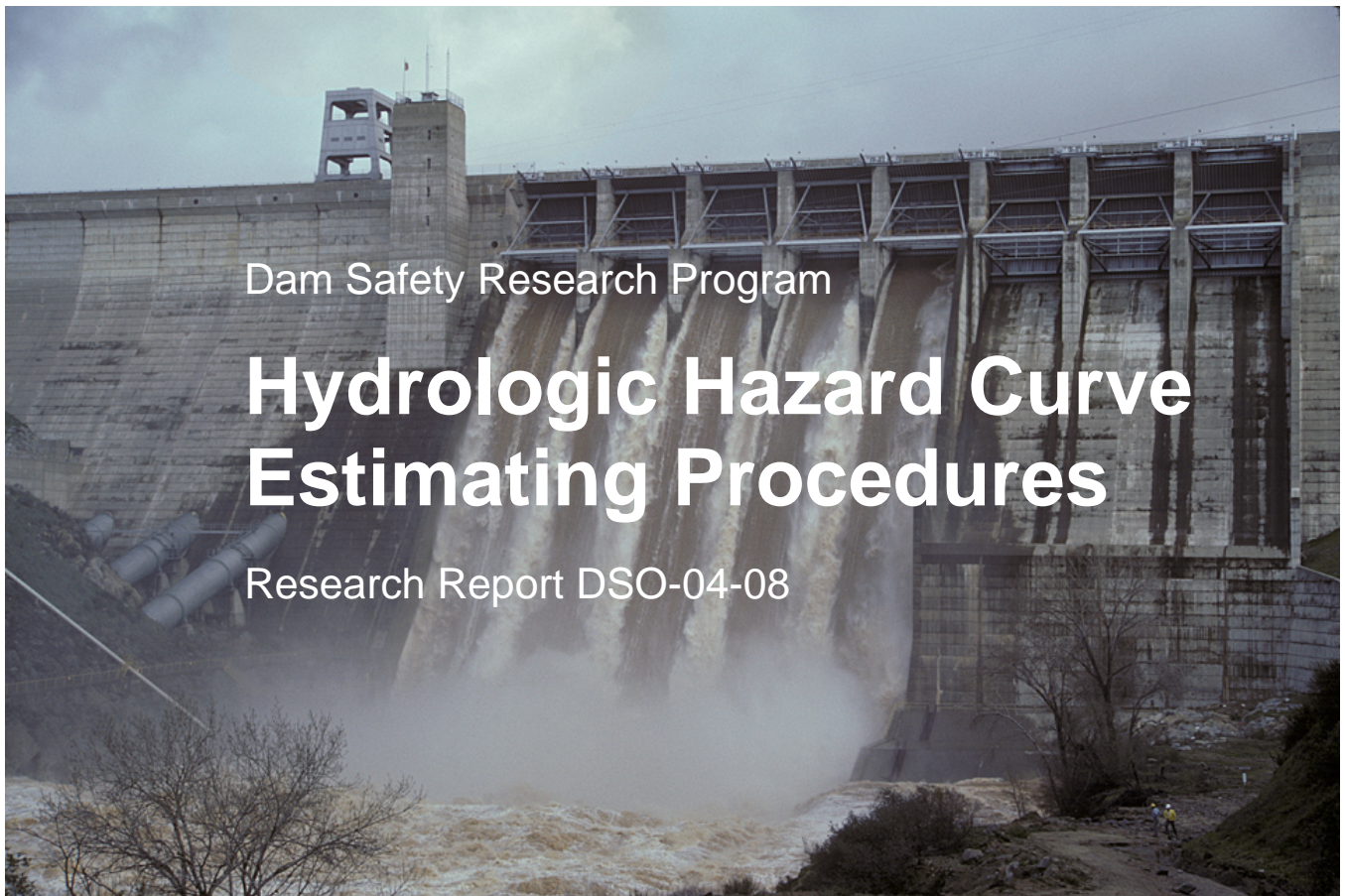


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Managing Water in the West



Dam Safety Research Program

Hydrologic Hazard Curve Estimating Procedures

Research Report DSO-04-08



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Hydrologic Hazard Curve Estimating Procedures

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**U.S. Department of the Interior
Bureau of Reclamation**

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Executive Summary

The objective of this project is to establish a prescriptive procedure for generating hydrologic hazard curves for use in dam safety evaluations. These curves can then be used for risk analysis and prioritization of further work at Bureau of Reclamation (Reclamation) dams and other U.S. Department of the Interior facilities. Hydrologic hazard curves are defined as graphs of peak flow and volume (for specified durations) versus Annual Exceedance Probability (AEP). The range of AEPs that are displayed on these graphs is from 0.99 to 0.00000001 (1 x 10⁻⁸ or 100 million years).

Reclamation has developed an approach toward developing hydrologic hazard curves for use in evaluating dam safety issues. The procedure relies on extracting information from existing studies to the fullest extent possible. The procedures and analysis techniques defined in this report allow for the possibility, and even plausibility, that peak discharge and volume estimates may exceed the probable maximum flood (PMF). This is a function of the uncertainty and inconsistency among and between analysis techniques. Therefore, in these cases, the PMF is believed to represent the upper limit to hydrologic risk.

This report recommends that the approach for developing hydrologic hazard curves consider the dam safety decision criteria, potential dam failure mode and dam characteristics, available hydrologic data, possible analysis techniques, resources available for analysis, and tolerable level of uncertainty. Dam safety decision criteria determine the probabilistic range of floods needed to address hydrologic issues. The potential dam failure mode and dam characteristics impact the type of hydrologic information needed to assess the problem. The approach chosen to answer specific hydrologic issues should consider a tolerable level of uncertainty. To reduce the uncertainty in the estimates, additional data collection and use of more sophisticated solution techniques may be required.

Reclamation currently uses a combination of seven hydrologic methods to develop hydrologic hazard curves. These general techniques include:

- Flood frequency analysis with historical/paleoflood data
- Hydrograph scaling and volumes
- The GRADEX Method
- The Australian Rainfall-Runoff Method
- Stochastic event-based precipitation runoff modeling with stochastic event flood model
- Stochastic rainfall-runoff modeling with CASC2D
- The PMF

It is believed that increasing the level of effort and sophistication of analysis technique will increase the level of confidence associated with the results.

The amount of effort expended on analyzing a hydrologic hazard depends on the nature of the problem and the potential cost of the solution. Reclamation suggests a staged approach toward

evaluating a hydrologic safety issue. Initially, very little effort is expended to determine the magnitude of the hydrologic hazard. Reclamation attempts to make use of all the available studies for the site of interest in the initial characterization. Often, the PMF study is the only hydrologic study available before the start of a probabilistic investigation. When other hydrologic studies have been performed, available data will be used to decrease uncertainty in results as well as provide an overall assessment of hydrologic risk.

Dam safety evaluations usually begin with an initial characterization of hydrologic risk. If detailed studies have been conducted for the site of interest, they are summarized, consolidated, and presented to the risk assessment team. About two-thirds of Reclamation's dams can safely accommodate the PMF; when the PMF is selected as the inflow design flood, no additional work may be required unless other hydraulic issues need evaluation. Additional hydrologic work begins with a flood frequency analysis developed for peak flows and volumes. It is believed that this type of information is sufficient to address hydrologic issues and make dam safety decisions at about 80 percent of the remaining dams. For the sites that still have potential safety problems, more sophisticated solution techniques than the initial flood frequency analysis may be required.

When planning more detailed studies, the goal is to achieve a balance between the amount of hydrologic analysis needed to address the issues and the level of effort required to conduct the study. As the studies get more detailed, the results should become more precise and contain less uncertainty.

When multiple methods are used, alternative hazard curves are developed by weighting results from the individual analyses. A team of hydrologists evaluates the alternatives and selects the one most representative for the site for use in the risk assessment. Selection of the final hydrologic hazard curve depends on the experience of the hydrologists and the assumptions that went into each analysis.

Three case studies, Los Banos, Fresno, and A.R. Bowman Dams, are presented in the report to illustrate the variety of methods available. These sites were chosen to demonstrate the use of the initial characterization of the flood hazard and more detailed followup studies, where available. The A.R. Bowman example shows how multiple studies were combined into a single flood hazard curve for use in risk assessment.

1. Introduction

The Bureau of Reclamation's (Reclamation) Dam Safety Program is seeking a procedure for developing hydrologic hazard curves for use in evaluating and prioritizing the need for dam safety modifications at Reclamation and other U.S. Department of the Interior (Interior) facilities. Hydrologic hazard curves are defined as graphs of peak flow and volume (for specified durations) versus Annual Exceedance Probability (AEP). The range of AEPs that are displayed on these graphs is from 0.99 to 0.00000001 (1×10^{-8} or 100 million years). The objective of this project is to establish a prescriptive procedure for generating hydrologic hazard curves for use in dam safety evaluations. These curves can then be used for risk analysis and prioritization of further work at Reclamation dams and other Interior facilities.

This project builds upon the Logan Workshop held in 1999 that was convened to provide a framework for Reclamation to assess flood hazards. The workshop produced the report, *A Framework for Characterizing Extreme Floods for Dam Safety Risk Assessment*. Hydrologic research has led to advances in flood estimation procedures that allow improvements to the framework. This report describes current approaches used by Reclamation to determine flood loadings for its dams.

2. Background

2.1 General

Reclamation's Dam Safety Program mission is "To ensure that Reclamation dams do not present unacceptable risks to people, property, and the environment" (Bureau of Reclamation, 1993). As the owner of over 350 storage dams in the western U.S., Reclamation is committed to providing the public and the environment with adequate protection from the risks that are inherent in collecting and storing large volumes of water. Traditional design and analysis methods have focused on selecting a level of protection based on spillway evaluation flood loadings, which were usually based on the probable maximum flood (PMF) (Bureau of Reclamation, 1999).

Since 1995, Reclamation has used a risk assessment process to determine an appropriate level of public protection by evaluating a full range of loading conditions and possible dam failure consequences. This is in contrast to the traditional approach of using upper bound events without regard to their likelihood of occurrence and without assessment of their incremental consequences. As a water resources management agency, Reclamation strives to provide decisionmakers with risk-based information founded upon current or emerging water resources management and public safety practices (Bureau of Reclamation, 1999).

Risk assessment methods provide techniques to organize and plan the data collection and technical studies necessary to evaluate dam safety issues at a site. The risk assessment process allows the risk assessment team to consider the possible adverse outcomes to a given loading condition and compute the risk associated with each possible outcome. The process involves

identifying all of the possible loading conditions, dam responses, exposure conditions, and consequences. The overall risk from the dam is the accumulation of the risks associated with each of these factors (Bureau of Reclamation, 1999).

When evaluating hydrologic hazards, a systematic means of developing flood hazard relationships is needed for risk-based assessments to determine hydrologic adequacy for Reclamation dams. The nature of the potential failure mode and characteristics of the dam and reservoir dictate the type of hydrologic information needed. For some sites, only a peak-discharge frequency analysis may be required, while at other sites, flood volumes and hydrographs may be required.

2.2 Public Protection Guidelines

Guidance for providing adequate and consistent levels of public protection in the evaluation and modification of existing dams and the design of new structures are described in the *Guidelines for Achieving Public Protection in Dam Safety Decisionmaking*, (Bureau of Reclamation, 2003). The reader may refer to the guidelines for a complete description of the assessment measures used by Reclamation in making dam safety decisions.

Determining an appropriate level of public protection involves assessing the existing risks, determining the need for risk reduction, and, where needed, evaluating specific alternatives to reduce risk. Because the total needs for the agency's financial and human resources generally exceed the available resources, the Public Protection Guidelines were prepared to assist Reclamation staff in presenting public safety information to decisionmakers for prioritizing among projects and allocating limited resources.

Reclamation's Public Protection Guidelines consist of two assessment measures of risk that are to be considered in the decision process for a dam: (1) the probability of dam failure and (2) the life loss consequences resulting from unintentional reservoir release. The annual probability of failure guideline considers the accumulation of risks from Reclamation's total inventory of dams. The life loss guideline deals with agency public trust responsibilities.

3. Process

The selected approach for developing hydrologic hazard curves for dam safety risk assessment must consider the potential dam failure mode and dam characteristics, available hydrologic data, possible analysis techniques, resources available for analysis, and tolerable level of uncertainty. The potential dam failure mode and dam characteristics impact the type of hydrologic information needed to assess the problem. Some problems may require only a peak-discharge frequency curve, while others may need complete hydrographs. The available data, possible analysis techniques, and resources available determine the approach chosen for addressing the problem.

The process that follows focuses on developing a systematic process for estimating hydrologic hazard curves that can be used for dam safety decisionmaking. It recognizes that additional studies do not always lead to better decisions. Therefore, the process relies on using existing data and previous analyses as much as possible.

3.1 Data Sources

Developing hydrologic hazard curves for risk assessment uses the length of record and type of data to determine the extrapolation limits for flood frequency analysis. Extrapolation beyond the data is often necessary to provide information needed for dam safety risk assessments. The sources of information used for flood hazard analyses include streamflow and precipitation records and paleoflood data.

Streamflow records consist of data collected at established gaging stations and indirect measurements of streamflow at other sites. Streamflow data can include estimates of peak discharge as well as average or mean discharge for various time periods. Most streamflow measurements on U.S. streams began after 1900, with only a few records dating back that far. Most often, streamflow records at a single site range in length from about 20 to 60 years. In some cases these records can be extended to about 150 years using historical information.

Precipitation and weather data used in hydrologic models can include rainfall, snowfall, snow water equivalent, temperature, solar radiation, and wind speed and direction. These data are available from various sources and vary greatly in record length and quality throughout the United States. Some of these types of data (i.e., snowfall, snow water equivalent, solar radiation, and wind) are limited to record lengths of less than about 30 years; rainfall and temperature data are available for some stations for up to 150 years, but in most cases are limited to less than 100 years.

Paleoflood hydrology is the study of past or ancient flood events which occurred before the time of human observation or direct measurement by modern hydrological procedures (Baker, 1987). Unlike historical data, paleoflood data do not involve direct human observation of the flood events. Instead, the paleoflood investigator studies geomorphic and stratigraphic records (various indicators) of past floods, as well as the evidence of past floods and streamflow derived from historical, archeological, dendrochronologic, or other sources. The advantage of paleoflood data is that it is often possible to develop records that are 10 to 100 times longer than conventional or historical records from other data sources in the western United States. Paleoflood data generally include records of the largest floods, or commonly, the limits on the stages of the largest floods over long time periods.

3.2 Flood Frequency Extrapolation

The type of data and the record length used in the analysis form the primary basis for establishing a limit on credible extrapolation of flood estimates. The data used in the analysis provide the only basis for verification of the analysis or modeling results, and as such, extensions beyond the data cannot be verified. The greatest gains to be made in providing credible estimates of extreme floods can be achieved by combining regional data from multiple sources.

Thus, analysis approaches that pool data and information from regional precipitation, regional streamflow, and regional paleoflood sources should provide the highest assurance of credible characterization of low AEP floods. The information that follows was developed in a workshop sponsored by Reclamation and documented in Bureau of Reclamation, 1999.

For Reclamation dam safety risk assessments, flood estimates are needed for AEPs of 1 in 10,000 and ranging down to 1 in 100,000,000. Developing credible estimates at these low AEPs generally requires combining data from multiple sources and a regional approach. Table 3-1 lists the different types of data that can be used as a basis for flood frequency estimates and the typical and optimal limits of credible extrapolation for AEP (Bureau of Reclamation, 1999). In general, the optimal limits are based on the best combination(s) of data envisioned in the western U.S. in the foreseeable future. Typical limits are based on the combination(s) of data that are commonly available and analyzed for most sites.

Table 3-1.—Hydrometeorological data types and extrapolation limits for flood frequency analysis (Bureau of Reclamation, 1999)

Type of data used for flood frequency analysis	Limit of credible extrapolation for annual exceedance probability	
	Typical	Optimal
At-site streamflow data	1 in 100	1 in 200
Regional streamflow data	1 in 500	1 in 1,000
At-site streamflow and at-site paleoflood data	1 in 4,000	1 in 10,000
Regional precipitation data	1 in 2,000	1 in 10,000
Regional streamflow and regional paleoflood data	1 in 15,000	1 in 40,000
Combinations of regional data sets and extrapolation	1 in 40,000	1 in 100,000

Many factors can affect the equivalent independent record length for the optimal case. For example, gaged streamflow records in the western United States only rarely exceed 100 years, and extrapolation beyond twice the length of record, or to about 1 in 200 AEP, is generally not recommended (Interagency Advisory Committee on Water Data [IACWD], 1982). Likewise, for regional streamflow data the optimal limit of credible extrapolation is established at 1 in 1,000 AEP by considering the number of stations in the region, lengths of record, and degree of independence of these data (Hosking and Wallis, 1997). For paleoflood data, only in the Holocene epoch (or the past 10,000 years) is climate judged to be sufficiently like that of the present climate for these types of records to have meaning in estimating extreme floods for dam safety risk assessment. This climatic constraint indicates that an optimal limit for extrapolation from paleoflood data, when combined with at-site gaged data, for a single stream should be about 1 in 10,000 AEP. For regional precipitation data, a similar limit is imposed because of the difficulty in collecting sufficient station-years of clearly independent precipitation records in the orographically complex regions of the western United States. Combined data sets of regional gaged and regional paleoflood data can be extended to smaller AEPs, perhaps to about 1 in

40,000, in regions with abundant paleoflood data. Analysis approaches that combine all types of data are judged to be capable of providing credible estimates to an AEP limit of about 1 in 100,000 under optimal conditions.

In many situations, credible extrapolation limits may be less than optimal. Typical limits would need to reflect the practical constraints on the equivalent independent record length that apply for a particular location. For example, many at-site streamflow record lengths are shorter than 100 years. If in a typical situation the record length is only 50 years, then the limit of credible extrapolation might be an AEP of about 1 in 100. Similarly, many paleoflood records do not extend to 10,000 years, and extensive regional paleoflood data sets do not currently exist. Using a record length of about 4,000 years, a typical limit of credible extrapolation might be an AEP of 1 in 15,000 based on regional streamflow and regional paleoflood data.

The information presented in table 3-1 is intended as a guide; each situation is different and should be assessed individually. The limits of extrapolation should be determined by evaluating the lengths of records, number of stations in a hydrologically homogeneous region, degree of correlation between stations, and other data characteristics that may affect the accuracy of the data.

Ideally, one would like to construct the flood frequency distribution for all floods that could conceivably occur. However, the limits of data and flood experience for any site or region place practical limits on the range of the floods to which AEPs can be assigned. In general, the scientific limit to which the flood frequency relationship can be credibly extended, based upon any characteristics of the data and the record length, will fall short of the PMF for a site. However, there is a need in dam safety risk assessment to determine the probability of occurrence of very large floods with very small AEPs.

Floods can be categorized, according to the Australian Rainfall and Runoff: A Guide to Flood Estimation (Nathan and Weinmann, 2001), as large, rare, and extreme. These flood categories are shown in figure 3-1. Large floods generally encompass events for which direct observations and measurements are available. Rare floods represent events located in the range between direct observations and the credible limit of extrapolation from the data. Extreme floods generally have very small AEPs, which are beyond the credible limit of extrapolation but are still needed for dam safety risk assessments. Occasionally, Reclamation has an interest in floods with an AEP as low as 1 in 108.

Extreme floods border on the unknowable. Uncertainty is very large and unquantifiable. Since data cannot support flood estimates in this AEP range, hydrologists and engineers must rely on our knowledge and understanding of hydrologic processes to estimate extreme floods. Oftentimes, these floods may result from unforeseen and unusual combinations of hydrologic parameters generally not represented in the flood history at a particular location. One potential upper bound to the largest flood at a particular site of interest is the PMF.

