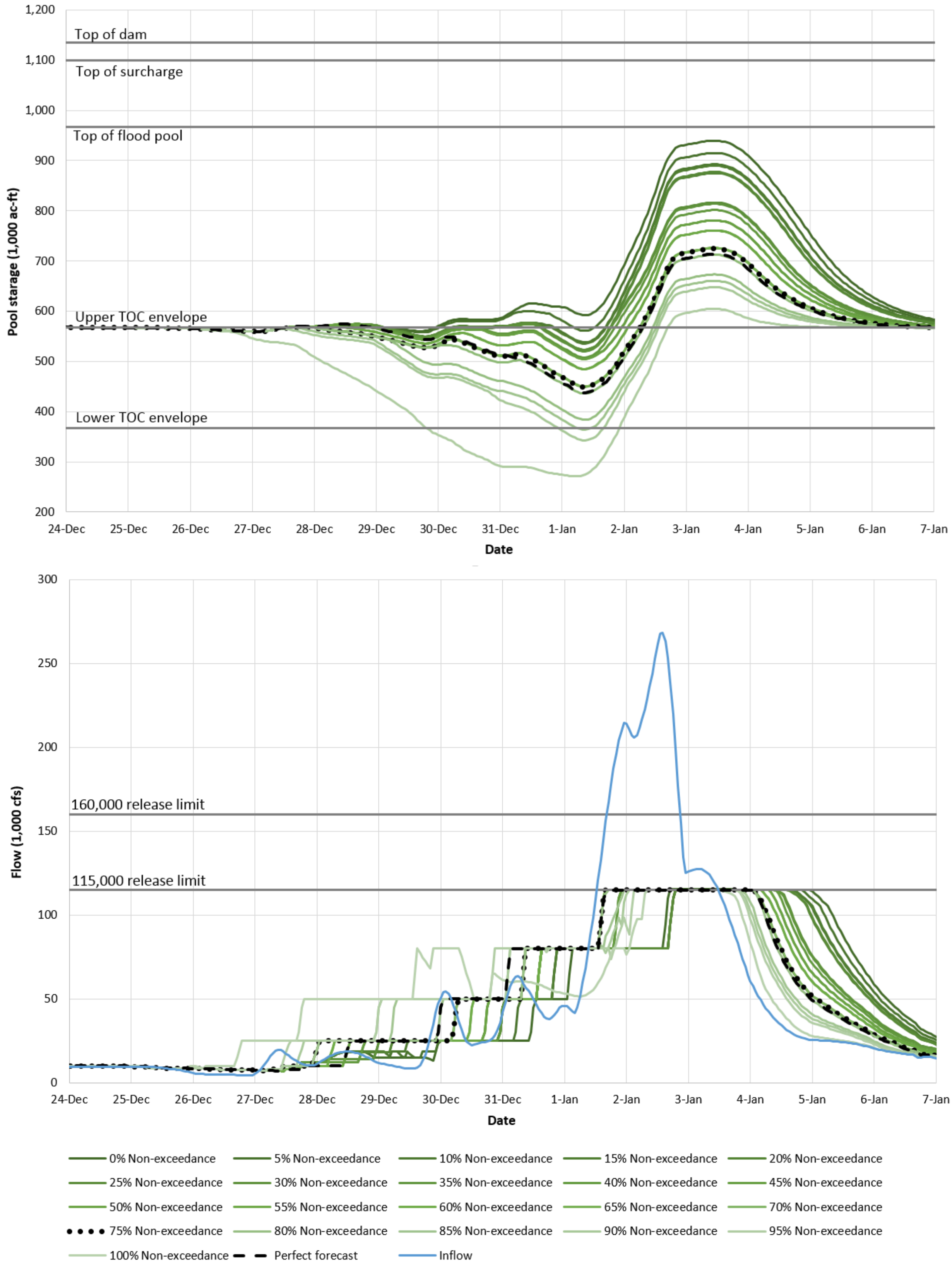


Figure G- 40: EST Hydrographs WY 1997 Pattern ACE=1/100

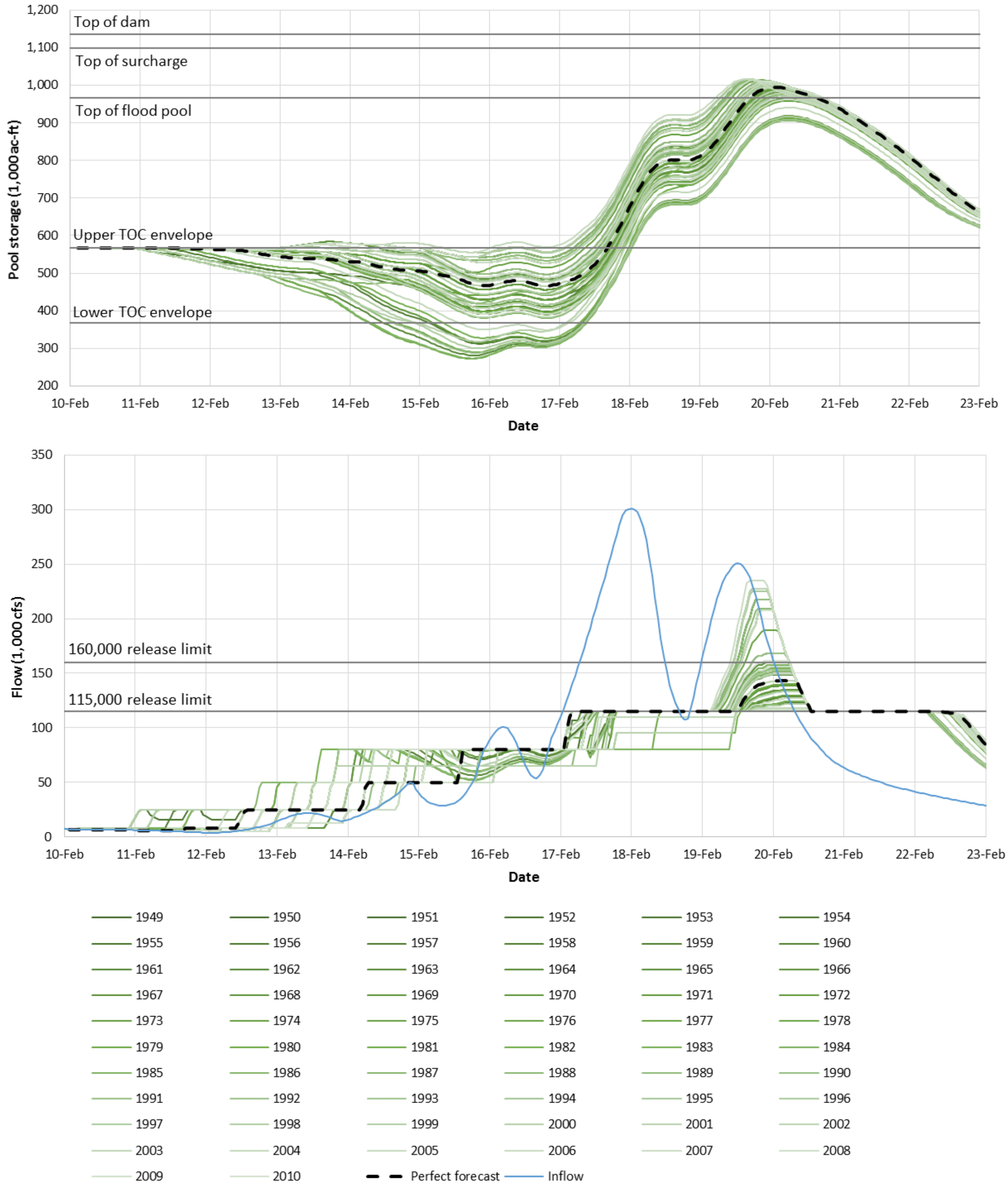