Reclamation uses the PMF as the upper limit of flood potential at a site for storm durations defined by the probable maximum precipitation (PMP). If peak flows or volumes calculated using probability or statistically based hydrology methods exceed those of the PMF, then the

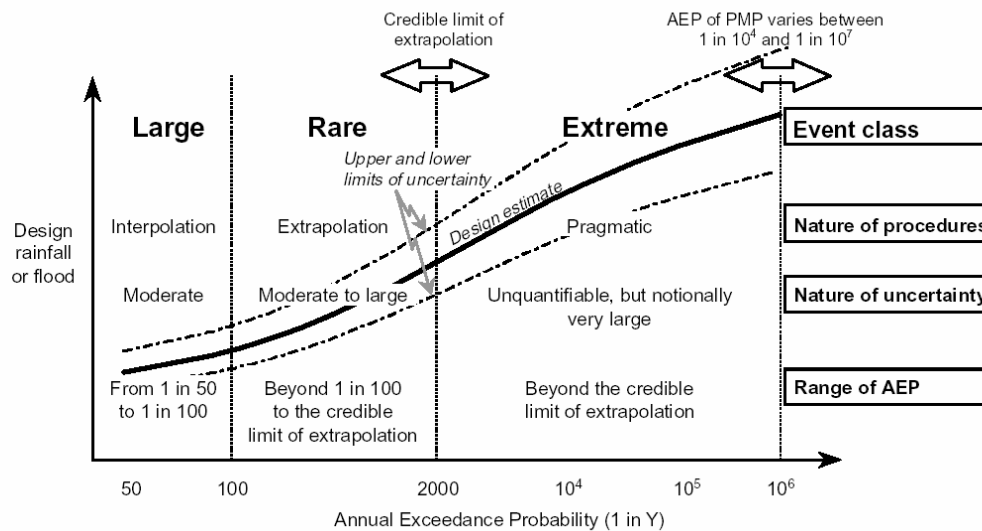


Figure 3-1.—Characteristics of notional floods (Nathan and Weinmann, 2001).

PMF is used in evaluating the hydrologic risk and as a theoretical and practical upper limit to statistical extrapolations. The PMF is defined as “the maximum runoff condition resulting from the most severe combination of hydrologic and meteorological conditions that are considered reasonably possible for the drainage basin under study” (Cudworth, 1989). If the PMF has been properly developed, it represents the upper limit to runoff that can physically occur at a particular site. Various storm types, sequences, and durations are taken together with the most severe hydrologic parameters in its development. Extrapolation of statistical analyses can become unbounded for flood distributions that exhibit positive skewness; therefore, Reclamation uses the PMF to limit extrapolation to flood discharges that are physically possible.

3.3 Flood Peak and Volume Relationships

Hydrologic hazard relationships display peak flow and flood volumes for various durations versus AEP. Figure 3-2 is a hypothetical example of the type of relationship needed to address hydrologic dam safety issues. Floods with AEPs as low as $1 \text{ in } 10^8$ are desired to encompass the full range of events needed for dam safety risk assessment. The next section of this report describes the potential approaches for developing the flood peak and volume relationship. Some of the approaches will also produce flood hydrographs, which can be routed through the reservoir.

4. Analysis Techniques

The main probabilistic and engineering hydrology methods that are currently being used, applied, and under investigation by the Flood Hydrology Group are summarized in this section of the report. There are seven general techniques:

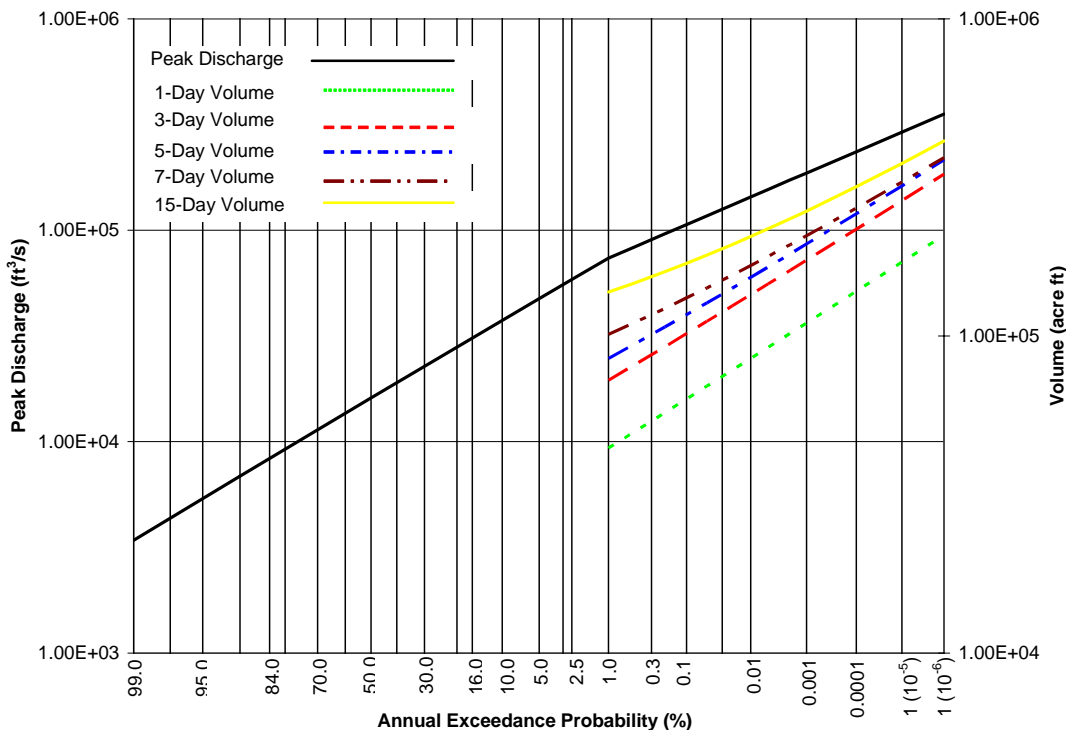


Figure 3-2.—Example hydrologic hazard curve.

- Flood frequency analysis with historical/paleoflood data
- Hydrograph scaling and volumes
- The GRADEX Method
- The Australian Rainfall-Runoff Method
- Stochastic event-based precipitation runoff modeling with the stochastic event flood model (SEFM)
- Stochastic rainfall-runoff modeling with CASC2D
- The PMF

Other models and approaches are briefly noted by reference in each section. General sources of models and approaches for estimating extreme floods are listed in Maidment (1993), Singh (1995), and Bureau of Reclamation (1999). Methods to calculate extreme floods and associated probabilities have recently been revised and published in the United Kingdom (Institute of Hydrology, 1999) and Australia (Nathan and Weinmann, 2001).

4.1 Flood Frequency Analysis with Historical/Paleoflood Data

There are three main techniques that Reclamation currently uses to develop a peak-flow frequency curve and integrate streamflow (gage) data, historical data, and paleoflood data. The first is a mixed-population graphical approach (England et al., 2001). The two other techniques are statistical models that use gage, historical, and paleoflood data. The Expected Moments Algorithm (EMA) (England, 1999) uses moments to estimate the parameters of a log-Pearson Type III (LP-III) distribution and is consistent with Bulletin 17B (IACWD, 1982). A Bayesian maximum likelihood approach is used by FLDFRQ3 (O'Connell, 1999) to estimate a peak-flow frequency curve with historical and paleoflood data and uncertainties. All three techniques have been used for estimating flood peaks at various Reclamation dams.

4.1.1 Historical and Paleoflood Data

Many different kinds of historical and paleoflood data can be used for flood frequency analysis. Historical flood data are typically extreme floods that have occurred and were described in some qualitative or quantitative fashion before establishing a stream gaging station. The typical information that is available for historical floods is the date of occurrence and the height of the water surface (Thomson et al., 1964). In many cases, people physically mark, on a relatively permanent surface, the approximate high-water mark of a flood (Thomson et al., 1964; Leese, 1973; Natural Environment Research Council, 1975; Sutcliffe, 1987; Fanok and Wohl, 1997).

Paleoflood hydrology is the study of past or ancient floods that occurred before the time of human observation or direct measurement by modern hydrologic procedures (Baker, 1987). The basic types of paleoflood indicators that are useful for flood frequency analysis are paleostage indicators and botanical evidence (Wohl and Enzel, 1995; Baker, 2000). Recent investigations, techniques, and analyses for collecting and using paleoflood data are discussed in House et al. (2002). Fluvial geomorphic evidence includes erosional and/or depositional features that are used to infer paleostages or non-inundation levels. The fluvial geomorphic evidence used in paleoflood and flood frequency studies that represents paleostage indicators includes: silt lines, scour lines, slackwater deposits, boulder and gravel bars, and modified geomorphic surfaces (Costa, 1978; Baker, 1987; Kochel and Ritter, 1987; Jarrett and Costa, 1988; Salas et al., 1994; Jarrett and England, 2002; Levish, 2002). Botanical evidence consists of vegetation that records evidence of a flood (or several floods) or indicates stability of a geomorphic surface for some time period. Botanical evidence of floods includes: corrosion scars, adventitious sprouts, tree age, and tree-ring anomalies (Hupp, 1987).

The historical and paleoflood data can generally be represented with four major data classes (Stedinger et al., 1988): floods of known magnitude, floods of unknown magnitude that are less than some level, floods of unknown magnitude that exceed some level, and floods with magnitudes described by a range. Historical and paleoflood data generally are described in terms of exceedance or non-exceedance of a discharge threshold (Q_0). To correctly interpret the data, one needs to understand the mechanisms by which historical and paleoflood records document the magnitudes of floods that either did, or did not, occur (Stedinger et al., 1993). In many situations, one knows the magnitude of each flood. Annual (gage) peak discharge estimates,

historical floods, and paleofloods whose magnitudes are known are described by “floods of known magnitude” class. For example, the solid bars depicted in figure 4-1 describe known floods in the gage and historical period.

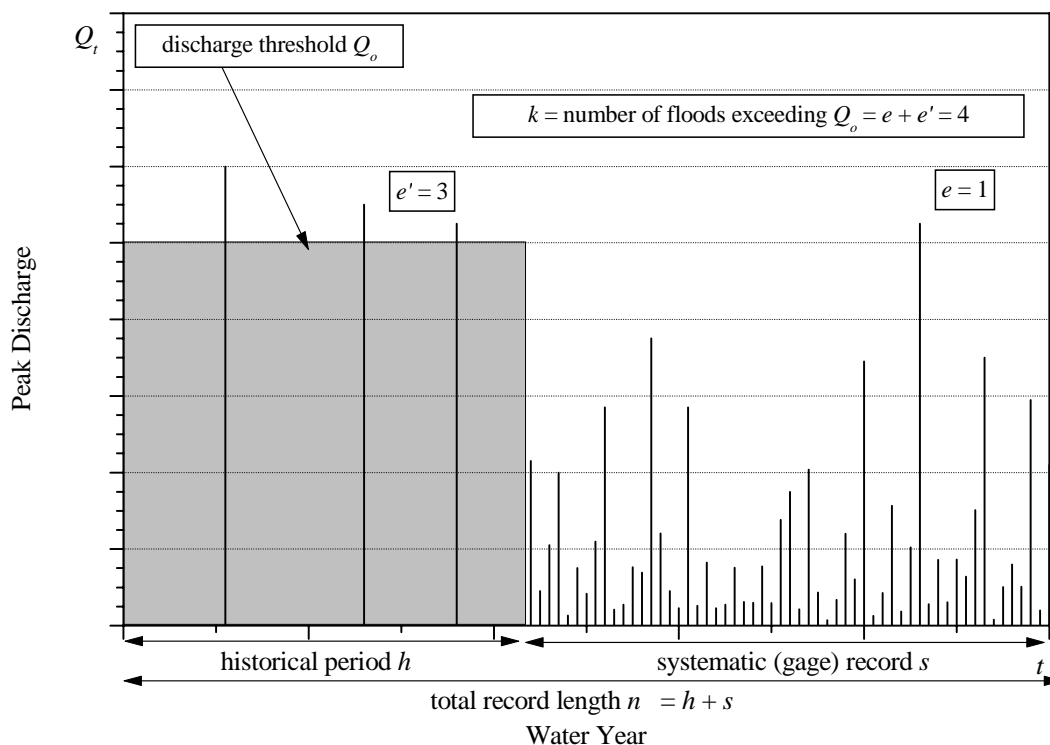


Figure 4-1.—Example of peak discharge time series with historical period and discharge threshold Q_o . The shaded area represents floods of unknown magnitude less than Q_o .

The most common situation for using historical and paleoflood data in flood frequency analysis is that a peak discharge Q is known to be smaller than some threshold Q_o , but the magnitude of Q is unknown. The shaded region in figure 4-1 represents these unknown floods that are below a threshold Q_o . The total record length (n) is the sum of the systematic (s) and historical/paleoflood (h) record lengths ($n=s+h$). We define the number of observations that exceed the threshold in the systematic record (s) as e , which is equal to 1 in figure 4-1. The number of known observations in the historical period (h) is designated e' (equal to 3 in figure 4-1); it is also known that the values are greater than Q_o . We define k as a random variable equal to the number of observations greater than Q_o in the entire record n , where $k = e + e'$. The number of observed floods is denoted g , where $g = s + k - e$.

In some cases, one may know that no floods exceeded the discharge threshold, or $k=0$. Data in this case have been termed a “non-exceedance bound” (Levish et al., 1994; Levish, 2002), where one has knowledge that no flood has exceeded a designated threshold or geomorphic surface in some time period. An example of a non-exceedance bound is knowledge that a river has not inundated “Main Street,” a bridge deck spanning a river, or a geomorphic surface in some time period. Knowledge that $k = 0$ is valuable information that can be used in flood frequency analysis.

Historical and paleoflood information may be described in terms of a flood that exceeded a threshold, with no upper bound. In some cases, one knows only that a flood was larger than some level and does not know the magnitude of the flood. One knows the number of floods that exceeded the discharge threshold. Stedinger and Cohn (1986) have termed this category as “binomial censoring,” in which the exact magnitude of a value is unknown except that it exceeded a lower threshold (see also Russell, 1982). This situation is common for some types of botanical investigations where one can, at present, determine only the minimum stage for plant damage (Hupp, 1988).

There are many situations in which one does not know the exact magnitude of a flood, but that it lies within a range or interval. Interval censoring is used when the exact magnitude of a flood is unknown, but is known to be between some upper and lower amount (Stedinger et al., 1988; Cohn et al., 1997). This class can be used to describe floods with measurement uncertainty. In some cases, the upper threshold, Q_u , can vary for each observation, depending on the data source.

4.1.2 Mixed-Population Graphical Approach

A mixed-population graphical peak-discharge frequency approach has been developed by Reclamation (England et al., 2001). The graphical approach is an at-site frequency method and the frequency curve is constructed in two distinct parts: (1) standard hydrologic statistical methods are used to define a frequency curve for return periods less than and including the 100-year return period (e.g., IACWD, 1982; Ries and Crouse, 2002) and (2) graphical methods are used for estimates greater than the 100-year return period. Peak discharge estimates from gaging stations are used to define the first part of the curve and at-site paleoflood data are used to define the second part of the curve. The first part is estimated assuming an LP-III distribution. One of three at-site techniques and associated computer programs is typically used to estimate the parameters of the LP-III distribution, calculate quantiles, and estimate confidence intervals: (1) the Bulletin 17B Method (IACWD, 1982) and *FREQY* (Carson, 1989); (2) expected moments methods (Cohn et al., 1997) and *EMA* (England, 1999); or (3) Bayesian maximum likelihood and *FLDFRQ3* (O’Connell, 1999). Historical information is included in the at-site frequency analysis when it is available. Historical data can be used to adjust a so-called “high outlier” using *FREQY*, *EMA*, or *FLDFRQ3*. Low outliers can be adjusted using IACWD (1982) methods. The second portion of the frequency curve is estimated assuming a 2-parameter log-Normal (LN-2) distribution. It is defined between the 100-year and the available paleoflood data return periods, and extrapolated beyond the paleoflood data using this LN-2 distribution. Two points are typically used to estimate this portion of the flood-frequency curve: (1) the LP-III model 100-year peak discharge estimate and (2) the midpoint in time and discharge of the paleoflood data. Logarithms (base 10) of the peak flows and standard Normal variates of return periods are used to estimate the LN-2 parameters using least squares (England, 2000). The LN-2 distribution was found to reasonably represent daily standardized precipitation in the western United States (Lane, 1997).

The mixed-population graphical approach is generally used as an initial tool to estimate flood hazard curves. The approach has been developed so that one can estimate an extreme flood frequency curve at any location in the western United States with a minimal amount of effort using existing streamflow data and some site-specific paleoflood data. There are two main assumptions of this graphical approach for estimating extreme flood probabilities: the upper

portion of the frequency curve is appropriately defined by the 100-year peak discharge and paleoflood data and the extrapolation of this portion of the curve using a LN-2 model is appropriate. An example peak-flow frequency curve using the graphical approach is shown in figure 4-2. The approach has been reviewed by Kuczera (2000). Kuczera pointed out the major weaknesses were the use of an envelope curve, lack of confidence intervals, and extrapolation. Kuczera recommended that regional growth curves be used to compliment the use of envelope curves.

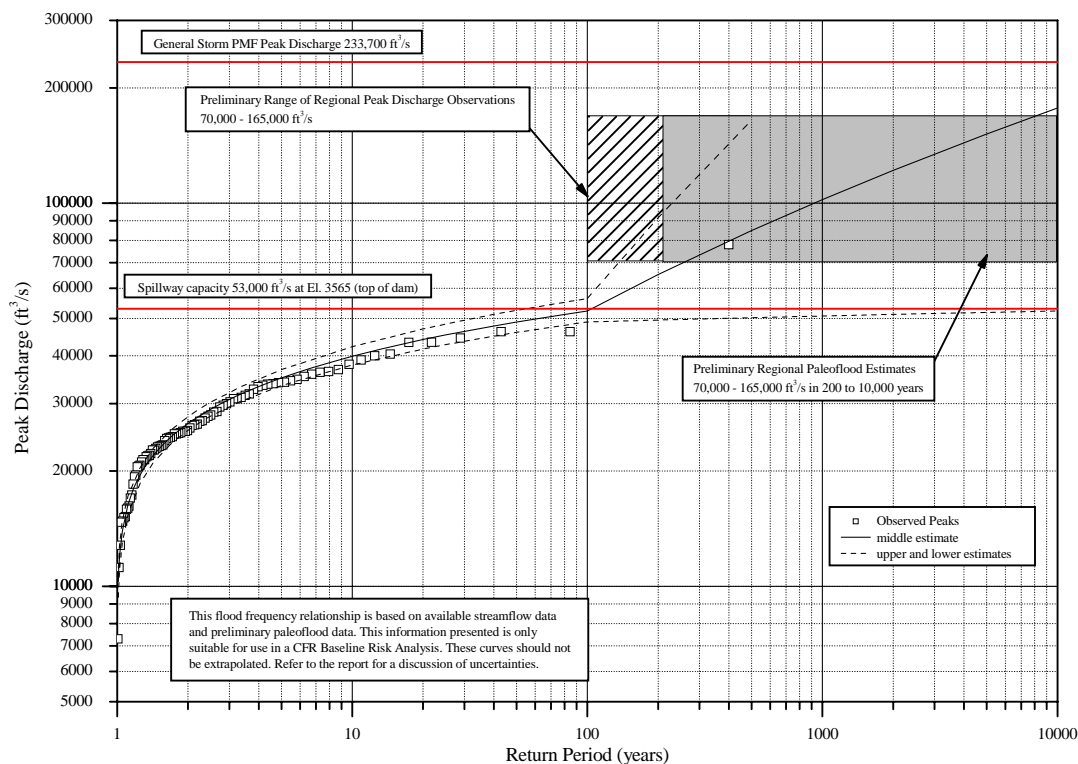


Figure 4-2.—Example application of mixed-population graphical flood frequency curve using peak discharges on the South Fork Flathead River near Hungry Horse, Montana.