Figure G- 41: EMS Hydrographs WY 1986 Pattern ACE=1/200

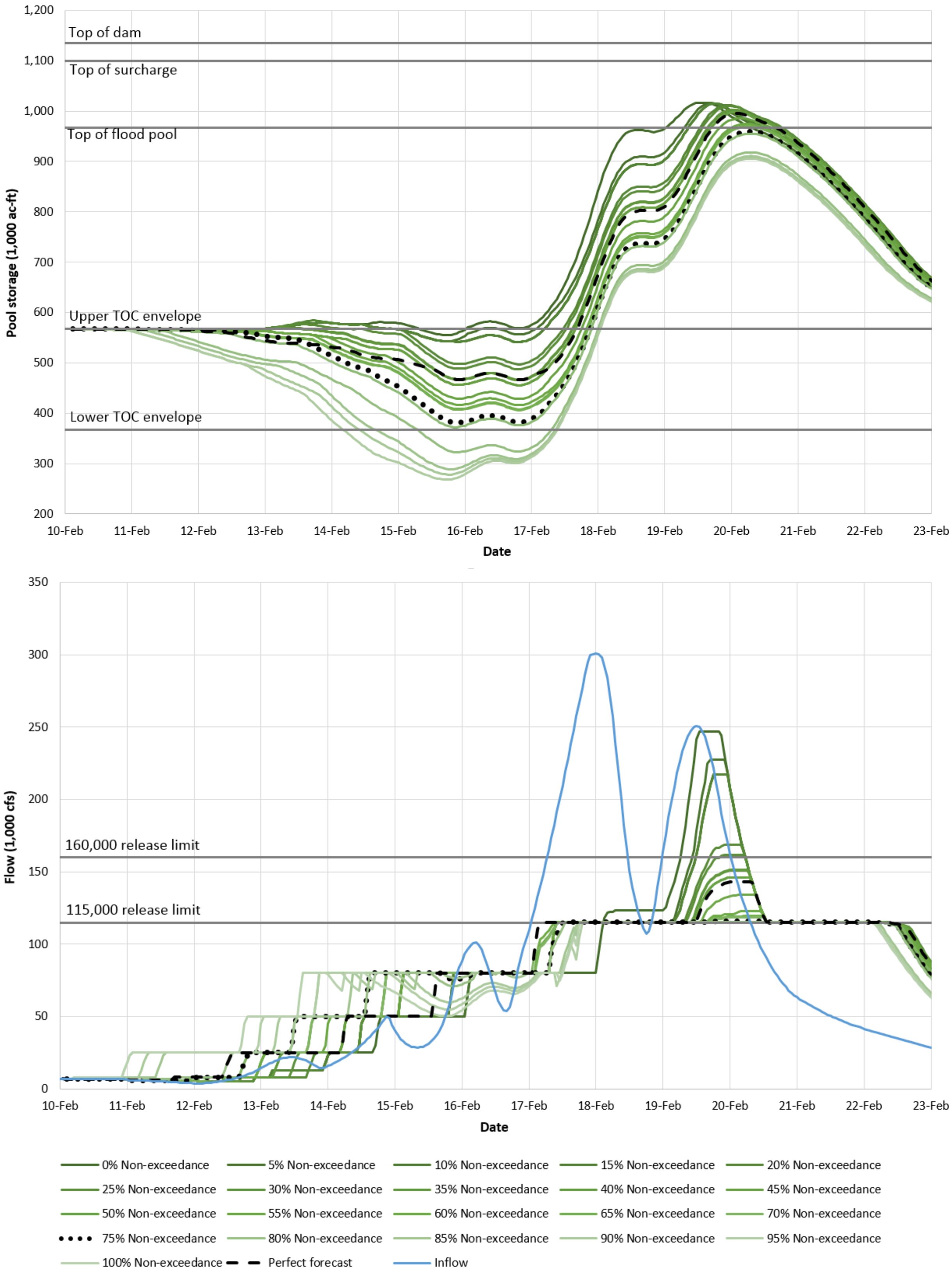


Figure G- 42: EST Hydrographs WY 1986 Pattern ACE=1/200

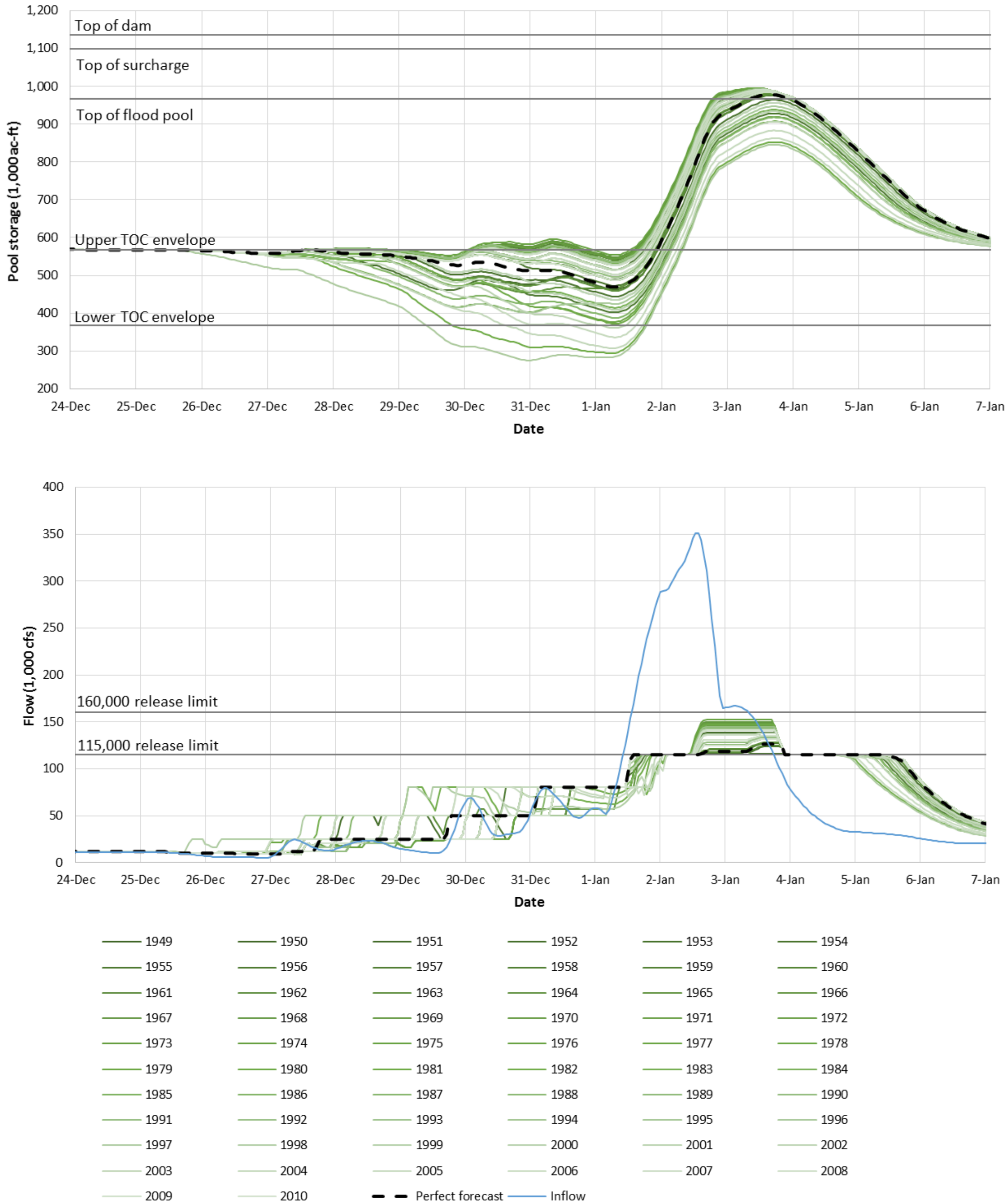
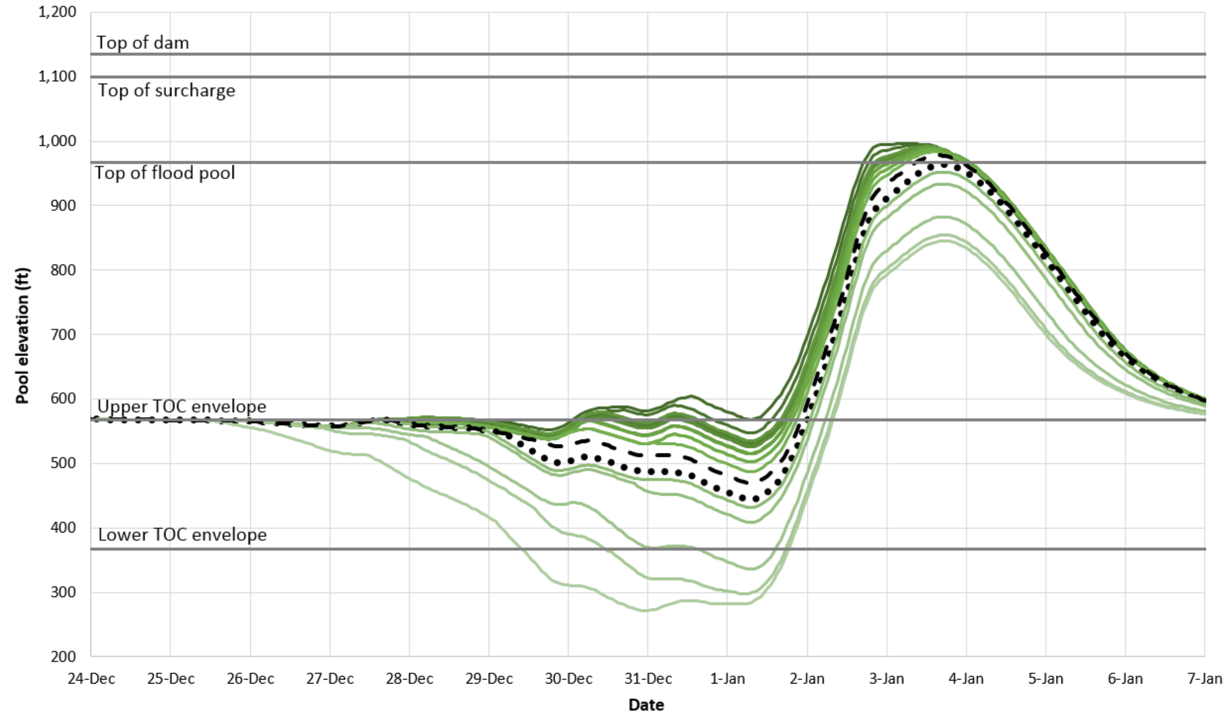