4.1.3 Expected Moments Algorithm

The EMA (Lane, 1995; Lane and Cohn, 1996; Cohn et al., 1997, 2001) is a new moments-based parameter estimation procedure that was designed to incorporate many different types of systematic, historical, and paleoflood data into flood frequency analysis. EMA assumes the LP-III distribution is the true distribution for floods. EMA was designed to handle the four different classes of historical and paleoflood data beyond the applicability of the Bulletin 17B historical weighting procedure (IAWCD, 1982). As noted by Cohn et al. (1997, 2001) and England (1998), EMA is philosophically consistent with, and is an improvement to, the Bulletin 17B method of moments procedure when one has historical or paleoflood information. EMA is specifically designed to use historical and paleoflood data, in addition to annual peak flows from gaging stations, in a manner similar to Maximum Likelihood Estimators (Lane and Cohn, 1996). It is a more logical and efficient way to use historical and paleoflood data than the current Bulletin 17B historical method, and it is a natural extension to the moments-based framework of Bulletin 17B.

The five basic steps of EMA are:

- (1) Estimate an initial set of the three sample statistics $(\hat{\mu}, \hat{\sigma}^2, \hat{\gamma})$, from the floods with known magnitudes. These floods are typically observations from the gaging station record and possibly some historical or paleofloods. At this step, floods with unknown magnitudes and magnitudes described by a range are not included.
- (2) Based on the initial sample statistics from step (1), estimate a set of the LP-III distribution parameters $(\hat{\tau}, \hat{\alpha}, \hat{\beta})$.
- (3) From the set of LP-III parameters from step (2), estimate a new set of sample moments based on the complete data set: known-magnitude floods, floods less than some threshold(s), unknown magnitude floods that exceed some threshold(s), and floods described by a range.
- (4) From this new set of moments, estimate a new set of LP-III parameters.
- (5) Compare the parameters from step (4) to those computed from step (2). Repeat steps (3) and (4) until the parameter estimates converge. The main equations used by EMA are listed in Cohn et al. (1997), England (1999), and England et al. (2003).

EMA has been rigorously peer reviewed in the literature (Cohn et al., 1997, 2001; England et al., 2003a, 2003b) and provides a suitable flood frequency model. EMA has been applied at many sites for peak-flow frequency (England et al., 2003b). The National Research Council applied EMA for 3-day annual maximum mean floodflows on the American River (NRC, 1999). An example peak-flow frequency curve with EMA is shown in figure 4-3. There are several limitations with the current version of EMA: (1) the program assumes that the distribution is LP-III, (2) software has not been fully developed to implement the confidence interval technique of Cohn et al. (2001), and (3) low outlier and regional skew methods with EMA have been recently developed (Griffis et al., 2003), but not tested with actual data.

4.1.4 FLDFRQ3

FLDFRQ3 (O'Connell, 1999; O'Connell et al. 2002) uses a Bayesian maximum likelihood procedure to estimate parameters of various distributions. The Bayesian approach includes measurement uncertainty in the parameter estimation procedure. This approach uses a "global" parameter integration grid in order to identify ranges of probability distributions that are consistent with the data (O'Connell, 1999). Two measurement error sources are included: peak discharge measurement errors and errors in paleohydrologic bound ages. Bayesian methods (Tarantola, 1987) and likelihood functions modified from Stedinger and Cohn (1986) are used to incorporate data and parameter uncertainties. Two options can be used to find the "global" maximum likelihood estimate in FLDFRQ3 (O'Connell, 1999): simulated annealing and the downhill simplex method. In FLDFRQ3, one is able to choose among five main three-parameter probability distributions to assume a peak discharge parent distribution. These distributions are the Generalized Extreme Value, Generalized Logistic, Generalized Normal, Generalized Pareto, and Pearson Type III (P-III) (Hosking and Wallis, 1997). These distributions include two

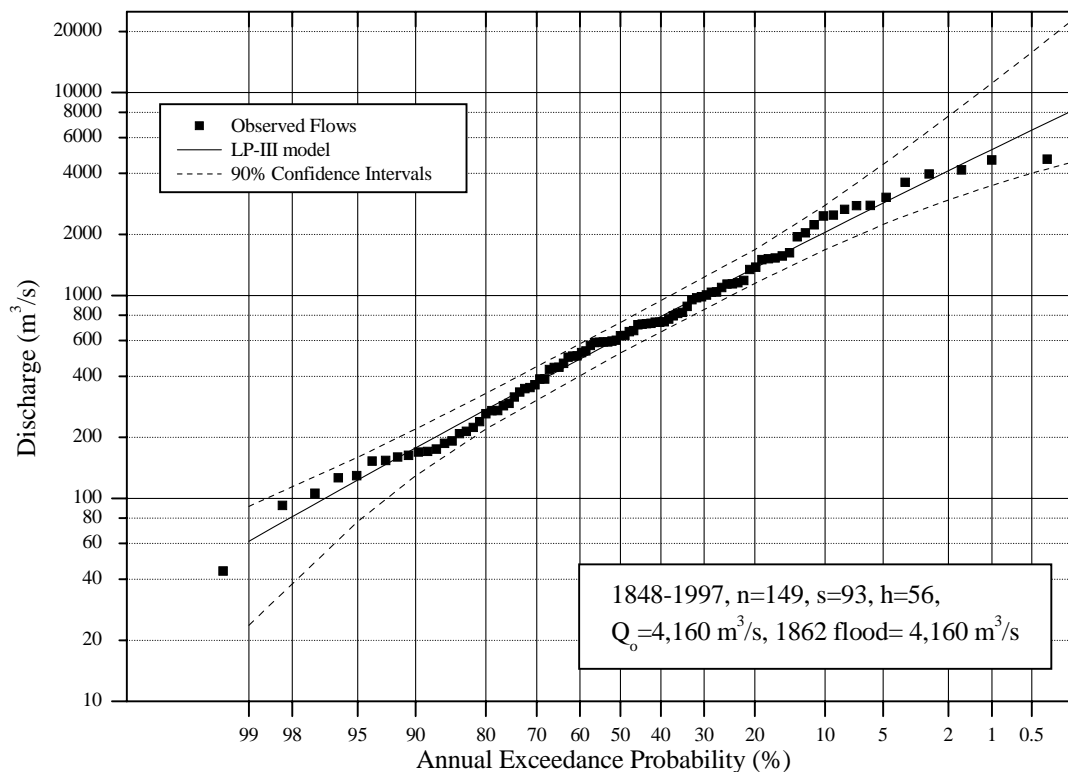


Figure 4-3.—Example application of EMA for American River annual maximum 3-day mean discharge frequency analysis.

logarithmic transform options for P-III models to include the LP-III. O’Connell (1999) provides details of the numerical approach used for estimating distribution parameters and uncertainty using grid integration.

There are generally three main steps in running FLDFRQ3 (O’Connell, 1999): input and data check, parameter estimation for a particular distribution, and generating parameter uncertainties for a particular model (e.g., LP-III) using grid integration. The data are grouped into two broad classes: data with normal uncertainties, such as peak discharge, and values in a range with potentially variable probability density and skew within the range, such as paleohydrologic bound discharges and ages and discrete paleofloods. After entering and checking data, the parameter estimates are obtained from the data and assumed model. The user then checks the appropriateness of the model and estimated parameters. There can be several steps here to determine the “best models” (there can be more than one) that fit the data and the model parameters. Finally, the user estimates the parameter uncertainty given the chosen model and parameter combination. O’Connell et al. (2002) demonstrate how to combine results of several models and their parameter uncertainties using a likelihood criterion.

FLDFRQ3 has been rigorously peer reviewed in the literature (O’Connell et al., 2002) and contains suitable flood frequency models for all levels of analysis. It has been used at many sites

for peak-flow frequency, such as Folsom Dam (Bureau of Reclamation, 2002), Seminole and Glendo Dams (Levish et al., 2003), and Pathfinder Dam (England, 2003). An example peak-discharge frequency curve using FLDFRQ3 is shown in figure 4-4.

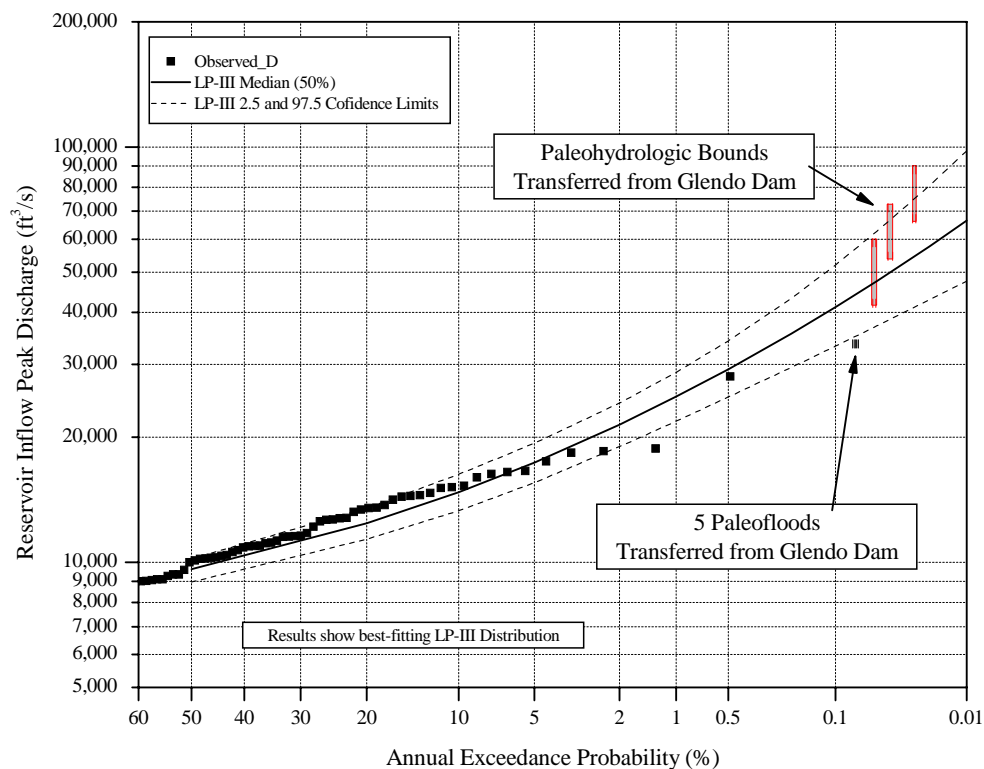


Figure 4-4—Annual peak-discharge frequency inflows to Pathfinder Dam, Wyoming, from best-fitting LP-III distribution using FLDFRQ3 (England, April 2003).

4.2 Hydrograph Scaling and Volumes

Practical tools have been developed for estimating probabilistic hydrographs that can be used in risk analyses for dam safety. These tools are presented in England (2003a) and are summarized below. The key feature of the approach is to use peak-discharge frequency curves that include paleoflood data as a basis to develop hydrographs and volume frequency curves. The methods are relatively flexible and can be tailored to different types of investigations. The methods need to be adjusted depending on the available data at the site and region of interest. For example, if a peak-discharge frequency curve developed using the graphical approach is available, one could use less detailed methods to develop hydrographs because the data might not warrant sophisticated techniques. In contrast, if detailed, high-quality peak discharge and paleoflood data are available, one could use more refined methods such as the SEFM (MGS Engineering Consultants, Inc. [MGS], 2001) discussed below.

Probabilistic hydrographs can be constructed based on streamflow estimates from gaging stations, historical data, and paleoflood data. Four components are used: (1) a peak discharge-probability relationship, (2) an extreme storm duration probability relationship, (3) relationships between peak discharge and maximum mean daily flow volumes, and (4) observed hourly flow

hydrographs that have regulation effects removed. The key idea is calibration or scaling of hydrographs to match peak discharge for a given probability. The approach relies completely on the specification of a peak-flow frequency curve that describes the probabilities of interest, based on paleoflood data.

There are four major assumptions for developing the hydrographs: (1) the probability of peak discharge represents a probability of the composite hydrograph, (2) unit hydrograph assumptions apply to the basin, (3) direct runoff volumes can be estimated from daily flow hydrographs, and (4) the recorded streamflow observations, historical information, and paleoflood data in the river basin of interest provide an adequate sample so one can extrapolate peak discharge probabilities, peak-volume relationships, and hydrographs for extreme floods. Maximum mean discharge (\bar{Q}_d) for n-day periods is related to peak discharge (Q_p) by a power function:

$$\log \bar{Q}_d = a + b \log Q_p \quad (1)$$

The assumed known variable is peak discharge (Q_p), with an associated exceedance probability estimate from the frequency curve. The quality of the regression relationship expressed in equation (1) depends principally on the data from the site of interest and the flow duration (n). Mixed-population flood data (e.g., from thunderstorms, snowmelt, or rain-on-snow) can lead to difficulties in obtaining statistically significant relationships. Good regression fits are typically found for shorter duration (1- to 7-day) flow volumes; the relationships become progressively worse for longer durations. The maximum n-day hydrograph ordinates are linearly scaled, based on the selected n-day volume.

An alternate approach to using streamflow data is to use hydrographs from rainfall-runoff models as a basis for scaling. In these cases, there are typically no flood hydrograph data at the site of interest. A design flood hydrograph, a PMF hydrograph, or other suitable hydrograph for the basin is obtained. The hydrograph can then be scaled in some linear fashion to match peak flows from a peak-flow frequency curve. The analyst needs to be careful to ensure that flood volumes do not exceed physical limits when applying this scaling procedure.

Probabilistic hydrographs, developed from scaling streamflow observations or from rainfall-runoff models, are combined with recommendations for initial reservoir levels for hydrograph routing. Reservoir routing issues and selection of varying initial levels are discussed in England (2003a). One can then determine a maximum reservoir level by routing the given hydrograph and initial reservoir level. Initial reservoir levels can sometimes have a large effect on maximum reservoir level estimates for extreme floods. Maximum reservoir elevation probability estimates depend on the inflow hydrograph peak, volume, shape, and probability estimate. The initial reservoir level can also be a major factor. The selection of an appropriate initial reservoir level is of considerable importance in determination of spillway adequacy (Nathan and Weinmann, 1999, p. 57). For estimating maximum reservoir levels for design floods such as the PMF, Reclamation uses a fixed initial reservoir level. This initial reservoir level is usually set at the top of active conservation or bottom of the flood control pool. This assumption has been criticized as being unduly conservative. Newton (1983, p. 914) notes that current practice for most agencies is to assume conservatively high initial pool levels for routing PMFs. Instead of using a fixed initial reservoir level for routing hydrographs, variable initial reservoir levels are

needed for risk analysis. Initial reservoir levels and associated exceedance probabilities should be estimated from daily reservoir elevation estimates for the period of record at the site of interest.

4.3 GRADEX Method Analysis Technique

Much of this description of the GRADEX Method is paraphrased from the Ph.D. dissertation, “Methodology for Estimating the Upper Tail of Flood-Peak Frequency Distributions Using Hydrometeorological Information,” by Mauro Da Chunha Naghettini, completed in partial fulfillment of the requirements for the Ph.D. degree at the University of Colorado, Department of Civil, Environmental, and Architectural Engineering, 1994. Naghettini, Potter, and Illangasekare later described the same method in the Water Resources Research publication in 1996. Some additional comments related to Reclamation dam safety needs are inserted when appropriate.

In its 1988 report, the National Research Council Committee on Estimating the Probabilities of Extreme Floods identified principles for improving the estimation of floods with AEPs on the order of 10^{-3} or smaller. These principles are: “(1) ‘substitution of space for time’; (2) introduction of more ‘structure’ into the models; and (3) focus on extremes or ‘tails’ as opposed to or even to the exclusion of central characteristics” (NRC, 1988). The methodology proposed in Naghettini’s Ph.D. dissertation (1994) presents techniques for the estimation of extreme flood peaks and volumes that make strong use of these principles. The main objective is to develop a peak-flow frequency curve for the extremely rare probabilities. To do so, the method involves a peak to volume relationship and the derivation of a frequency curve of extreme flood volumes based on extreme regional rainfall statistics. The method is useful to current Reclamation dam safety needs in that it provides a means to produce frequency curves for rare flood volumes and also some apparatus to define peak flows for the extreme flood volumes. It can also be used to create hydrographs based on the flood volumes and peaks, if needed.

The method relies on extrapolating a conventionally estimated probability distribution of flood volumes. To strengthen this step, the GRADEX Method, originally developed by Guillot and Duband (1967), is incorporated. The GRADEX Method has been used extensively in France since about 1967 for various improvements and hydrologic safety investigations and spillway renovations at numerous hydroelectric dams and facilities. The French Committee on Large Dams has prepared the publication, *Small Dams* (undated), that outlines the very basic steps that can be used to perform such calculations in France. The main GRADEX Method is based on two assumptions:

- (1) That, asymptotically, the upper tail of the flood volume distribution is exponential with the same scale parameter as that which describes the upper tail of the distribution of rainfall volumes for the basin. Figure 4-5 graphically displays this assumption.
- (2) That any increase in total precipitation during a severe rain event, falling on already saturated ground, will produce a corresponding increase in volume of the resulting flood.

The estimation of the rainfall scale parameter has been enhanced in this application from the original French methodology by incorporating the work of Smith (1989), who developed a regional model for estimating the upper tail of a frequency distribution based on extreme order statistics. Figure 4-5 depicts the GRADEX Method.

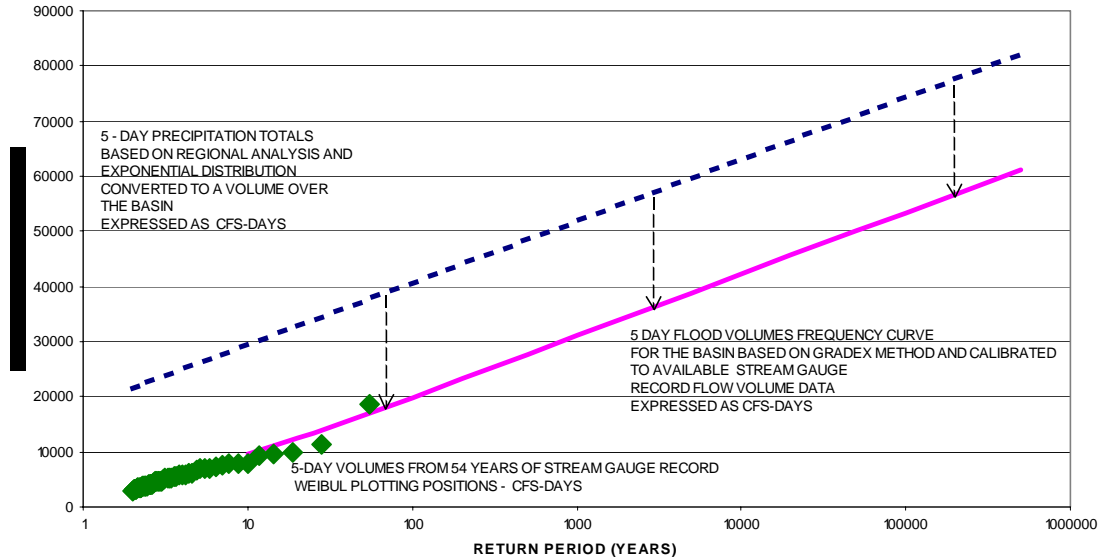


Figure 4-5.—GRADEX Method of volume frequency curve calculation.