Figure G- 43: EMS Hydrographs WY 1997 Pattern ACE=1/200



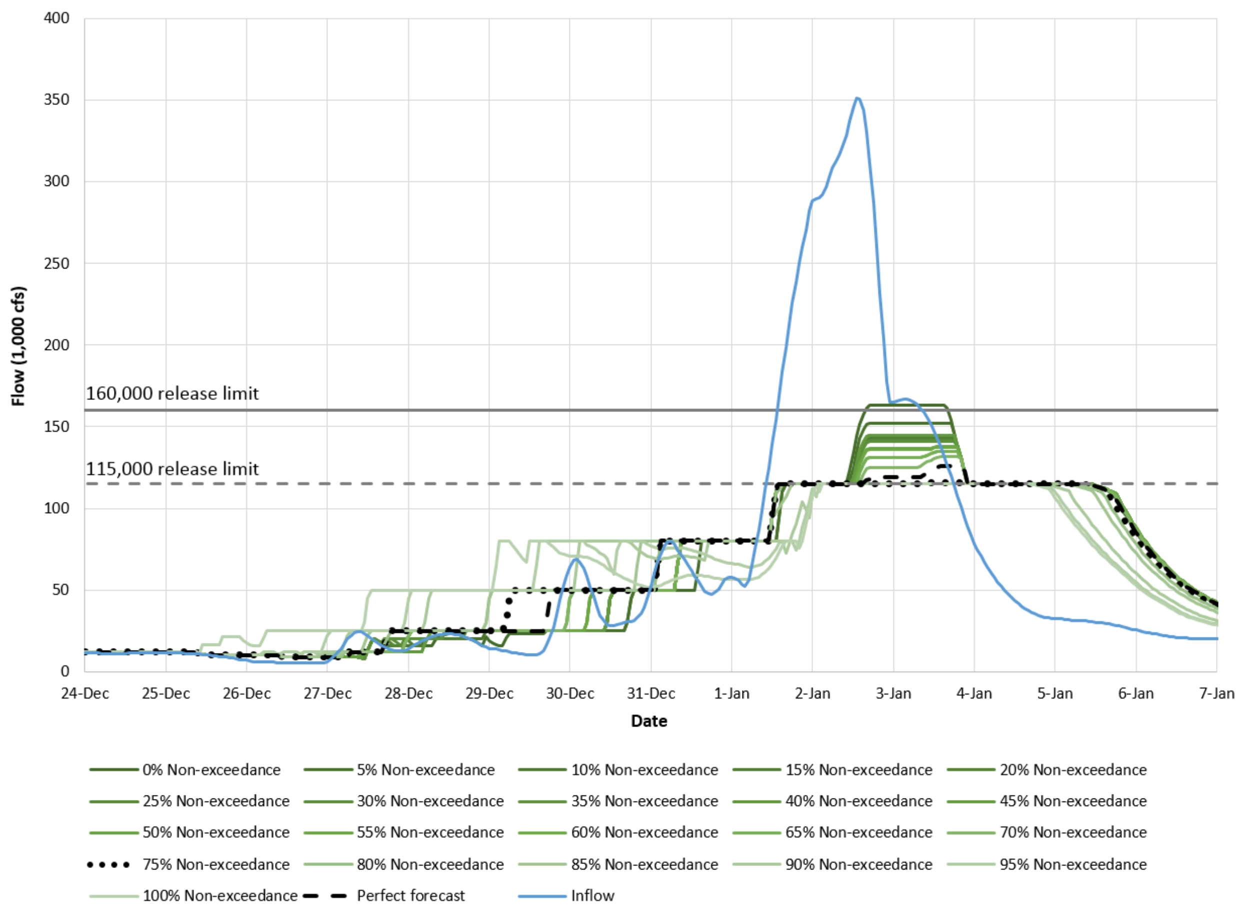


Figure G- 44: EST Hydrographs WY 1997 Pattern ACE=1/200

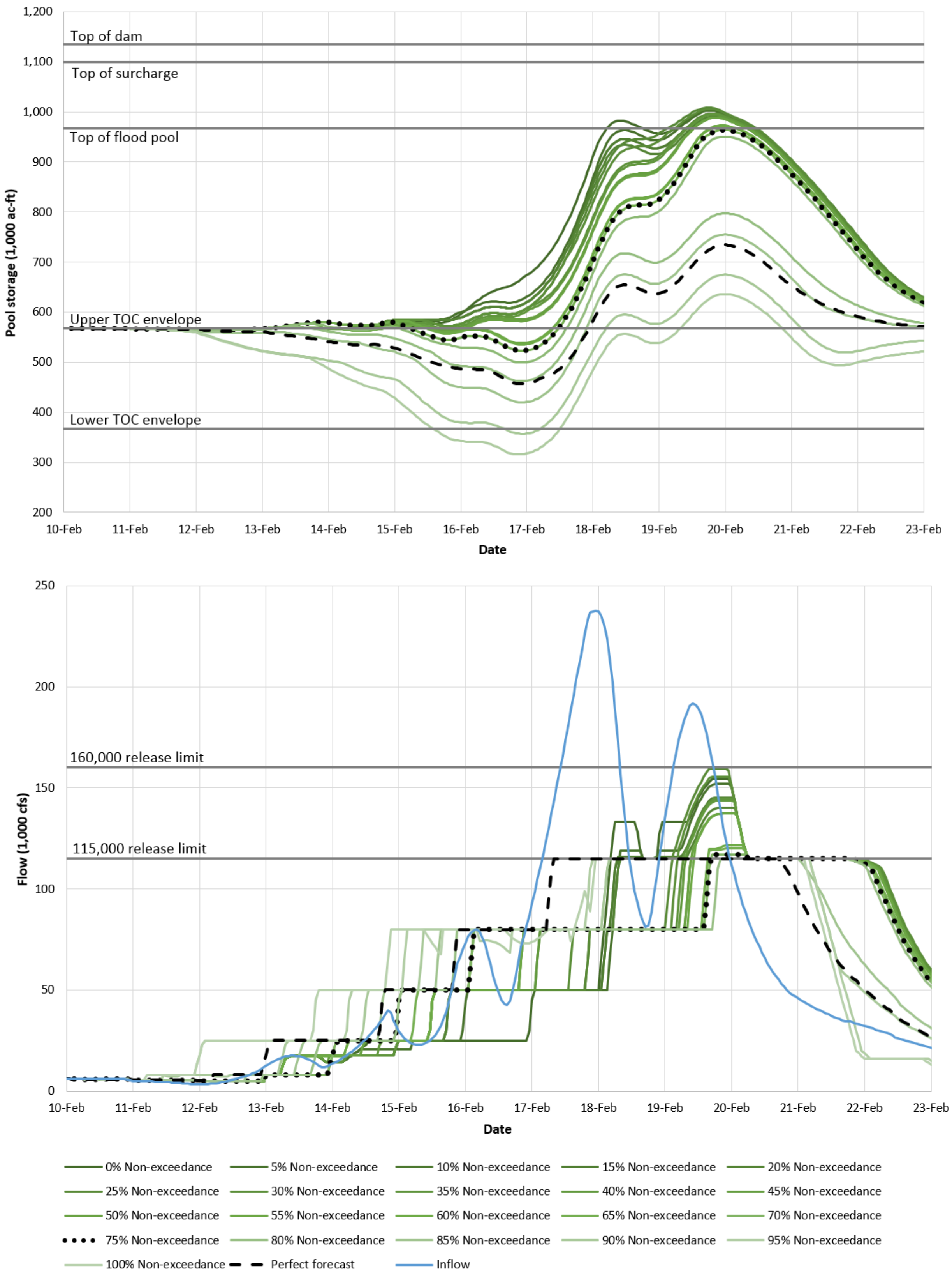


Figure G- 45: EST Hydrographs WY 1986 Pattern ACE=1/100 24-hr Late Forecast

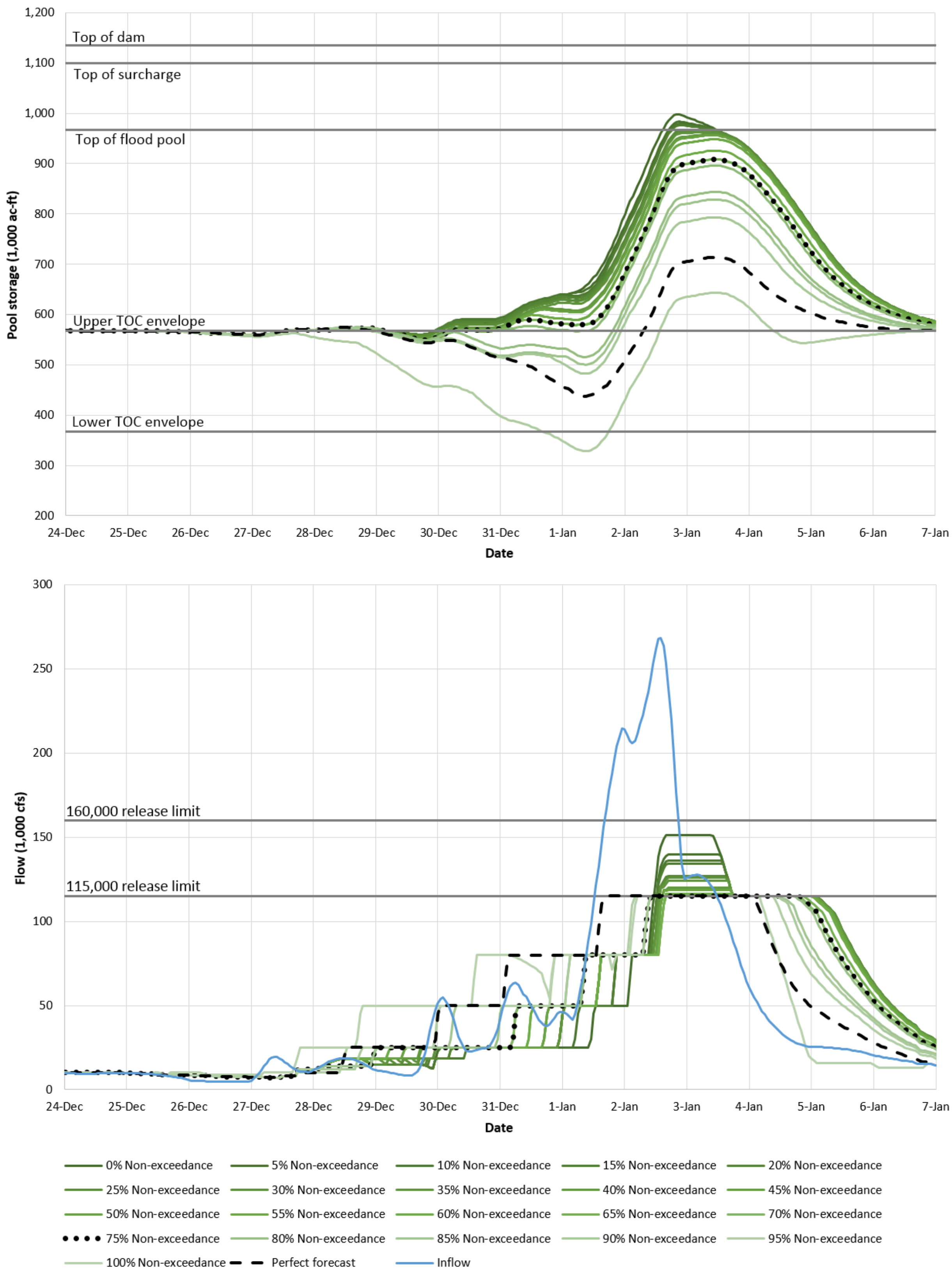


Figure G- 46: EST Hydrographs WY 1997 Pattern ACE=1/100 24-hr Late Forecast

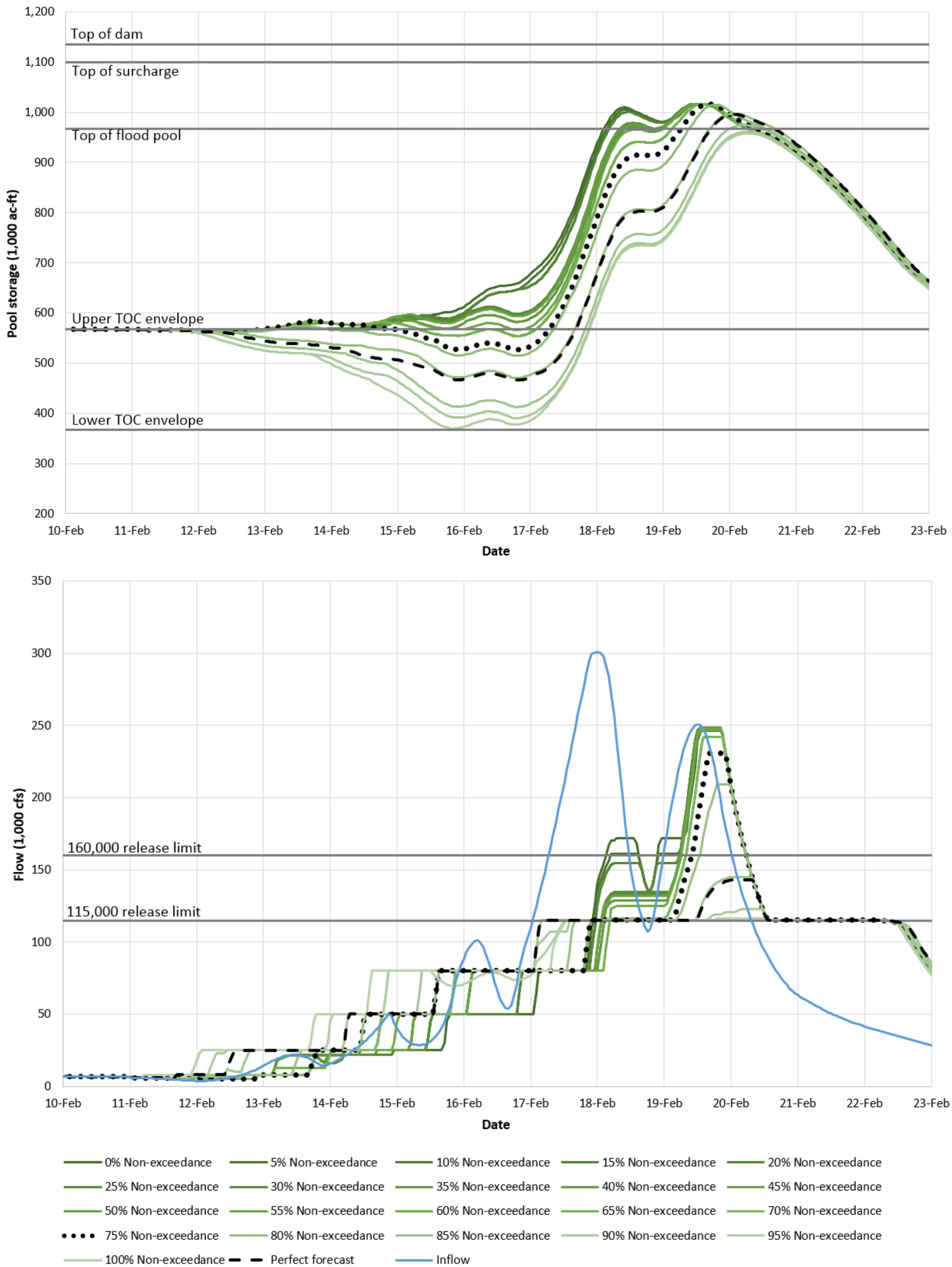