The location where the extrapolated flood volume curve takes over from a more conventional analysis of stream gage volume data, such as using LP-III, is not fixed, but must be assumed. In the literature from France, this return period ranges from about 10 years, for very impermeable basins, to 50 years for very permeable basins.

The first assumption of the GRADEX Method given above refers to the upper tail of the rainfall volume distribution, which is assumed to be a generalized Pareto density function of the form:

$$g_p(p|s,K) = 1/a [1 - ((Kp)/a)]^{1/K - 1} \quad \text{if } K \neq 0 \tag{2}$$

Which will reduce to a simple exponential density function of the form:

$$g_p(p|s,K) = (1/a)\exp(-p/a) \quad \text{if } K = 0 \tag{3}$$

Where the positive constants K and a are the location and scale parameters, respectively. The scale parameter a is a function of various physical components of the available rain gage data sets such as elevation and mean annual precipitation (MAP). If $K > 0$ then the distribution of rainfall for all sites has an upper bound; if $K < 0$ it is unbounded. If $K = 0$ the upper tail of the distribution is exponential with a scale parameter a . The parameter estimates are found by fitting a distribution that asymptotically exhibits an exponential upper tail (e.g., Exponential, Gumbel, Gamma, or log-Normal) to rainfall maxima. Combining the two GRADEX assumptions causes

the upper tail of the flood volume distribution to also be exponential with the same scale parameter a (the GRADEX parameter) as the one estimated for the upper tail of the distribution of rainfall volumes, except for a necessary conversion to units of volume instead of precipitation depth.

The first step in the GRADEX Method involves selecting a critical duration. This begins with an examination of the time series of unregulated daily flows for a stream gage record deemed to be hydrologically similar to the basin being studied or a record of reservoir daily inflows. What is required is a series of independent flood events (hydrographs) that have occurred over the entire length of the unregulated streamflow record. These flood events should be rain-generated, as opposed to floods derived from snowmelt. Also, the rain-generated flood events should all be of the same storm type. For these reasons, the stream gage record analysis should be limited to a “season” when the rain floods of the same type are most likely to occur based on historic experience. Once the season is selected, the daily streamflows for each year within that season are examined. A threshold discharge, Q threshold, is set. The number of daily flows above this threshold value is observed. Multi-day events with several days of flow above the threshold are observed. The number of and the duration of each of these multi-day events are then calculated. It is desired to obtain a set of independent flood events with nearly the same number of events as the number of years in the length of record. If the number of events calculated is too large or small, then the threshold Q value is raised or lowered until approximately the number of events equals the number of years in the stream gage record. The average duration for the entire set of events is then calculated. This average duration, generally raised to the next highest number of days, will become the critical duration d used for the rest of the study. Once the critical duration d is determined, the average flow discharge of all of these events can be calculated. A second flow value, termed the reference discharge, is also determined such that 90 percent of the selected flood events will have average d -day flow values less than this discharge value. An approximate return period is also placed on this reference discharge value by the inverse of the Gringorten plotting position formula.

$$\text{Reference return period} = 1/((i - 0.44)/(N + 0.12)) \quad (4)$$

Where N is the total number of years of record and i is the rank of the selected reference discharge. This part of the analysis can require much hydrologic judgment. Often, a set of daily flows will show a pattern that is above the selected threshold Q for 1, 2, or 3 or more days, then drop just below the threshold for 1, 2, or 3 days, and then continue for a few more days above the threshold. Decisions have to be made as to whether this should all be considered one flood event or separated into two or more events. Rainfall records from the area may help with this decision, but, generally, it is left to the analyst to make the decision. Independence of the events is generally assumed if the time from the end of the first event to the beginning of the next event is longer than the critical duration that is calculated for this set of events.

In hydrology, this process is often referred to as a marked point process. Much literature is available dealing with statistical assumptions related to events derived by the marked point process. By its nature, the set of floods derived by this process may include several events in any one year, and no events for several years. This is what is desired because the data will be used to help calibrate the GRADEX-derived flood volume information from rainfall totals for the critical duration. It is often the case that several large rainfall events can occur in any one year.

Cautions that are given for the selection of the critical duration are: (1) that selection of too short a duration might result in non-exponential, probably heavier-than-exponential, upper tails for the rainfall volumes and (2) adoption of too long a duration might result in poor peak-volume relationships. Since the goal of the application of the GRADEX Method to Reclamation dam safety investigations is to create a good volume relationship, it is advised to raise the computed critical duration value to the next higher full day.

In conventional applications of the GRADEX Method, the parameter a can be estimated by fitting an exponentially tailed distribution to seasonal or annual rainfall maxima. The simplest estimation procedure of the GRADEX parameter is to fit a Gumbel distribution to a series of annual maximum rainfall events for a duration d that is equal to the watershed critical duration, or some other measure based on time of concentration calculations. However, the most frequently used estimation procedure is to fit an exponentially tailed distribution to seasonal (sometimes monthly) rainfall maxima and then combine the seasonal (monthly) distributions to obtain the annual distribution. What can be shown is that the annual frequency curve, which would no longer be a strictly exponential curve, will tend to have the same shape or slope (GRADEX) as that of the month that produces the largest rainfall amounts, especially at the extreme upper end. A slightly more conservative approach is to use smaller durations of seasonal maxima rainfall totals for even smaller durations, even 24 or 48 hours.

Estimation of the GRADEX parameter of flood volumes requires that different units for expressing the rainfall be used. If the drainage area and critical duration d are expressed as mi^2 and days, respectively, and the GRADEX parameters are to be expressed in English units, then:

$$\text{Flood Volume GRADEX} = [(26.89 * \text{DA})/d] * \text{Rainfall GRADEX} \quad (5)$$

Similar expressions exist for computations done in metric units.

Usually, following the French examples, the extrapolation of the flood volume distribution according to the GRADEX parameter starts at the 10-year flood for small and relatively impervious basins, or at the 20-year flood for larger basins, or possibly the 50-year flood for watersheds showing very little topographical relief or high infiltration capacity.

The current application of the GRADEX Method applies a new methodology to estimate the slope of the rainfall durations for a critical duration d within a specified season. This new approach combines deterministic constraints with contemporary statistical techniques, extracting the maximum information from the available data. The regional rainfall frequency model described in this section is based on the premise that meteorological processes affecting large rainfall events may be different from those affecting smaller rainfall events. The model is an adaptation of a regional flood frequency model developed by Smith (1989) and is based on results from extreme value theory. In this model, the parameters a and K in the Pareto or exponential distribution functions (equations 2 or 3) are determined based on a regional analysis of the largest d -day rainfall totals for several daily rainfall stations that are shown to be or believed to be homogeneous and to represent the meteorological conditions of the basin under study. The parameter a is further allowed to be a function of the basin mean annual precipitation and the basin mean elevation.

$$a = S_i = \exp(c + b_1 W_{i1} + b_2 W_{i2}) \quad (6)$$

Where W_{i1} and W_{i2} are the natural logarithms of the set of rain gage elevations and mean annual precipitation values for each gage site i , respectively. The constants b_1 , b_2 , and c are determined as part of the parameter estimation process. This is an improvement over the French general cases where only one rain gage or set of regional information reduced to one point for any basin may be used. Further, no consideration of elevation or mean annual precipitation is given in the standard GRADEX analysis.

The mathematical process to estimate the parameters K , c , b_1 , and b_2 from the data set of rainfall totals proceeds as a maximum likelihood parameter estimation process. A log-likelihood function is then formed.

$$L_n(K, b_1, b_2, c) = \sum \sum \ln[g_p(Zg / K, b_1, b_2, c)] + C \quad (7)$$

Where Zg is the set of random variables of d -day rainfall totals above the threshold precipitation value at each gage site. The double sum is once for all of precipitation values above the threshold value at each site and then summed over all sites.

Partial derivatives of the log-likelihood function with respect to the four parameters to be estimated (K , b_1 , b_2 , and c) are derived. In taking the partial derivatives, the additional constant C in the log-likelihood function is eliminated. These partial derivative functions are then set equal to zero, and a series of four non-linear equations with four unknowns (if $K \neq 0$) or three non-linear equations with three unknowns (if $K = 0$ is assumed) are formed. As part of the parameter estimation process, statistical tests are performed to see if the three-parameter exponential distribution form is equally valid for the data set as is the four-parameter Pareto distribution. In almost all cases, this is true. The assumption that the extreme rainfall totals can follow an exponential distribution is validated, and the rest of the GRADEX Method follows. The three parameters are then used to form the single scale parameter a (equation 5) for a single parameter exponential distribution form. In cases where the statistical test does not prove the validity of the three-parameter exponential distribution form, the rainfall total data sets need to be further investigated as to homogeneity.

Software to solve the complex sets of non-linear equations was adopted from the MINPACK software package originally developed in 1980 at the Argonne National Laboratory. This software is now free and in the public domain. Only the most extreme rainfall totals for the critical durations d at each daily rainfall stations are used as data. Once the equations are solved, the scale parameter a is estimated.

Readers who are interested in the complete theoretical and mathematical background are referred to Naghettini (1994, chapter 4). The remainder of this discussion deals with the hydrological and meteorological details of this method.

The method requires that all stations selected have a common period of record that is as long as possible. Daily rainfall totals for all official rainfall gage stations in the United States are available on compact disks from Hydrosphere Corp. (2002). Several stations near the basin being studied need to be selected and their periods of record noted. These rain gage records

should represent climate and meteorological conditions similar to conditions in the basin being studied. Stations too far from the study area or too high or low in elevation should not be used. The same continuous period of record should be available for each rain gage selected. It is also advisable to avoid selecting too many stations in any one area, which would then overly weight the climate and rainfall records in that very localized area compared to the rest of the surrounding areas for the basin being studied.

The method relies on data from the rainfall gage records that cover the same continuous period of time for each gage. If large gaps in the gage record are found (even though the beginning and ending dates may cover the continuous period needed), the record should be discarded. Recorded rainfall data is subject to many errors, omissions, and other anomalies. Within each rain gage record, missing days, days with accumulated rainfall from several previous days, and days with only a trace of precipitation or other notations are noted. Analyzing the daily rainfall totals involves summing the total rainfalls for the number of days previously defined as the critical duration d for this basin based on analysis of the appropriate stream gage records. The process is complicated by the need to eliminate all the days with missing data or with special notes, such as when the recorded value was already an accumulated value. Any multi-day total rainfall that includes such data is then set to zero and eliminated from further consideration. Trace values are set to zero for the day that they were reported and then they are allowed in the summation process. Any extremely large daily rainfall totals need to be further checked against official hardcopy records, and the correct daily values for these dates are inserted in the analysis if changes are needed. In the process, independence of the rainfall total events also needs to be ensured. The start dates of any two multi-day events must be more than the critical duration d apart.

The method requires selection of a number of multi-day total rain events at each gage equal to the number of common years of record for all selected gages. A threshold d -day total rainfall for each gage is selected such that exactly the same number of independent d -day rain totals is above this value as are in the continuous period of record covered by all the rain gages in the analysis. Further, a reference total precipitation value for each rain gage is also selected such that 90 percent of the previously selected events are below this reference precipitation value. The threshold and reference precipitation values are used later in the statistical analysis. Only the top 10 percent of the d -day rainfall totals are used in the regional analysis. This amounts to a form of top-end fitting for the precipitation totals.

To further facilitate the computations, the rainfall multi-day totals are reduced by subtraction of the reference precipitation amount for each rain gage. This step is necessary to eliminate very large numbers in the calculations that follow. This is a form of “indexing” and is common in many regional flood methodologies.

The upper order statistical method calculates the slope (or GRADEX parameter) of the best-fit decaying exponential distribution of the top 10 percent of the d -day total indexed precipitation amounts for each selected rain gage site. The selected station elevations and mean annual precipitations help weight the slope parameter. Knowing the basin’s mean elevation and MAP, a GRADEX parameter fit specifically to the drainage basin being studied can be calculated. The result is the slope of the decaying exponential distribution of the most extreme precipitation amounts that the selected precipitation data suggest can occur over the drainage basin. The

distribution of d -day total index precipitation values can then be used with knowledge of the contributing drainage area for the basin to create associated d -day volumes as shown in equation 5, above. This distribution of d -day volumes now has a slope, but it must also be fit to the actual reservoir d -day inflow volumes at the lower return periods. This is done through a statistical procedure. The fitted curve will match the experienced stream gage d -day volumes near the computed reference Q value previously computed. The resulting curve can be extended to very high return periods based on the second basic assumption of the method, that all large flood volumes will occur from rain falling on already thoroughly saturated conditions in the contributing areas of the basin, and any increase in a d -day rainfall will result in a corresponding increase in d -day inflow volume to the reservoir.

Since the GRADEX parameter a is calculated using a maximum likelihood estimate, it is further possible to place a confidence bound on this parameter. Note that for one-parameter distributions such as the exponential, the natural logarithm ratio between the estimated likelihood function and a true likelihood function for the one parameter can be proportional to a chi-square distribution with one degree of freedom. This process is displayed on the Web page, http://www.weibull.com/LifeDataWeb/likelihood_ratio_confidence_boundsexp.htm. By a trial and error process, the upper and lower confidence bounds associated with the a parameter estimate can be determined for some set confidence level.

Once the slope and location of the flood volume curve for the d -day durations have been established, the question of what is the probability that a particular volume of flooding will be equaled or exceeded in any year can be answered. The more common question is what is the volume of flooding that will be exceeded on average only once in a stated return period, T_c = number of years. To answer that question, the calculated exponential distribution and associated confidence bounds, need to be inverted. The inverse of the distribution has the form:

$$\hat{x}(T_c) = \hat{\beta}_1 + \hat{a} \ln(T_c + \hat{\beta}_2) \quad (8)$$

Where:

$\hat{x}(T_c)$ is the d -day flow value for any required return period, in $\text{ft}^3/\text{s-days}$, T_c is the return period (in years) for which a d -day flow estimate is required, \hat{a} is the previously estimated GRADEX slope factors converted to volume units, and $\hat{\beta}_1$ and $\hat{\beta}_2$ are constants that can be estimated from a system of simultaneous equations that are formed knowing the mean of the sample of d -day flood discharges and the reference d -day discharge with an approximate return period. Both of these discharge values and the reference discharge return periods are previously computed from the daily inflow record for the location of the study. The $\hat{x}(T_c)$ value can then be further converted to more common volume units such as acre-feet for a specified number of days.

The original goal of the method, as presented in Naghettini (1994), was to produce a peak-flow frequency curve. In this procedure, a known set of peak flows associated with d -day volumes can be determined from the stream or reservoir inflows, assuming peak flows have been

recorded. This set of paired data for the period of the streamflow record can be further extended by the use of various rainfall-runoff models. Calibrated rainfall-runoff models can be created for some of the largest events in the stream gage record if appropriate rainfall data are also available.

In the original presentation of the method, it is suggested that several large storms, all of the same type and from meteorologically similar areas, could be transposed into the basin. For each of these large storms, the calibrated rainfall-runoff models can then be rerun with the transposed storm precipitation data and a new peak flow and hydrograph can be generated. Additional sensitivity analysis runs can be made by varying certain parameters in the rainfall-runoff model that affect the peak, such as the lag time or other parameters related to unit hydrograph development. The peaks and d -day volumes from all of the additional transposed storms can then be added to the original set of peak and volume data. Regressions on this extended set of peaks and volumes can provide the necessary information to help determine a peak flow for a selected volume at some rare return period that has been calculated by the GRADEX Method. In both the French and American literature for the GRADEX Method, it is suggested that the regression between volume and peak data should not be linear. The French literature states that the ratio of peaks to a d -day volume will increase with increasing return periods. A regression procedure known as LOWESS (Locally Weighted Regression and Smoothing of Scatter Plots) (Cleveland, 1979) can be used to perform the non-linear curve fitting required for this procedure.

Reclamation's practice with the method has not involved multiple storm transpositions. For each application, some attempt has been made to create a calibrated rainfall-runoff model using HEC-HMS (U.S. Army Corps of Engineers, 2002). The largest one or two floods from the stream gage record and the best available rainfall data are used to create the calibrated runoff model. The model is calibrated to match as nearly as possible the peak and the entire volume of flooding, which may be longer than the d -day critical duration determined earlier. Once the calibrated rainfall-runoff model is completed, the historic rainfall information, with both temporal and spatial distribution, is increased by a constant ratio at each time period. The resulting peak and d -day volume of the hydrograph is recorded. Additional runs are made and the lag times are reduced by 10 or 20 percent to account for the fact that the historic flood may not have resulted from such intense rainfall as the desired higher return period floods might produce. With more intense rainfalls, it may be that the basin lag times should be reduced to allow for quicker formation of the flood peaks. The extended peak and d -day volume set is then fit with the LOWESS procedure. Using this non-linear regression, peak flows associated with the various volumes for different return periods by the GRADEX Method can be estimated. This method will also allow for production of the entire hydrograph with exactly the required d -day volume estimated by the GRADEX Method. Examples of the results of this computation can be seen in the Fresno Dam example at the end of this report.

In the publication, *Small Dams*, (French Committee on Large Dams, undated), an empirical equation is given that will produce an entire hydrograph with a specified peak, time to peak, and the desired time step. One of the parameters in that equation can be varied by trial and error until the desired volume of the hydrograph over any period of time, such as d -days, is achieved. This represents another strictly empirical method to derive a hydrograph once a peak and volume for the desired return period are known.

Some other concerns have become apparent in the application of the GRADEX Method to some dams in the Reclamation inventory. The first concern is with the possible additional volume of flooding that may result from snowmelt that may not be explicitly considered in the GRADEX Method. The available literature indicates that a separate snowmelt volume analysis should be undertaken. For each year of stream gage record, the maximum snowmelt volume for some time period larger than d -days should be estimated. A separate LP-III (or any other distribution) analysis of the snowmelt volumes can be constructed and extrapolated to rare return periods. This frequency curve of snowmelt flood volumes can be used with a combined probability analysis of the rain flood GRADEX d -day volumes. The resulting frequency curve will display the probability of getting a flood volume composed of both snowmelt and rain flood volumes. What becomes apparent for the large return periods is that the GRADEX rain flood curve will dominate the combined probability volumes. The combined probability curve is almost identical to the GRADEX curve at the large return periods. For a large return period, the probability of getting a flood with X acre-feet composed of Y acre-feet of snowmelt, plus Z acre-feet of rain generated flood volume ($X = Y + Z$), is nearly identical to getting the rain flood alone with X acre-feet of volume. An example of this type of combined probability analysis is given in the Fresno Dam example later in this report. It is recommended that in all future applications of the GRADEX Method to Reclamation dams some attempt be made to create a separate snowmelt flood volume frequency curve and a combined probability analysis with the GRADEX rain-flood curve be made to ensure that the rain floods dominate at the rare return periods.

A second concern is with drainage area size. The GRADEX Method is based on assumed basin average rainfall. Because of this, there is a clear question as to its applicability to large basins. The original French literature limits the size of the drainage basins where the GRADEX Method can be applied to about 10^4 square kilometers, or about 3,800 square miles. It is noted that few storms with greater aerial coverage exist in the rain gage data. The application of the method to such larger drainage sizes would not produce defensible results. To approach this problem, the suggestion is that the larger basin be broken into smaller parts along logical lines, such as at major tributary confluences, such that each part is no larger than 3,000 square miles. The GRADEX Method could be applied to each separate part, and a combined probability analysis could then be performed with the resulting curves for each part. The resulting frequency curve would show the volume of flooding that could occur resulting from contributions from each separate part of the basin. This approach has not yet been tried for any Reclamation dams.

For the full application of the method for any final design-level study, additional effort should be made to determine a homogeneous set of rain gages for use in the GRADEX Method. Naghettini (1994) provides some useful suggestions and examples along these lines. Due to time and money constraints, this has not been done in any Reclamation studies to date.

4.4 Australian Rainfall-Runoff Method

The Australian Institution of Engineers developed and published an approach for estimating large to extreme floods in 1999 and revised the method in 2001 (Nathan and Weinmann, 2001). The focus of this work is on estimating floods with very low probabilities of occurrence. The floods developed using this technique usually have AEPs ranging between 1 in 50 and 1 in 10 million. Uncertainties involved in estimating floods increase with increasing sizes of floods.

The following discussion describes Reclamation's experience with estimating rare and extreme floods using the Australian approach.

Three categories of floods are considered – large, rare, and extreme. Large floods typically have probabilities of occurrence ranging from 1 in 50 to 1 in 100. Rare floods include floods with AEPs extending from 1 in 100 to the credible limit of extrapolation, generally around 1 in 2,000. Extreme floods involve estimating floods for the AEPs beyond the limit of credible extrapolation. For risk analysis purposes, rare and extreme floods are of most interest.

Rare floods include events between the largest observed flood and the credible limit of extrapolation. The credible limit of extrapolation depends on the type and amount of data used for flood frequency analysis. Generally, regional flood and precipitation data, and the inclusion of paleoflood data, allow extrapolation out to around 1 in 2,000 or 1 in 5,000. It is important to note that floods in this category contain considerable uncertainty because estimates are outside the range of observations.

Extreme floods extend beyond the credible limit of extrapolation from the data to AEPs out to 1 in 10 million. Estimating these floods requires prescriptive measures, which do not allow the hydrologist to quantify the uncertainty of the estimates even though it is known to be very large. Extreme floods determined by these methods are intended to be consistent and as reasonable as possible given the state of current knowledge.

4.4.1 Approach

The procedures involved in the Australian Rainfall-Runoff Method are based on flood frequency analysis and rainfall-runoff modeling. Any of the flood frequency analysis techniques previously discussed in previous sections of this report are applicable to the Australian Rainfall-Runoff Method. The unique concept in this approach is the use of "AEP-neutral" parameters in the rainfall-runoff modeling process. This involves selecting model parameters such that the AEP of the 1 in Y rainfall amount produces a flood with a 1 in Y AEP.

Reclamation has used an event-based deterministic rainfall-runoff model to convert a 1 in Y AEP design rainfall into a 1 in Y AEP flood. A single set of hydrometeorological parameters and watershed characteristics are used to produce a flood event. No soil moisture or surface storage recovery is provided. Therefore, the deterministic model always produces the same output.

The major inputs to the deterministic rainfall-runoff model are: (1) precipitation (rainfall and snowfall), (2) losses (infiltration/interception), (3) physical watershed characteristics for runoff and routing simulations (drainage areas, watershed and channel slopes, lag times, antecedent moisture, etc.), (4) precipitation-runoff transformation function, and (5) runoff conveyance and routing mechanisms. Model output includes runoff hydrographs at user-specified locations, maximum peak discharges, and total runoff volumes.

Deterministic event-based precipitation-runoff modeling applies design rainfall distributions and volumes to watersheds for which runoff response is characterized by unit hydrographs and generalized loss-rate functions. Calculations proceed from upstream to downstream in the watershed. Subbasin hydrographs are routed and combined at the points of interest.

A design storm (rainfall and basin snow cover) is the primary model input. Typically, a time series of basin-average rainfall for a preselected duration and frequency is input to the model. A 1-hour to 72-hour duration storm event is typically simulated. Appropriate duration storm events should be derived from local and regional rainfall records. In the process of developing extreme floods, the Australian Rainfall-Runoff Method assigns an AEP to the PMP, and is solely a function drainage area size. Reclamation assigns an AEP to the PMP on an as-needed basis and does not endorse the drainage area relationship used by the Australians. Reclamation considers the proximity to moisture sources, areal coverage of the storm, and other factors in assigning an AEP to the PMP.

Excess precipitation is estimated by subtracting losses typically due to infiltration and interception. A variety of infiltration models are available and range from constant uniform loss rates to approximate theory-based functions (Green and Ampt, Philips equation). Antecedent storm assumptions can have a severe impact on basin infiltration estimates. Since the deterministic rainfall-runoff model is based on a single event, soil moisture storage and recovery during and between storms is not considered.

The amount of watershed information required is a function of the type of precipitation-runoff model used. Two classes of models are currently used—lumped and distributed parameter models. Lumped parameter models consider the system as being spatially averaged. In contrast, a distributed system considers hydrologic processes at various points in space and defines model variables as functions of the space dimensions. Some lumped parameter models that are widely in use are HEC-1 (HEC, 1990), FHAR (Bureau of Reclamation, 1990), and RORB (Laurenson and Mein, 1995). Some distributed models that can handle single events include DR3M (Dawdy et. al, 1978), PRMS (Leavesley et. al, 1983), HEC-HMS, and WMS. Additional surface water models are discussed in DeVries and Hromadka (1993).

Transformation of rainfall excess to a direct runoff hydrograph is completed via a convolution integral using (1) unit hydrograph techniques or (2) kinematic wave routing for overland flow. The unit hydrograph has been used extensively for flood runoff estimation. Kinematic wave and distributed modeling approaches may be more appropriate for modeling non-linear systems. A good discussion about rainfall-runoff processes and floods, including practical issues comparing design floods and actual storms, is presented in Pilgrim and Cordery (1993). Beard (1990) presents a methodology for simulating floods of a given probability from hypothetical design storms derived from point rainfall.

Streamflow routing may be classified as either lumped/hydrologic (linear reservoirs, level-pool, Muskingum, etc.) or distributed/hydraulic (diffusive wave, kinematic wave, etc.). In lumped flow routing, streamflows are computed as a function of time at one location; however, in distributed flow routing, streamflows are computed as a function of time at several locations along the stream. Most precipitation-runoff models have adequate routing mechanisms. A detailed discussion of routing options is presented in Chow et al. (1988).

For risk-based dam safety studies, it is necessary to adopt an AEP-neutral approach, where the objective is to derive a 1 in Y AEP flood with an AEP equivalent to its 1 in Y rainfall. The factors that influence the transfer between rainfall and runoff can be characterized by probability distributions. Thus, ideally, the design hydrograph should be determined by considering the joint

probabilities of all the input factors. Stochastic methods are ideally suited to the AEP-neutral objective because they accommodate the observed variability of the inputs while still preserving the interdependencies between parameters. However, for the least important parameters, it may be appropriate to adopt a single representative value instead of the full distribution. Since the relationship between rainfall and runoff is non-linear, it is important to note that adoption of a single representative value for the major inputs will introduce bias into the rainfall-runoff transformation. Therefore, more important model inputs may require use of a joint probability approach.

The simplest approach to deriving AEP-neutral inputs is to use the correlation relationship between the two variables. For example, if it is necessary to derive a temporal relationship to use with the design rainfall magnitude, an appropriate relationship may be derived from the correlation between the largest observed storms and their temporal characteristics during the largest storms on record. When applying relationships based on a limited historical sample to large flood events, the inputs should be conditioned by physical reasoning. For instance, large snowmelt events may require large snowpacks and high temperatures, but the meteorological conditions required to sustain an extreme rainfall event may preclude the joint occurrence of extreme wind speeds. The concurrent wind speeds used in the transformation of snow into runoff must be bounded by a reasonable upper limit.

The selection loss parameters are required inputs common to all event-based rainfall-runoff models. With loss rates, there is evidence to suggest that loss rates are independent of flood magnitude for design floods up to 1 in 100 AEP, though, for more extreme events, it is possible that the loss rates depend on both the AEP and the duration of the design rainfall. When considering snowmelt design floods, it may be necessary to vary loss rates with snowpack extent (Nathan and Bowles, 1997).

The most appropriate approach required to achieve AEP-neutrality depends on the complexity of the system being modelled, the nature of the available data, and the requirements of the flood model. In many cases, it may be expedient to adopt model input parameters derived using regional data, and it will be necessary to supplement empirical evidence by physical reasoning. Calibration of the design flood estimates to flood frequency quantiles will help reduce the uncertainty in extreme flood estimates.

4.4.2 Calibration

Calibration of a flood event model for application to design flood estimation is traditionally restricted to the selection of model parameters to achieve a fit between observed and estimated hydrographs. Attention is focused on collecting streamflow and rainfall data corresponding to the largest events on record. Considerable effort is required to ensure that the temporal and spatial distribution of the rainfall data is representative of the actual event. The ability of a model to reproduce historic events certainly gives some confidence to the validity of subsequent flood estimates. However, the available historic information for floods is usually much smaller than the extreme floods of interest. In most watersheds, the AEPs of the calibration floods are likely to range between 1 in 10 and 1 in 25. While it would be expected that floods of this magnitude would activate some floodplain storage, the non-linear nature of the out-of-bank flood response is such that the streamflow routing characteristics of larger events may be considerably

different. Therefore, while calibration of the model provides valuable information on the flood routing parameters for small floods, caution is needed when using the model to estimate extreme floods of much larger magnitude.

Calibration of rainfall-runoff model results to flood frequency quantiles can provide important information on flood response characteristics for extreme flood events. With this approach, rainfall data are prepared for a specified AEP and then used with a given set of model parameters and input assumptions to derive a flood hydrograph. The peak (or volume) of the flood hydrograph can then be compared to the corresponding quantile obtained from flood frequency analyses. The model inputs associated with the greatest uncertainty can be varied within appropriate limits to ensure agreement between the selected flood quantile. It is recommended that model calibration be undertaken for a range of exceedance probabilities to ensure a consistent variation of parameters with flood magnitude. The approach is suited to ungaged watersheds using regional flood frequency methods as well as sites with limited information. The approach is considered to be particularly useful when combined with flood frequency information that uses paleoflood data.

4.4.3 Strengths and Limitations

Event-based deterministic rainfall-runoff modeling has a long track record in the engineering and hydrologic community and is a proven technique for generating design hydrographs. Reclamation uses deterministic precipitation-runoff modeling. From a technical standpoint, the approach is flexible and requires less effort than most of the more complex approaches. Model choice should be a function of the hydrometeorological and physical data available. For example, a distributed model could be applied in cases where detailed information was available; however, a simplified lumped-parameter model would be appropriate in less data-rich locations. Output from either model would be similar.

Limitations to this method arise from the need to calibrate model results to known historical flood events or to flood frequency analyses so that the AEP of the storm is the same as the resulting flood. Calibration can be difficult if the model is sensitive to many input parameters. A lack of good meteorological data can prove troublesome in developing design storms with appropriate temporal and special characteristics. Rainfall-runoff modeling in data-sparse locations may require a high level of regional data gathering and analyses to obtain the necessary hydrometeorological inputs.

4.5 Stochastic Event-Based Precipitation Runoff Modeling With the SEFM

The SEFM has been developed by MGS Engineering Consultants, Inc., in conjunction with Reclamation personnel. The SEFM was developed for analysis of extreme floods resulting from 72-hour general storms and to provide magnitude-frequency estimates for flood peak discharge, runoff volume, and maximum reservoir levels for use in hydrologic risk assessments at dams (Schaefer and Barker, 2002). The SEFM is fully described in MGS Engineering Consultants, Inc. (March 2001) and summarized in Schaefer and Barker (2002).

The basic concept of the SEFM is to employ a deterministic flood computation model and treat the input parameters as variables instead of fixed values. Monte Carlo sampling procedures are used to allow the hydrometeorological input parameters to vary in accordance with those observed in nature, while preserving the natural dependencies that exist between some climatic and hydrologic parameters. Example outputs from the SEFM are shown in figures 4-6, 4-7, and 4-8 (MGS, March 2001).

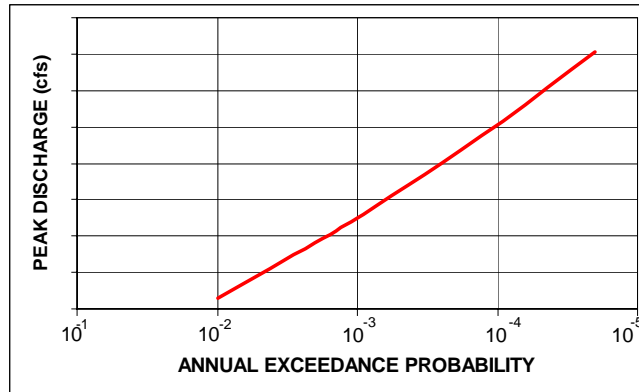


Figure 4-6.—Example of magnitude-frequency curve for peak discharge.

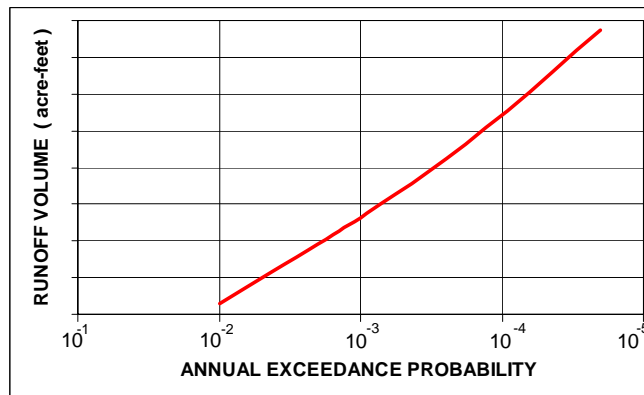


Figure 4-7.—Example of magnitude-frequency curve for runoff volume.

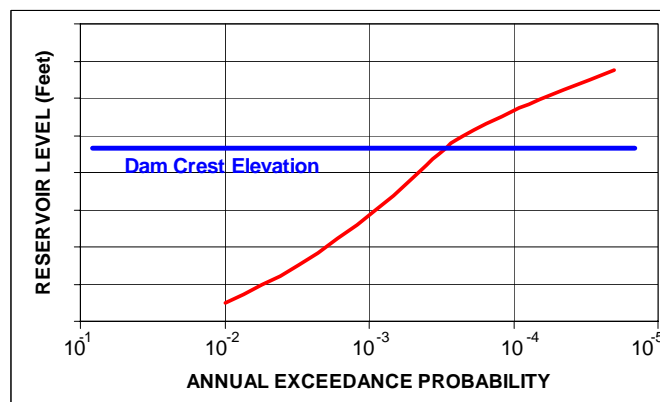


Figure 4-8.—Example of magnitude-frequency curve for maximum reservoir level.

A general flowchart for stochastic modeling with the SEFM is shown in figure 4-9. Thousands of computer simulations are conducted where each simulation contains a set of input parameters that were selected based on the historical record and, collectively, the simulations preserve the dependencies between parameters. The simulated floods constitute elements of an annual maxima flood series that can be analyzed by standard flood-frequency methods. The resultant flood magnitude-frequency estimates reflect the likelihood of occurrence of the various combinations of hydrometeorological factors that affect flood magnitude. The use of the stochastic approach allows the development of separate magnitude-frequency curves for flood peak discharge, flood runoff volume, and maximum reservoir level. Frequency information about maximum reservoir levels is particularly important for use in hydrologic risk assessments because it accounts for flood peak discharge, runoff volume, hydrograph shape, initial reservoir level, and reservoir operations. The precipitation estimates used in the SEFM are typically developed using regional precipitation frequency analysis with L-Moments (Hosking and Wallis, 1997). This approach uses a space-time substitution principle and assumes that there is sufficient precipitation data in a region to combine for subsequent model extrapolation to the probabilities of interest.

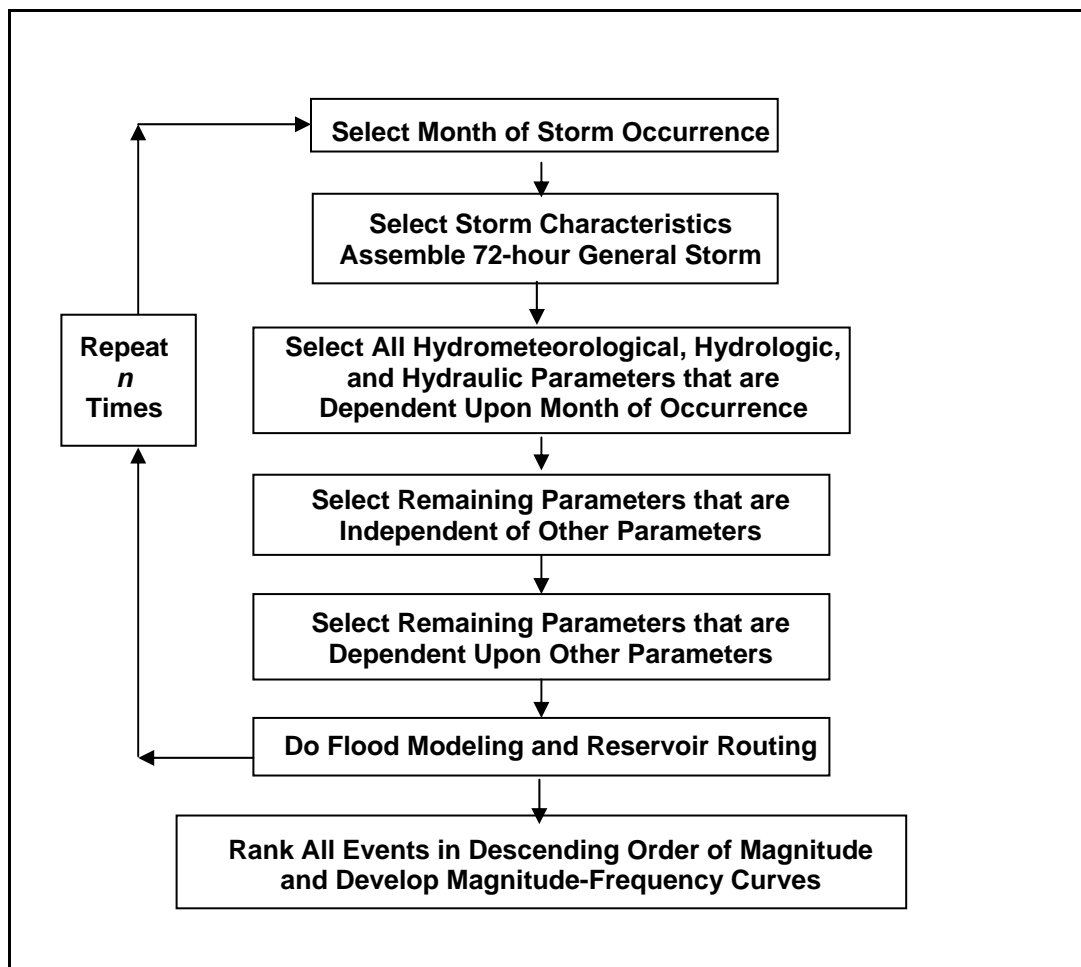


Figure 4-9.—General flowchart for the SEFM simulations (after MGS, 2001).

The general storm SEFM is composed of seven software components: data entry, input data pre-processor, multiple sample parameter test workbook, HEC-1 template file, stochastic inputs generator, HEC-1 rainfall-runoff flood computation model, and an output data post-processor. The flowchart shown below (figure 4-10) depicts the sequence of actions required for conducting the computer simulations using the software components. Each of these components is described in the SEFM Technical Support Manual (MGS, March 2001). The software (MGS, April 2001) helps the user input data, run the model, and visualize results.