Figure G- 47: EST Hydrographs WY 1986 Pattern ACE=1/200 24-hr Late Forecast

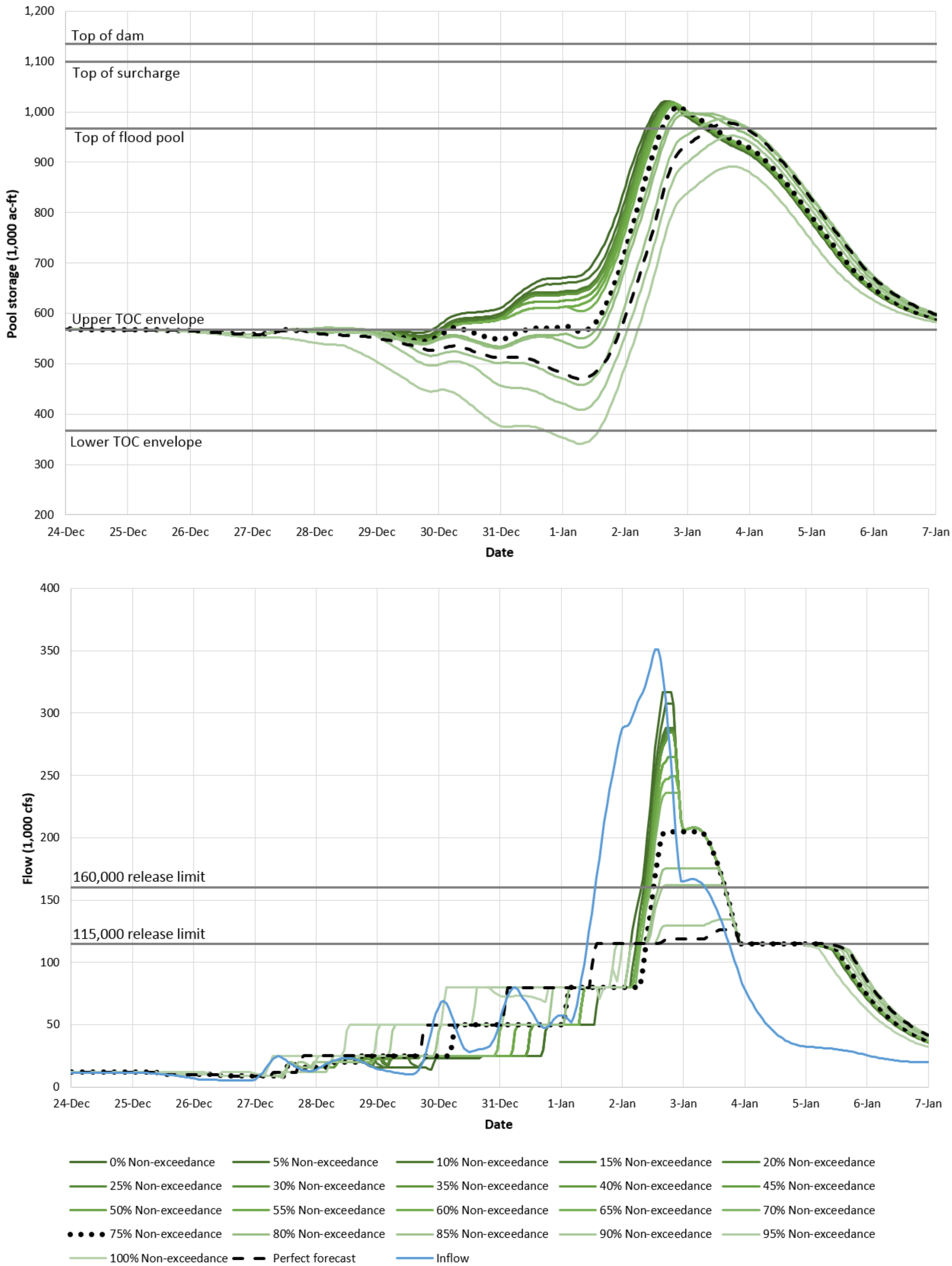


Figure G- 48: EST Hydrographs WY 1997 Pattern ACE=1/200 24-hr Late Forecast

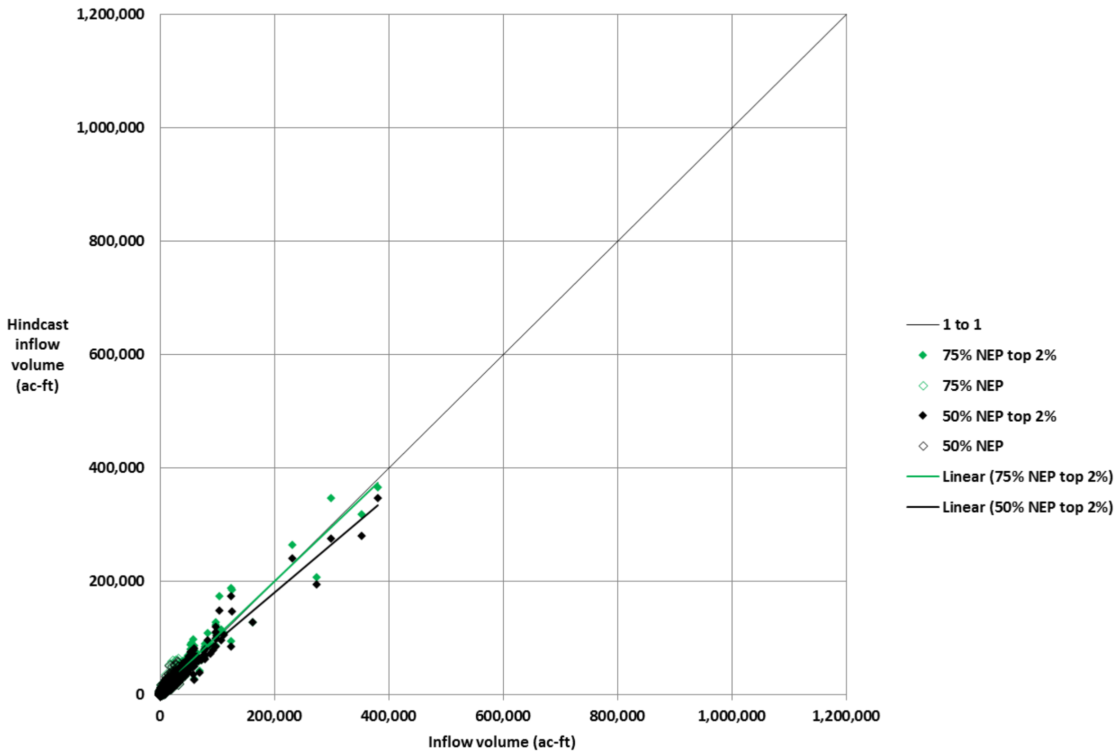


Figure G- 49: Period of Record Hindcast vs. Inflow 1-Day Volumes

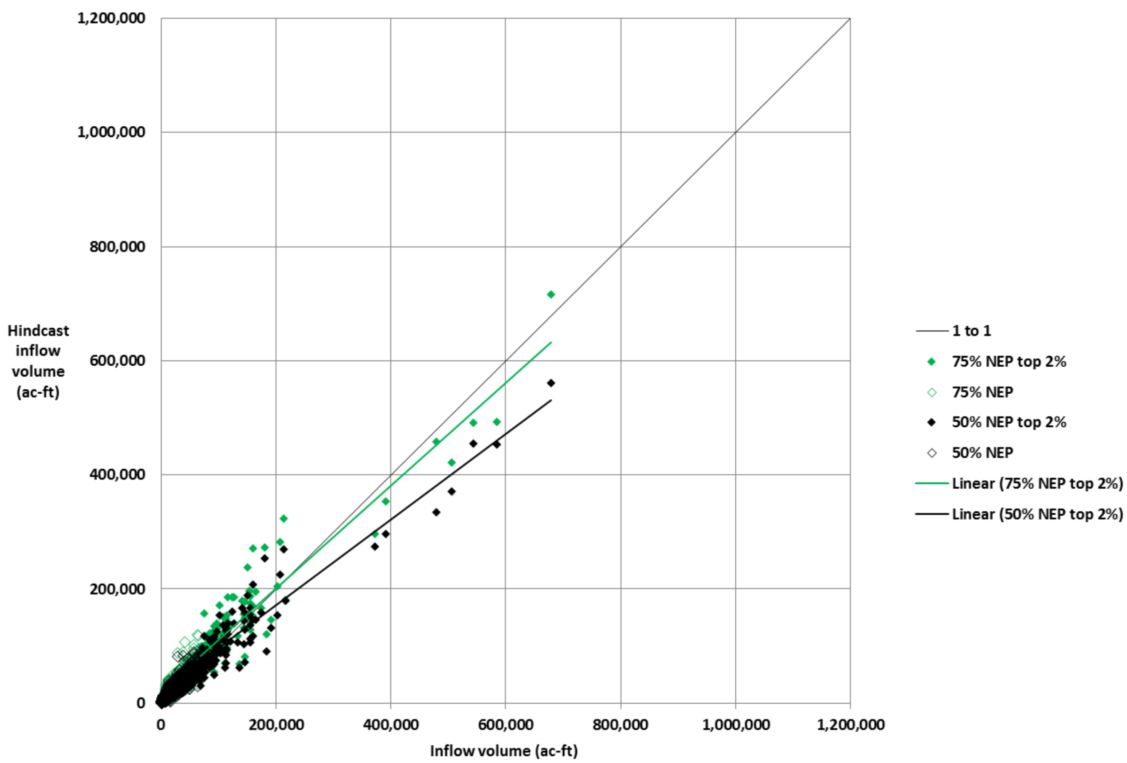


Figure G- 50: Period of Record Hindcast vs. Inflow 2-Day Volumes