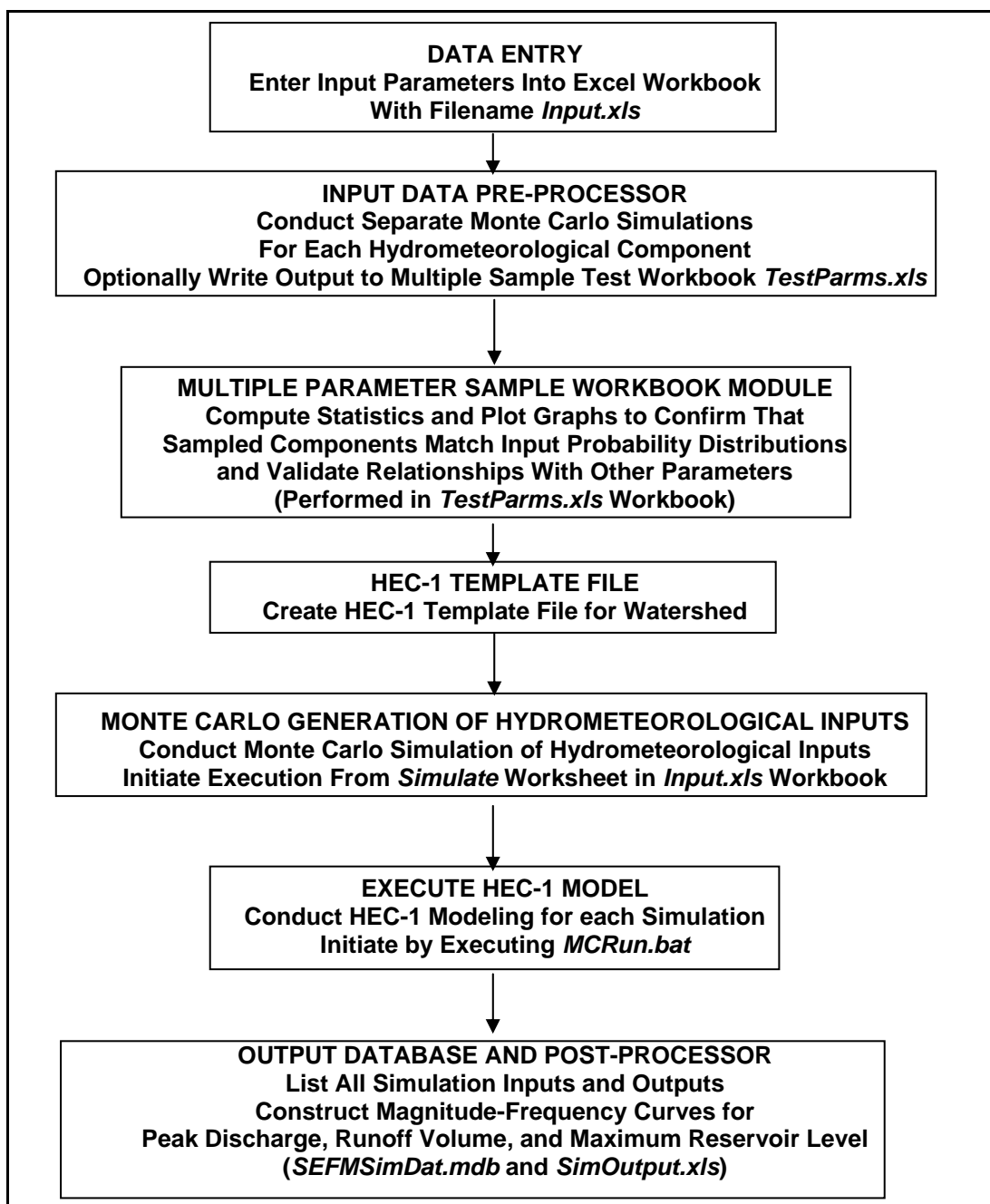


Figure 4-10.—General flowchart for the sequence of actions required for conducting the computer simulations using the SEFM.

The input processor to the SEFM is an Excel workbook. The workbook includes different spreadsheet screens for input (figure 4-11), for model execution, and for output (figure 4-12). The improvements are contained in software version 1.8 of the SEFM.

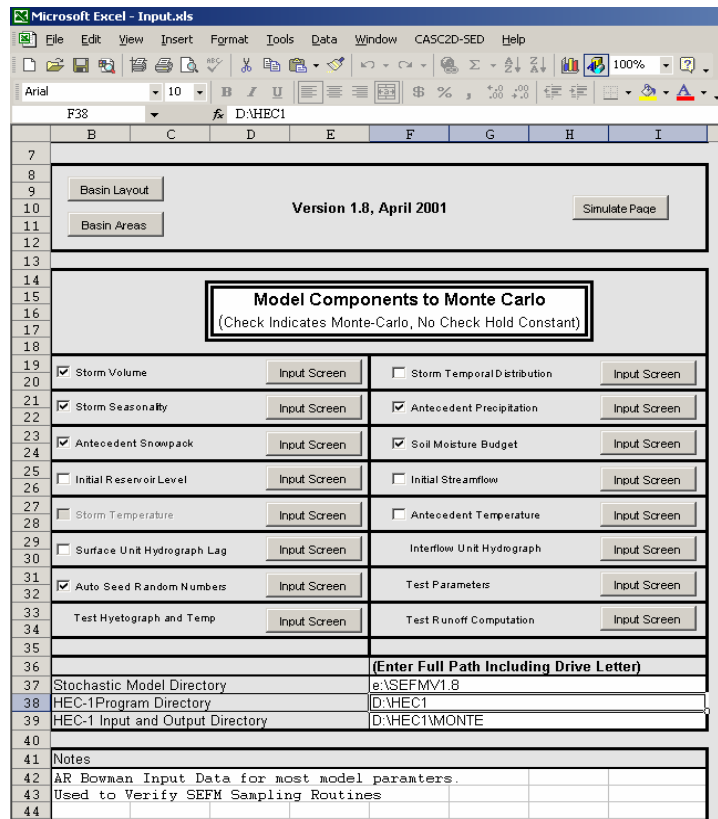


Figure 4-11.—The SEFM input screen.

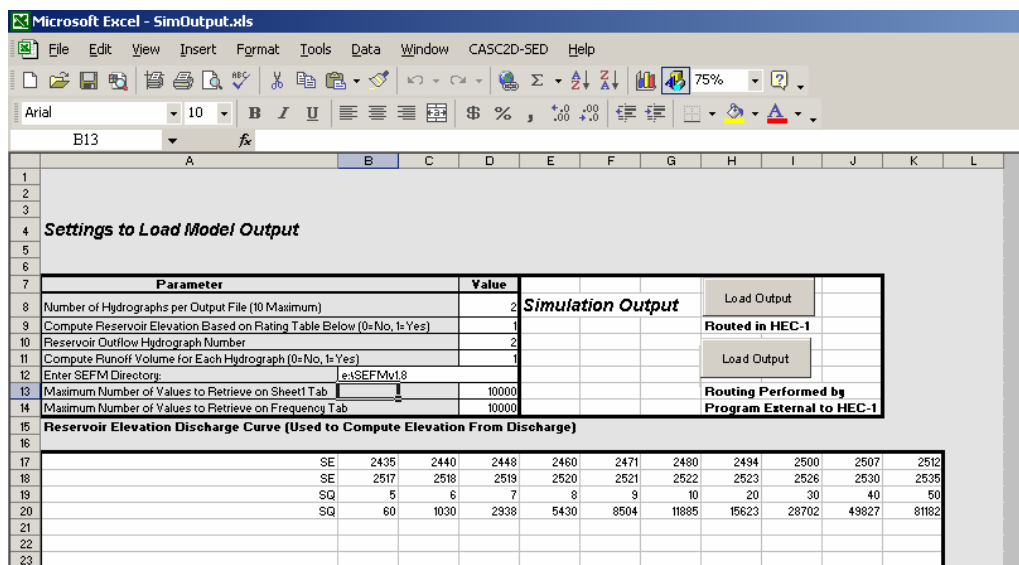


Figure 4-12.—The SEFM output simulation options.

The basic concept behind the SEFM was first used to explore PMF variability and extreme flood probability issues at Bumping Lake Dam for Reclamation (Barker et al., 1996; 1997). The prototype of the SEFM was developed for application to the A.R. Bowman watershed (Schaefer and Barker, 1997; 1998). The SEFM was subsequently generalized for use by Reclamation on other projects and applied at Keechelus and Cle Elum Dams (Bullard and Schaefer, 1999). A sensitivity analysis has been performed for A.R. Bowman Dam (MGS, January 2001) to determine the dominant features that affect model results. A brief review of the model (Singh, 1999) indicated that the model is a sound package for extreme flood modeling, and there could be some eventual improvements.

There are some current limitations to the SEFM and the applicability to certain Reclamation sites. The SEFM is currently configured for simulation of 72-hour general storms. There is no computational limit to the size of the watershed to which it can be applied. However, implicit in the development of the model is the condition that some hydrometeorological parameters are highly correlated spatially. As the watershed size increases, the requirement for high spatial correlation of multi-month precipitation and snowpack becomes more difficult to satisfy. This consideration suggests that the stochastic model is applicable to watersheds up to a nominal size of about 500 mi². For larger watersheds, the spatial variability of some hydrometeorological parameters may warrant that site-specific modules be developed to address the site-specific characteristics of the watershed under study (Schaefer and Barker, 2002, p. 732). Currently, the model does not handle thunderstorm events. Additional routines would need to be added to simulate any storms with durations that differ from 72 hours. These limitations may be relaxed as improvements are made to the model.

4.6 Stochastic Rainfall-Runoff Modeling With CASC2D

CASC2D is a fully unsteady, physically based, distributed-parameter, raster (square-grid), two-dimensional, infiltration-excess (Hortonian) hydrologic model for simulating the runoff response of a watershed subject to an input rainfall field for a particular storm event (Julien and Saghafian, 1991; Julien et al., 1995; Ogden and Julien, 2002). Major components of the model include rainfall interception, infiltration, surface and channel runoff routing using the diffusive wave method, soil erosion, and sediment transport. CASC2D is appropriate for simulating extreme floods and physically based extrapolations of frequency relationships combined with a derived distribution approach. The main differences between CASC2D and the SEFM are that CASC2D is a fully distributed model and uses hydraulic principles for runoff generation and routing precipitation excess. The SEFM is essentially a lumped model and uses the unit hydrograph as the basis for runoff and routing precipitation excess. Other differences are the infiltration models and routing mechanisms for river channels. CASC2D is also a somewhat experimental model that has not been used in extreme flood applications for dam safety, or for many applications outside academic research.

The idea and basis to use CASC2D for extreme flood modeling and prediction is centered on two concepts: (1) a derived distribution approach (e.g., Eagleson, 1972) can be used to estimate the extreme flood peak and volume probability distributions and (2) physically-based methods for flood runoff and routing provide a suitable and improved physical basis for the extrapolations of derived flood probability distributions. Ramirez (2000) summarizes the theory behind the

derived distribution approach. In the disciplines of science and engineering, relationships that predict the value of a dependent variable in terms of one or many basic (independent) variables are commonly developed. Physical systems are naturally complex. The functional form of the relationship between independent and dependent quantities or the values of the independent variables (or both) is not usually known with certainty. Techniques based on probabilistic assumptions can be used to account for this uncertainty. When the uncertainty derives from uncertainty in the independent variables, but not from uncertainty in the functional dependence, a derived distribution approach leads to the probability density function of the dependent variable. In this case, the functional form relating independent and dependent variables is assumed known with certainty. In such instances, it is possible to derive the probability density function of the dependent variable(s) from that of the independent variable(s) (Ang and Tang, 1975).

Several research applications use the derived distribution approach to estimate flood frequency curves; these show much promise. The pioneering study for flood frequency is Eagleson (1972). Bras (1990) discusses some of the potential applications of derived distributions in hydrology.

4.7 PMF Analysis Technique

The PMF is defined by Reclamation as “the maximum runoff condition resulting from the most severe combination of hydrologic and meteorologic conditions that are considered reasonably possible for the drainage basin under study” (Cudworth, 1989). Other agencies have developed somewhat different definitions, but all consider the PMF to be a “maximum runoff condition” that is “reasonably possible.”

In Reclamation practice, the basic model to convert PMP to runoff is the unit hydrograph. It is recognized that many other techniques, including sophisticated computerized models, are available for making this conversion. The unit hydrograph concept represents the modeling of the rainfall-runoff process as a linear system. The fact that the rainfall-runoff process is actually non-linear is one of the acknowledged shortcomings of the concept. However, if properly applied, the concept provides entirely satisfactory results for developing flood hydrographs resulting from extreme rainfall events.

During the period from the mid-1940s to the late 1990s Reclamation engineers designed and built most of the large storage dams currently in Reclamation’s inventory. During this period, Reclamation published three editions of the *Design of Small Dams*. This publication has served as a textbook and as a technical guide for numerous States and many foreign countries. Many States, consultants, and other agencies still use the methods and PMF philosophies expressed in the recent editions of those publications. The basic PMF methodologies described in those publications have served as the basis for thousands of dams in the U.S. and other countries designed by both Reclamation and non-Reclamation engineers. These publications are still in widespread use. Reclamation engineers often employed somewhat different techniques and nomenclature for deriving design floods, but the intent was nearly always to design for the PMF determined by the then current techniques. Some exceptions were made for small dams and diversion structures where it was believed that failure of the structure caused by overtopping would not produce any loss of life or major economic damages.

The steps involved in deriving the PMF hydrograph for a single basin have been described in many sources. The following steps are modified from the list given by the National Research Council (NRC, 1988). This list briefly summarizes the PMF calculation process currently followed by Reclamation and many other dam building agencies:

1. Divide the drainage area into subbasins, if necessary, and determine the appropriate drainage areas.
2. Derive a runoff model (unit hydrographs for Reclamation studies).
3. Determine the PMP using criteria contained in NOAA Hydrometeorological Report (HMR) series.
4. Arrange the PMP increments into a logical storm rainfall pattern.
5. Estimate for each time interval the losses from rainfall, due to such actions as surface detention and infiltration within the watershed.
6. Deduct the losses from rainfall to estimate rainfall excess values for each time interval.
7. Apply rainfall excess values to the runoff model for each subbasin.
8. Add to the storm runoff hydrograph allowances for stream base flow, runoff from prior storms, etc., to obtain the synthesized flood hydrograph for each subbasin.
9. Route the flood from each subbasin to a point of interest.
10. Compare the computed PMF peak and volume to the applicable envelope curve of peak and volume flows, if available.
11. Route the resulting inflow hydrograph through the reservoir storage, outlets, and spillways to obtain estimates of maximum storage, elevations and discharges, and durations at the dam.

Many factors influence the ultimate magnitude of the PMF hydrograph, but the intensity and duration of the rainfall are the most important. Considerable analysis and discussion of the derivation and application of PMP estimates has taken place in the past. The original definition of PMP dates back to the late 1930s. In 1981, Reclamation, the National Weather Service, and the U.S. Corps of Engineers adopted a mutually acceptable, uniform definition of the widely used term PMP. The PMP, as defined by these three agencies at that time, is “theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location at a certain time of the year.” PMP must always be termed as an estimate because there is no direct means of computing and evaluating the accuracy of the results. Since the mid-1980s, Reclamation has considered that the series of

HMRs prepared and updated by the National Weather Service provide the best estimates of PMP potential within the limits of each report. Figure 4-13 displays the current coverage of the continental United States by this series of reports.

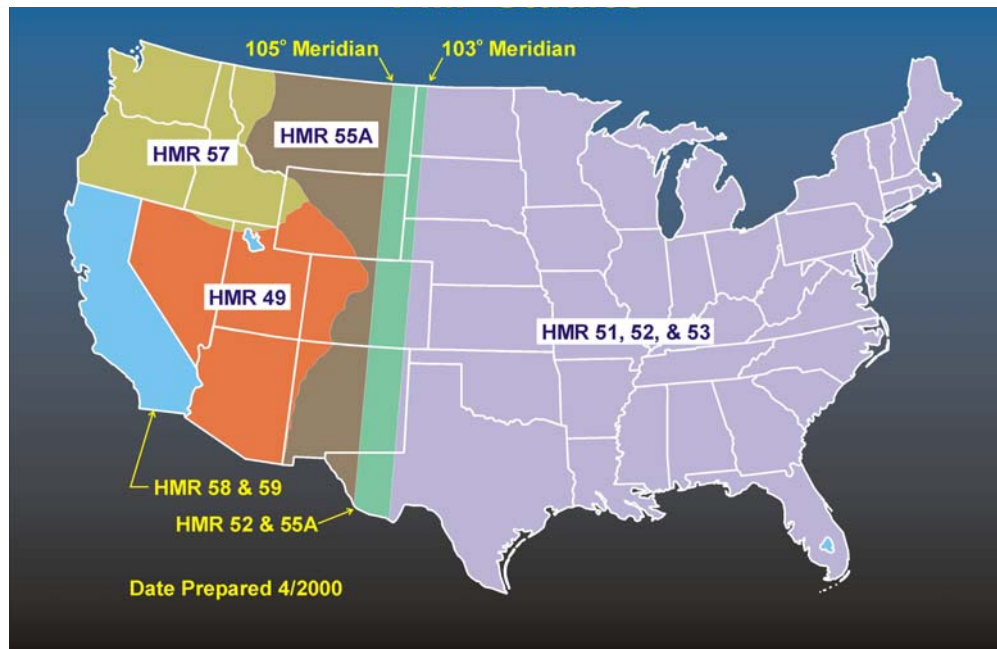


Figure 4-13.—Regions covered by generalized PMP studies.

Other aspects of the PMP to PMF conversion that are unique to Reclamation are discussed in the following paragraphs.

After determining the total PMP depths for specific time intervals from the appropriate HMR, a smooth depth-duration curve is created. This curve is then read at time intervals equal to the desired computation interval for the PMF hydrograph. Each individual precipitation total for each time increment is subtracted from the preceding time increment total PMP. The result is the incremental values of PMP to be used in the computation of the PMF hydrograph. Figure 4-14 displays a typical depth-duration and incremental value computation.

The temporal rainfall pattern most commonly used with Reclamation PMF studies places the maximum increment of rainfall at the 2/3 point of the storm and arranges the remaining increments of precipitation in descending order about this point. This distribution is applied throughout the United States and results from an examination of individual drainage area regionalized storm criteria combined with various hydrological tests. This arrangement, when combined with the unit hydrograph procedures, will produce a maximum runoff condition for the basin being studied that is still reasonable based on meteorological experience. Figure 4-15 displays this temporal distribution.

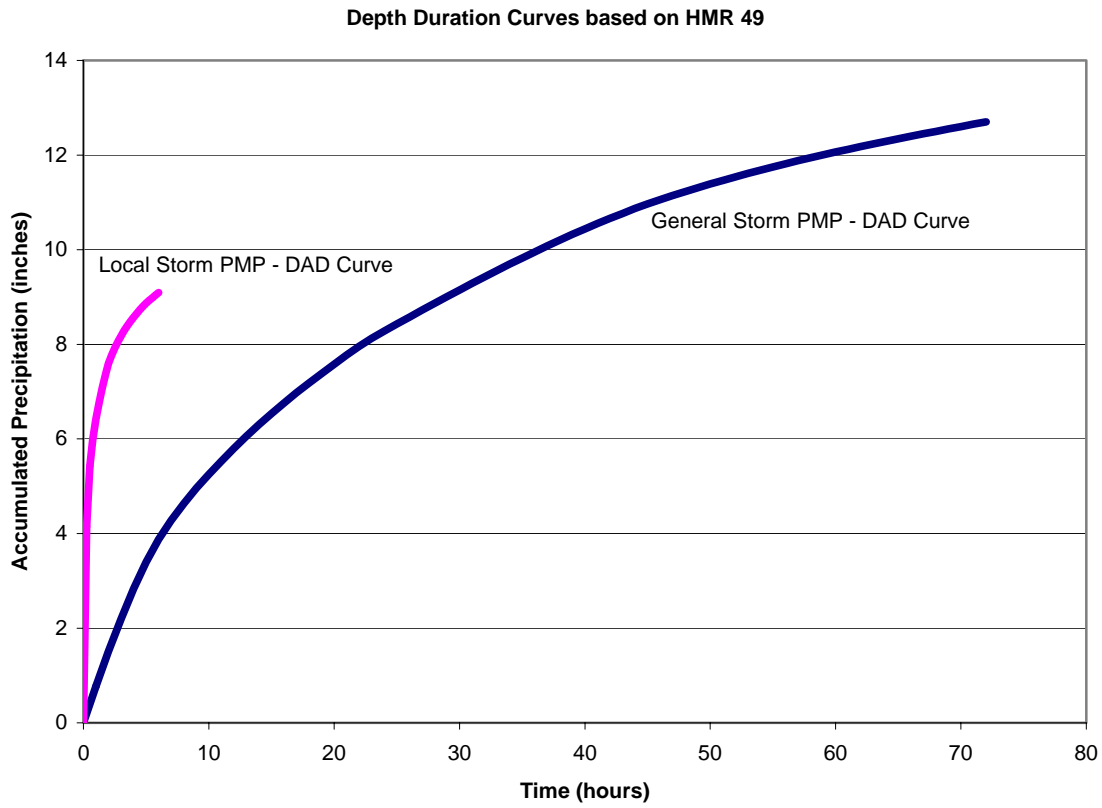


Figure 4-14.—Typical PMP depth duration curves.

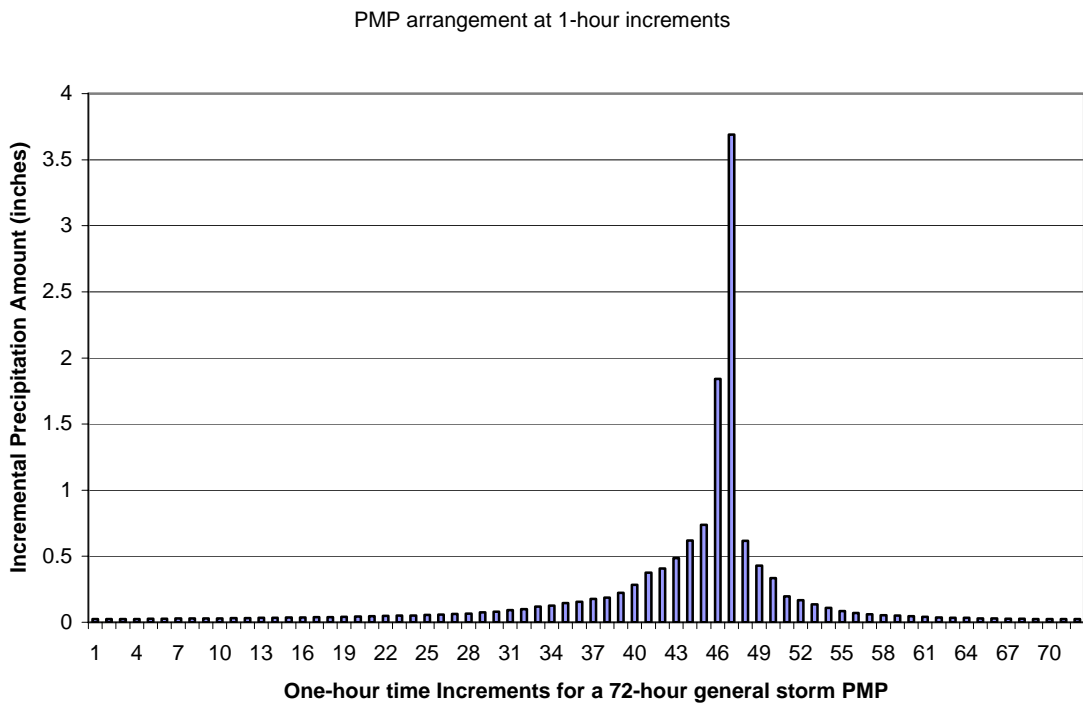


Figure 4-15.—Typical PMP temporal arrangement for a 72-hour storm.

In Reclamation PMF studies for large basins with many subbasins, consideration is given to “centering of the PMF” by allowing the spatial distribution of the PMP to vary across the entire basin. The storm is centered over a single subbasin, and the PMP for that subbasin is calculated along with the PMP for the total basin. PMP amounts over the other subbasins are calculated in a manner that preserves the total basin volume of the PMP but allows for heavier amounts over the subbasins nearest the assumed center of the storm. This calculation is termed “successive subtraction.” Volume centerings will center the entire PMP storm over the subbasin nearest to the center of the entire basin. Peak centerings will allow the PMP to be centered over the subbasin nearest to the dam. Volume centerings will typically produce a PMF hydrograph with the largest total volume. Peak centerings will typically produce a PMF with a higher peak but a lower volume. Both PMF hydrographs need to be routed through the dam.

Another unique feature of some PMF studies is the use of an elliptical storm pattern as specified in the appropriate HMR. The use of this pattern allows the PMP to vary within a large area in a manner that does not depend on subbasin sizes or shapes but, rather, on historic observations of large storms. This type of PMP calculation is most common for large general storms in the HMR 51-52 area. Local storm PMPs can also use a theoretical elliptical shape in the HMR 49 and HMR 59 areas.

Another good feature of the PMF calculations is that a series of dams all on the same river can be studied. By allowing the PMP to be centered over subbasins above each individual dam, the most severe cause of flooding at that dam can be obtained. Concurrent floods above the other dams on the same river basin can then be calculated for the same PMP event. Once the PMF hydrographs from the upstream dams are formed and routed through those structures, the complete PMF series of hydrographs for the downstream dam can be obtained. This is not always the case with methods based solely on statistical methods.

Local storm hydrographs are also computed for most Reclamation Dams if the HMR series provides specific data for deriving a separate local storm. Such storms are generally considered the thunderstorm type events and are limited in durational and areal coverage. Local storms will generally produce more intense precipitation and are usually most critical for smaller drainage basins. Local storms are considered spring or summer events and may become critical for dams and reservoirs where higher water surfaces are allowed in the spring and summer.

Lag time computations and unit hydrographs for Reclamation PMF studies have come from many past investigations of large floods. Basic unit hydrographs for sites where sufficient data were recorded were derived by standard means after subtracting out any base flows or snowmelt flows. With some knowledge of the basin average rainfall total and duration, the resulting hydrograph could be reduced to a unit hydrograph. The unit hydrographs were then put into a dimensionless form unique to Reclamation, with the flow ordinate being expressed as a dimensionless volume of flooding divided by the amount of flooding that could be expected from 1 inch of excess runoff in 24 hours (expressed as $\text{ft}^3/\text{s-day}/\text{ft}^3$ or often abbreviated as q). The time ordinate was expressed as a percent of the basin lag time plus 1/2 of the unit duration for that basin. Figure 4-16 is a sample of such a dimensionless graph. Over 60 dimensionless graphs are available to Reclamation engineers, but this number has been reduced to only 6 for publication in the *Flood Hydrology Manual* and in the latest edition of *Design of Small Dams*.

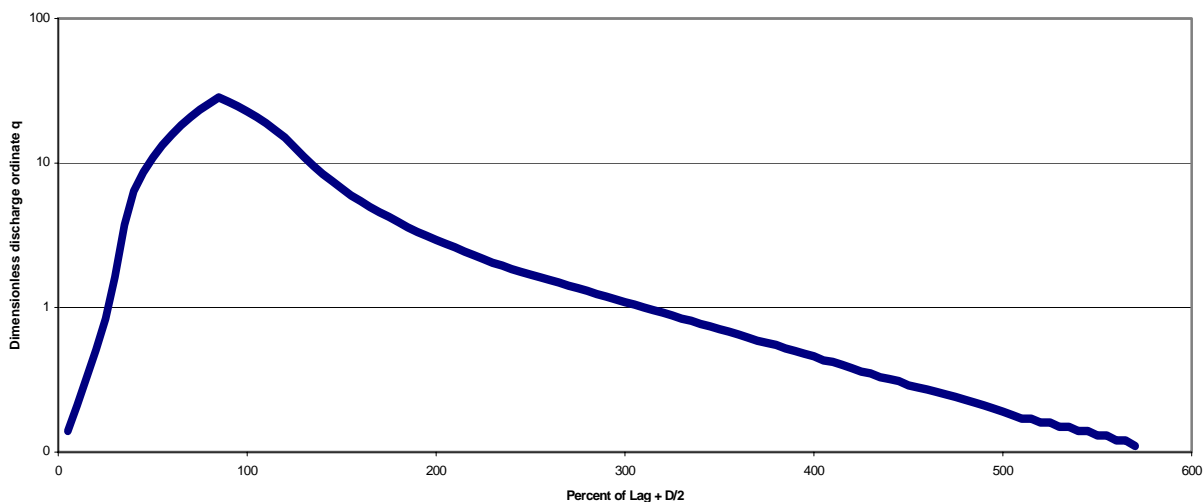


Figure 4-16.—Reclamation dimensionless graph.

The physical definition of the lag time used by Reclamation in PMF studies is the time from the mid-point of a unit of excess rainfall to the mid-point of the total volume of the resulting runoff hydrograph from that single unit of excess rainfall. With this definition in mind, much of the historic flood data collected by Reclamation engineers in the 1940 to 1970 time period was used to define relationships between the physical measurement of the length of the main channel, L (mi); the length to a point on the main channel opposite of the basin centroid, L_{ca} (mi); and the basin slope S (ft/mi). Such data were plotted on log-log graphs, and straight lines on these plots were drawn based on different basin vegetation and soil conditions. Figure 4-17 is a sample of such a plot from the Reclamation *Flood Hydrology Manual*.

The locations of the straight lines on these plots reflect the basin vegetation, soils, overall slope and, to some degree, the basin area and type of PMF being calculated. Such graphs also represent floods from much smaller events than the PMF, and some judgment must be applied to the lag numbers. PMF conditions will usually require somewhat shorter lag times than computed for more common flood events. Lag times for Reclamation PMF studies are based on this type of historical knowledge, reviews of other PMF studies prepared in the area, and reviews of pertinent basin topography, soils, and land use maps. For final design-level PMF studies, field trips to the basin in question by a qualified hydrologist are required to ensure that the information from the maps and other calculated basin parameters is valid. Generalized equations based on the lag curves such as displayed above are available to provide a computational procedure for calculating a lag time for each subbasin and storm type in the basin being studied.

With a computed lag time, drainage area, and dimensionless graph selected, the final unit hydrograph for each subbasin can be calculated. Procedures for this calculation are given in Reclamation's *Flood Hydrology Manual* and in *Design of Small Dams*.

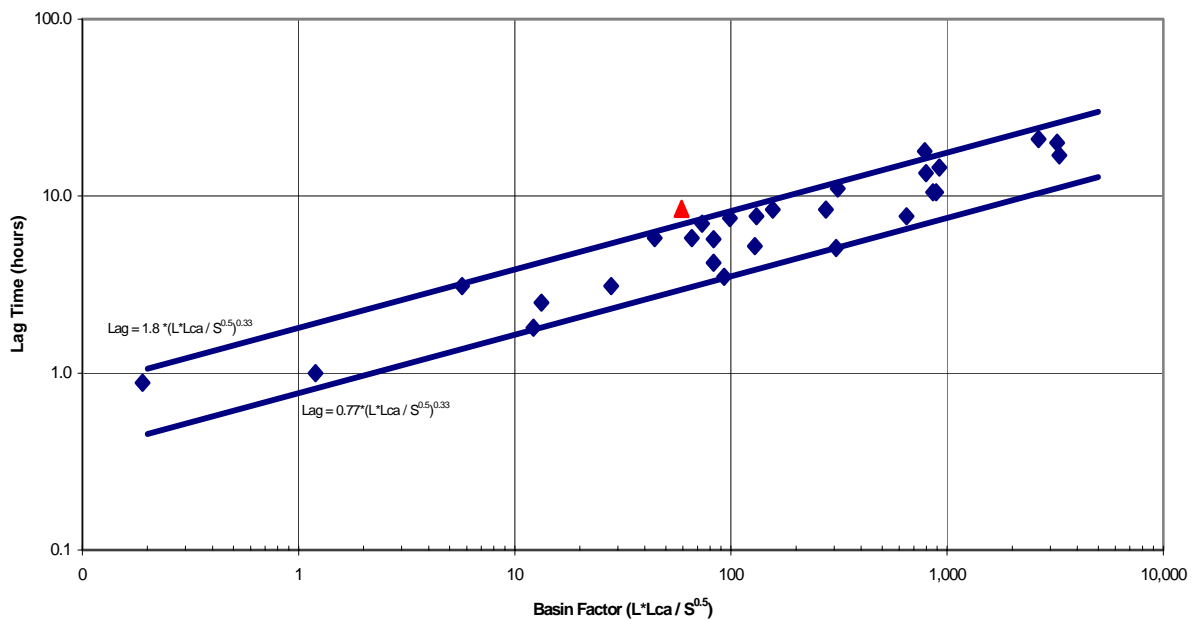


Figure 4-17.—Lag time versus basin factor data for Great Plains Region (from Reclamation *Flood Hydrology Manual*, 1989).

Loss rates for Reclamation studies always assume saturated basin conditions caused by antecedent flooding. Most often, the loss rates are derived by studying basin soils maps available from previous Soil Conservation Service (SCS) soils mapping reports or, more recently, by using the NRCS STATSGO computerized soils database. The SCS divided the soils into four basic hydrologic groups for rainfall-runoff studies. These groups range from very porous sandy or gravely soils with high infiltration rates to very tight clay soils with very low porosity and low loss rates for rainfall-runoff studies. The four soil categories have been assigned ranges of minimum loss rates. By determining the percentage or amount of the different soils in each subbasin, an average minimum loss rate can then be selected or computed from the published minimum loss rate ranges. Initial loss rates are seldom used in Reclamation PMF studies. The definition of the PMF requires the assumption of saturated soil conditions before the onset of the PMP. Under such an assumption, the initial loss rates will be completely satisfied, leaving only a minimum constant loss rate to be considered. It is recognized that more sophisticated loss rate algorithms exist, but for the PMF computation and assumptions, the use of a constant loss rate based on minimum losses for the upper soil layers in the basin is considered adequate.

Antecedent flooding is also considered for general storm PMF hydrograph computations in Reclamation studies. The general storm most often occurs at a time of the year when flooding is most likely to occur in the basin. It is likely that previous storms or a melting snow pack will provide some antecedent streamflow before the onset of the PMP. For basins without significant snowmelt contributions, the antecedent flood comes from a study of applicable streamflow records or, if needed, rainfall-runoff analysis. The desire is to derive a 100-year event by statistical means or by rainfall-runoff modeling if limited stream gage data are available. This derived flood hydrograph is then placed a number of days in front of the onset of the PMP. In

most parts of the central United States, there is a 3-day separation between the peak of the antecedent flood and the start of the PMP. This length of time between the antecedent rain flood hydrograph peak and the start of the PMP will vary near coastal areas. Figure 4-18 displays this type of application.

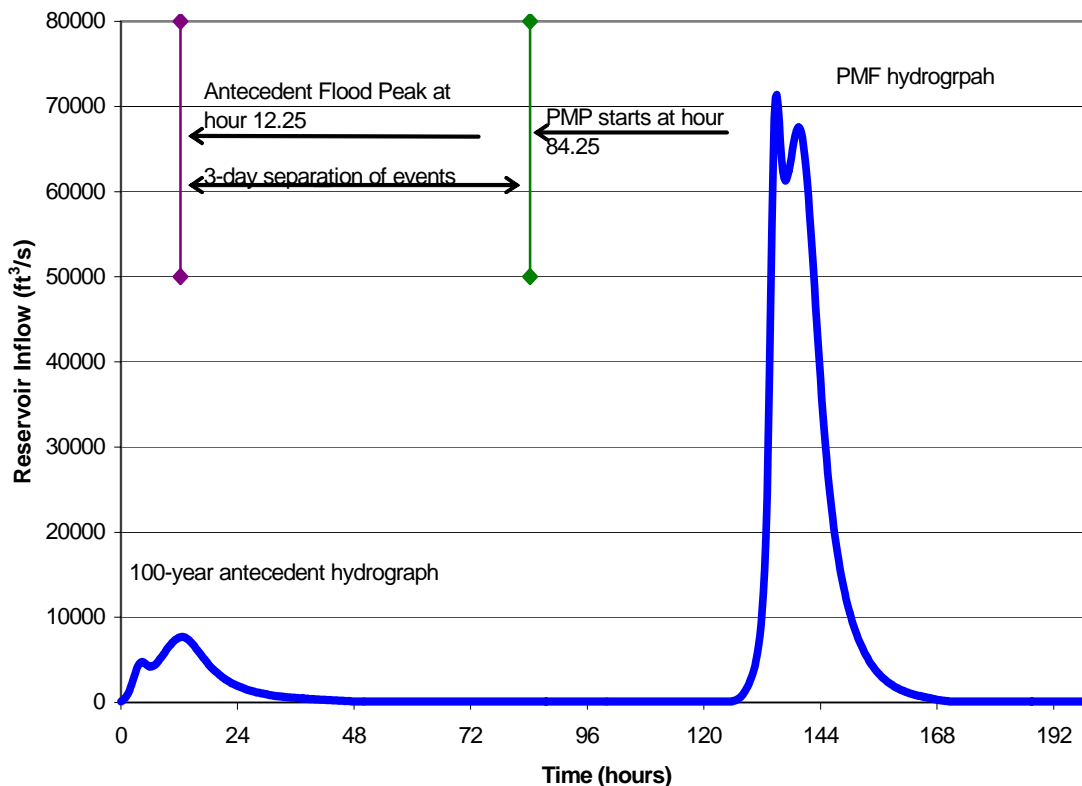


Figure 4-18.—PMF with antecedent 100-year floods.

In areas where snowmelt adds to the volume of the flooding, the antecedent flood in Reclamation PMF studies is often derived from an analysis of stream gage data. The data are limited to the season when snowmelt adds to the flooding. A large historic snowmelt flood may be selected or a statistical analysis of the volume of flooding for several different durations may be undertaken. The result is a 100-year flood volume for a duration longer than the base length of the calculated PMF rain flood hydrograph. If a historic flood event is selected, the daily flows for the hydrograph will be adjusted by a ratio such that the resulting volume for the selected duration is a 100-year volume that can be determined by a statistical analysis of appropriate gage records. The historic flood hydrograph may also be rearranged in time to provide a more normal hydrograph shape. If no historic flood hydrograph is appropriate, then the antecedent snowmelt hydrograph may be derived as a balanced hydrograph. If a snowmelt hydrograph is used as an antecedent flood, Reclamation places that hydrograph under the rain-generated portion of the PMF such that the peaks will exactly coincide. Figure 4-19 displays such a derived historic flood hydrograph for a 15-day antecedent event. Figure 4-20 displays the placement of the derived balanced hydrograph to coincide with the peak of the rain-generated PMF hydrograph for a basin.