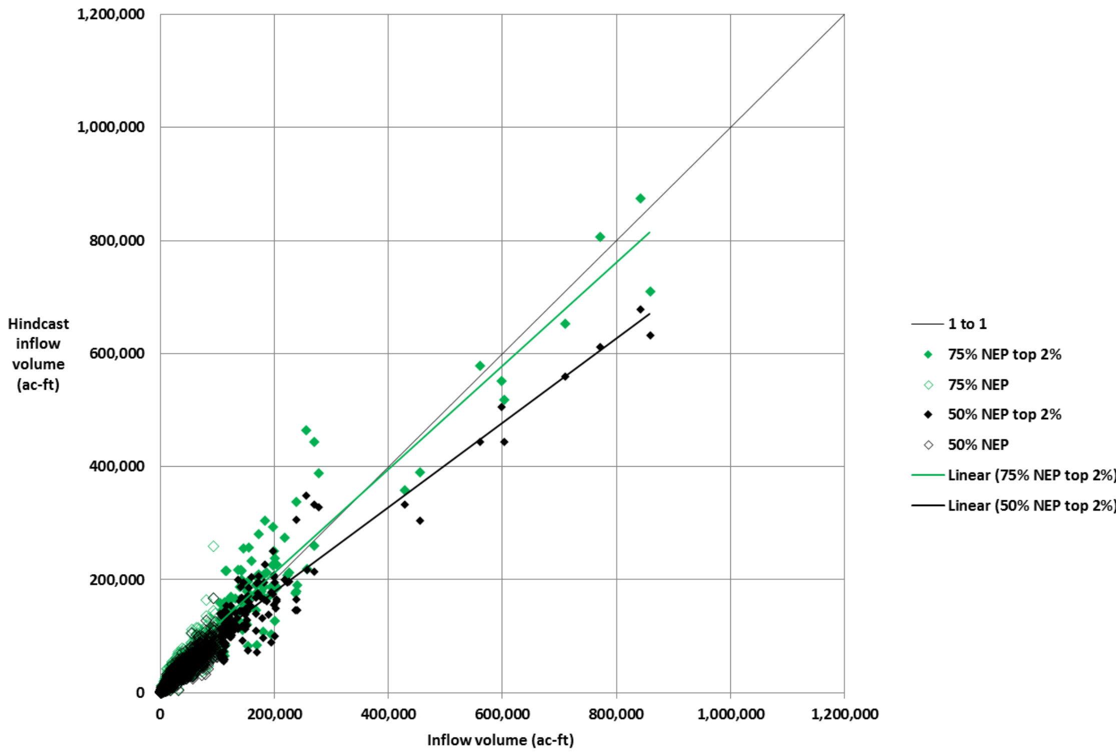


Figure G- 51: Period of Record Hindcast vs. Inflow 3-Day Volumes

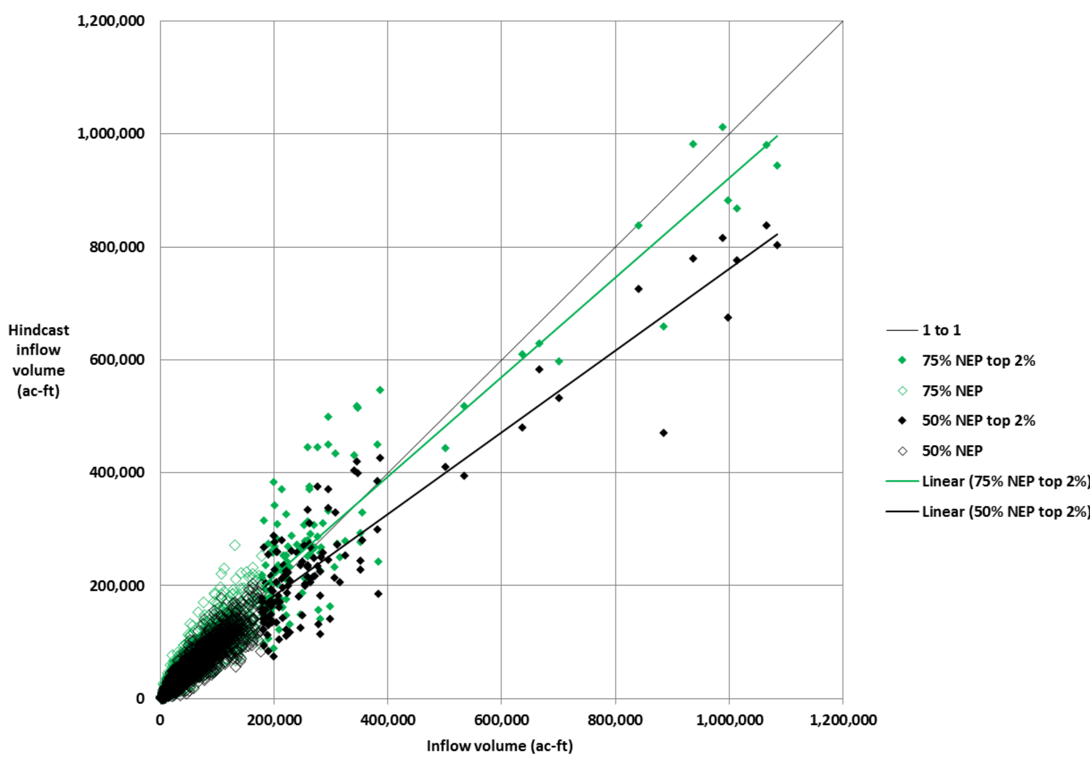


Figure G- 52: Period of Record Hindcast vs. Inflow 5-Day Volumes