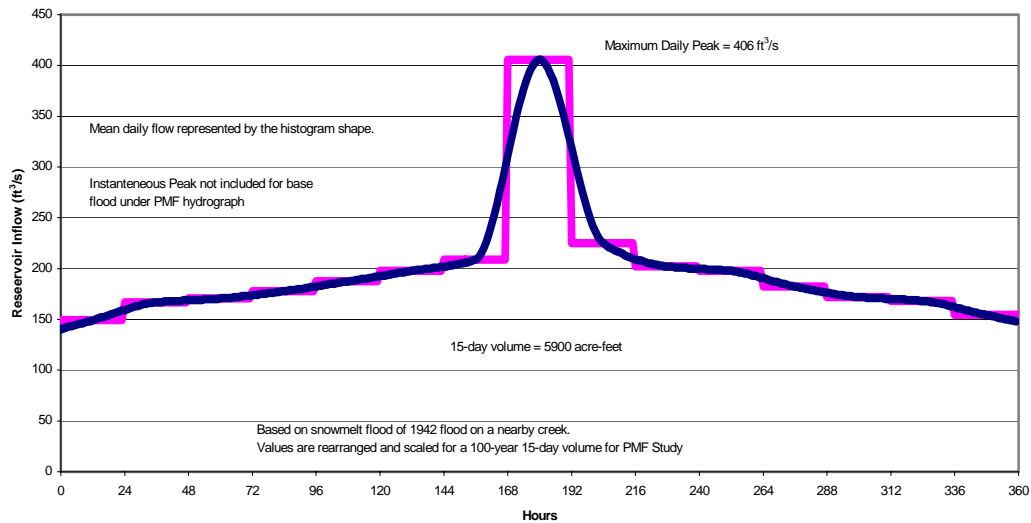


Figure 4-19.—100-year 15-day derived snowmelt flood from an historic snowmelt flood event.

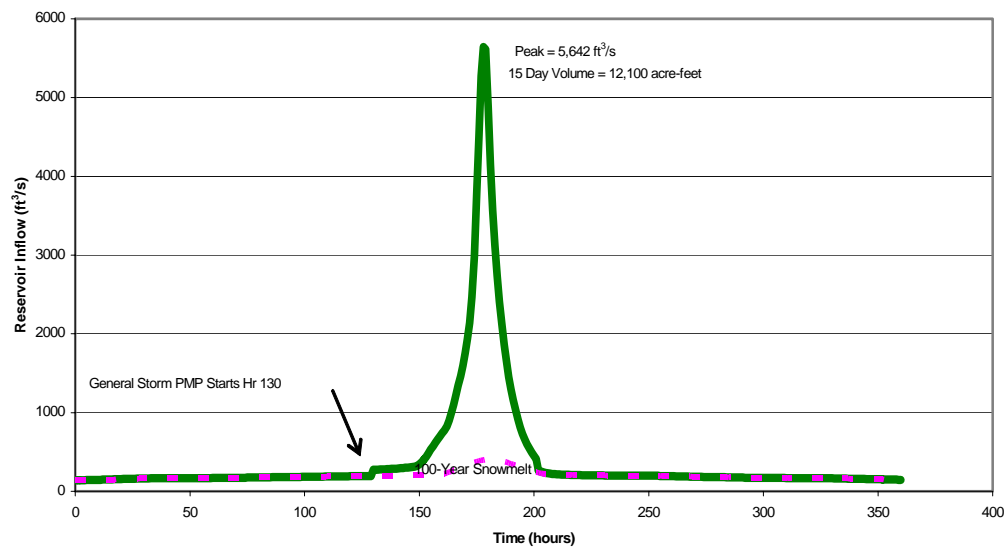


Figure 4-20.—General storm PMF with 100-year snowmelt antecedent flood.

When a snowmelt flood is required, the loss rate for the measured or estimated snow-covered areas in the basin is reduced to a constant 0.05 inch per hour. This is assumed to account for the fact that the 100-year snowmelt is already in progress at the start of the PMP and most of the available constant loss rate will have already been taken up by the melting snow.

In areas where deep snow packs are known to occur, such as the high Sierra Mountains in California, Reclamation will use a snowmelt computation program. This program will account for the effect of rain falling on a snow pack that may not be fully ripened and ready to melt at the start of the PMP. The snowmelt program requires additional meteorology data such as wind and temperature sequences compatible with the PMP derivation. The HMR series will provide good

information in this regard. Additional study of the historic snow gage records in or near the basin being studied is also required. The depth and initial density of the snow pack at various elevations in the basin are used in the computations. Such snowmelt computations provide additional depths of water to be added to the arranged PMP sequence. The combined snowmelt and PMP depths are then used with the unit hydrograph procedures to derive the desired PMF hydrographs.

Routing of floods between subbasins is also required. If sufficient data for several large flood events are available, routing coefficients for the Muskingum Method can be derived. Usually, little information is available for large floods at more than one gage in the basin being studied. In lieu of Muskingum routing, the most common procedure is an analytical procedure referred to as the Tatum Method. This method requires only an assumed travel time between the various subbasins and will attenuate the routed hydrographs by a simple arithmetical computation. This procedure does not affect the volume of the flooding, but it does have some impact on the PMF peak.

Envelope curve comparisons of the PMF peak and sometimes a volume comparison are also made. The comparison is only to ensure the hydrologist that the PMF peak or volume is not below experienced flood levels in the region of concern. If the PMF peak falls below an envelope curve of peak flows for a nearby region, some additional thought must be given to the derivation of the PMF or the derivation of the envelope curve values. There are no rules about how far above the envelope curve a PMF peak should be. This would depend on the derivation of the envelope curve. Envelope curves covering large regions might have additional high-peak flows and the PMF comparison would be closer. Smaller regions would most likely have PMF peaks much higher than the envelope curve.

5. Characterization of Hydrologic Hazards

Since no single approach is capable of providing the needed characterization of extreme floods over the full range of AEPs required for risk assessment, results from several methods and sources of data should be combined to yield a hydrologic hazard curve. The application of several independent methods applicable to the same range of AEPs will increase the credibility and resulting confidence of the results. The previous section of this report describes the possible approaches for estimating the hydrologic hazard.

To compute an initial hydrologic hazard curve for risk assessment, Reclamation uses all the available studies for the site of interest in the initial characterization. Often, the PMF study is the only hydrologic study available before the start of a probabilistic investigation. If a dam passes the PMF or a flood with an AEP low enough to meet dam safety risk criteria, no further studies may be needed.

The remainder of this section of the report describes Reclamation's approach toward characterizing the hydrologic hazard for a particular site of interest. The process begins with

an initial characterization and progresses through more detailed studies if the need arises. If detailed studies are already available for a particular site, these studies would be summarized and would form the basis of the initial characterization.

5.1 Initial Characterization of Hydrologic Risk

The initial characterization of hydrologic risk is first provided in terms of estimated peak discharges, which are then used to estimate flood volumes. Peak discharges are estimated using a mixed-population model, and the results are subsequently input to a linear scaling algorithm of volume critical hydrographs. The initial characterization is usually accomplished with minimal effort, about 15 staff days or less.

The mixed-population model used to estimate peak discharges is described by an LP-III distribution for return periods from 1 to 100 years and an LN-2 distribution for return periods greater than 100 years. Historical data are used to calibrate the LP-III model, and an informed hydrologic estimate of a single flood potential point (SFPP) with a return period of greater than 100 years must be made. The fitting of the LP-III model to the historical data is carried out using the Method of Moments (MOM) with which a regional skew may be used to fix or weight the distribution. The fitting of the LN-2 distribution is carried out analytically between the 100-year flood estimate and the SFPP. The value of the SFPP may take the form of a paleoflood non-exceedance bound, a paleoflood estimate, a historical data point with an estimated return period greater than 100 years, or any other estimate of a flood with a return period greater than 100 years believed to characterize the extreme values of the flood potential. Once the initial data requirements are met, a list of peak-discharge estimates is calculated and the results are provided in tabular as well as graphical format (table 5-1, figure 5-1). The analysis is extended beyond the SFPP to floods with AEPs as small as 10^{-8} , but peaks are not allowed to exceed the PMF for the site of interest.

Table 5-1.—Example peak-discharge estimates in tabular form

Annual exceedance probability	Return period	Peak discharge estimate (ft ³ /s)
0.01	100	26,500
0.005	200	31,100
0.002	500	37,700
0.001	1,000	43,200
0.0005	2,000	49,200
0.0002	5,000	57,700
0.0001	10,000	64,800
0.00005	20,000	72,300
0.00002	50,000	83,200
0.00001	100,000	92,000
0.000001	1,000,000	126,000
1E-07	10,000,000	167,800
1E-08	100,000,000	218,800

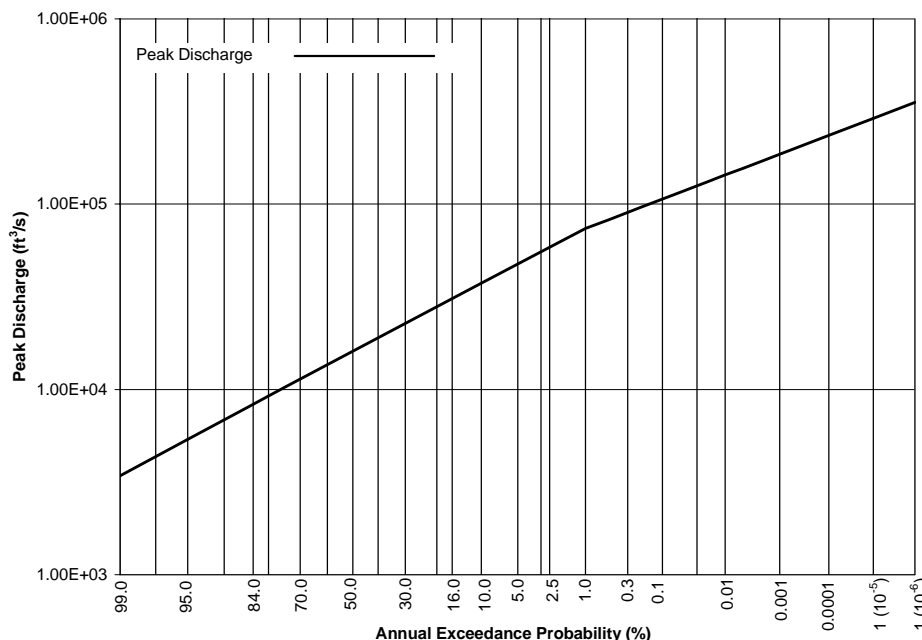


Figure 5-1.—Example of peak-discharge estimates in graphical form.

To make a volume estimate, a time series flood hydrograph estimate is required. The time series flood estimate may be a historical flood hydrograph or a PMF hydrograph; either must be believed to describe the rainfall-runoff response of the basin of interest at a wide range of return periods. Flood hydrographs are linearly scaled so that the peaks match the estimates from the peak-discharge analysis described above. When the length of data permits, 1-, 3-, 5-, 7-, and 15-day volumes are calculated as the maximum volume of water transported during the desired continuous time period.

When a rain-on-snow PMF hydrograph or rain-on-snow historical hydrograph is used as an input, the analysis technique requires two hydrograph ordinates for each time period – one for the snowmelt portion and one for the rainfall portion. Only the rainfall portion of the hydrograph is scaled to achieve the desired peak. The snowmelt portion of the hydrograph remains intact. For all other input hydrographs, the entire hydrograph is scaled linearly to match peak estimates. When volume estimates are calculated, the results are provided in tabular as well as graphical format (table 5-2, figure 5-2). Graphical results (peak and volume flows) can be provided within a combined chart (figure 5-2) or as two separate charts, as desired by the client (figure 5-1 and figure 5-3).

The major assumption involved with scaling PMF hydrographs to various return periods is that the dimensionless hydrograph used to develop the PMF is appropriate to describe the rainfall-runoff response of the basin at all return periods. Because the generation of a hydrograph from a dimensionless hydrograph involves only the convolution with time distributed rainfall rates, assumptions are further made that the rainfall spatiotemporal distribution does not change with flood magnitude and soil characteristic response is linear in nature. The latter assumption may be appropriate when floods of interest begin with saturated ground conditions. When these

Table 5-2.—Example of combined peak-discharge and volume estimates in tabular form

RETURN PERIOD (yrs)	PEAK DISCHARGE (cfs)	1-Day Volume (acre ft)	3-Day Volume (acre ft)	5-Day Volume (acre ft)	7-Day Volume (acre ft)	15-Day Volume (acre ft)
100	73,890	44,300	72,530	84,940	101,130	137,590
200	83,220	49,740	80,900	94,070	109,920	146,730
500	96,140	57,260	92,470	106,690	122,070	159,360
1,000	106,370	63,220	101,640	116,700	131,710	169,380
2,000	117,040	69,440	111,200	127,120	141,740	179,810
5,000	131,840	78,060	124,450	141,590	155,670	194,290
10,000	143,590	84,910	134,980	153,070	166,730	205,790
20,000	155,840	92,040	145,950	165,050	178,260	217,770
50,000	172,830	101,940	161,180	181,650	194,250	234,400
100,000	186,310	109,800	173,250	194,830	206,940	247,590
1,000,000	235,220	138,290	217,070	242,630	252,970	295,450
10,000,000	291,000	170,790	267,040	297,150	305,460	350,020
100,000,000	354,340	207,690	323,780	359,050	365,060	411,990

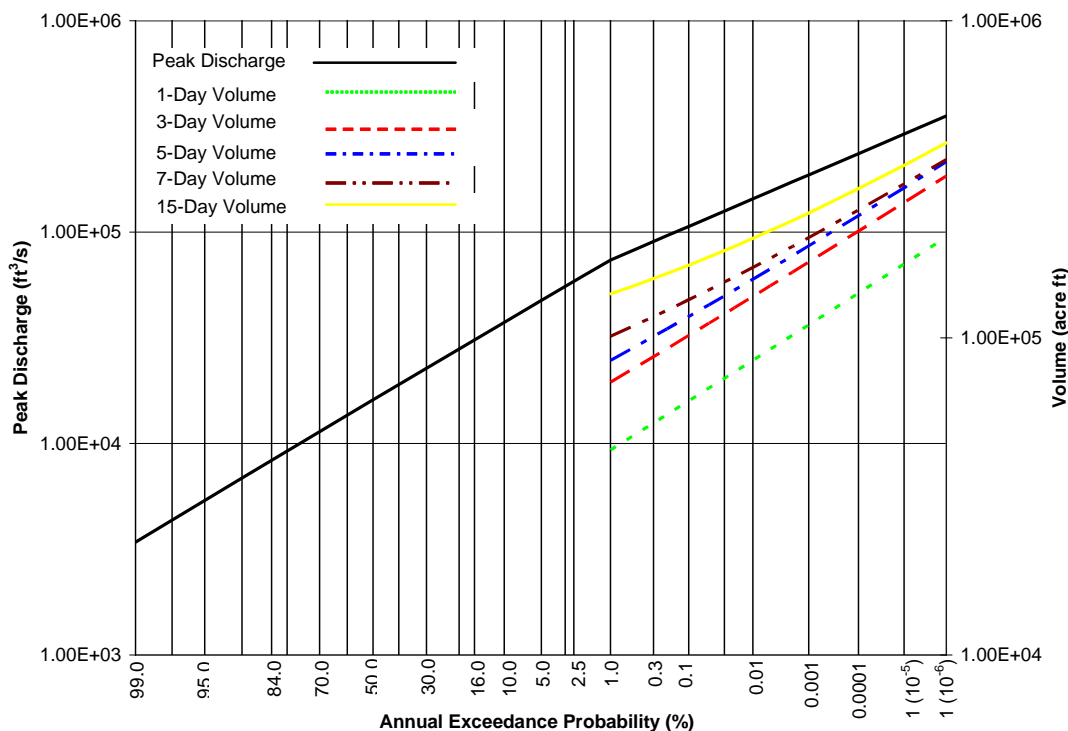


Figure 5-2.—Example of combined peak-discharge and volume estimates in graphical form.

assumptions are violated, predictions may be inaccurate by orders of magnitude. Similar assumptions can be made when scaling a historical hydrograph. These assumptions are mainly that the storm that caused the flood is similar in temporal and spatial characteristics to the storm that would cause a flood of a different return period and that the initial soil conditions are also

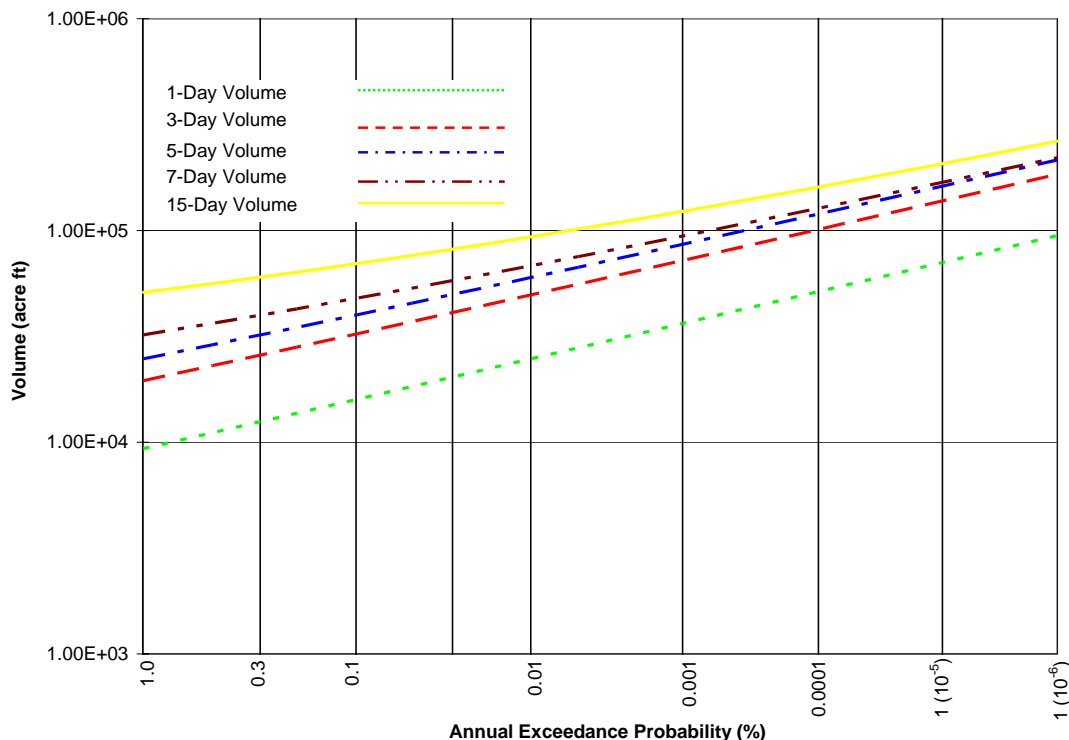


Figure 5-3.—Example of volume only chart.

similar. Soil infiltration responses are known to be highly non-linear in the unsaturated condition; therefore, as with the PMF scaling, it is the initial condition for which the soils are saturated that this type of scaling is most appropriate.

Reclamation uses the PMF as the upper limit of flood potential at a site for storm durations defined by the PMP. If peak flows or volumes calculated using probability or statistical-based hydrology methods exceed those of the PMF, the PMF is used in evaluating the hydrologic risk and as a theoretical and practical upper limit to statistical extrapolations. If the PMF has been properly developed, it represents the upper limit to runoff that can physically occur at a particular site.

Hydrograph ordinates for the peak-scaled hydrographs calculated during the volume analysis may also be provided at client request or as a standard form of initial hydrologic risk analysis. Ordinates for the scaled hydrographs have the same duration and number of ordinates as hydrograph data provided for input to scaling analysis, thus requiring no interpolation. The scaled hydrographs have an AEP associated with them, so they are suitable for use in reservoir routing studies and for dam safety risk assessment.

5.2 Detailed Hydrologic Studies

After the initial characterization of hydrologic risk, more detailed hydrologic studies may be necessary to better define the hydrologic problem, reduce uncertainty, and develop solutions.

Flood characterization for risk assessments uses the length of the data record and other characteristics of the data to determine the credible extrapolation limits used in the flood frequency analysis. Because Reclamation risk assessments may require estimation of floods with AEPs of 1 in 100,000,000, extrapolation of flood frequency relationships is required well beyond the limits warranted by the data.

The uncertainties associated with these flood estimates are likely to be substantial and an important attribute to convey into the risk assessment. Flood characterization should include a “best estimate” of the AEP of floods of different magnitudes and a description of the uncertainty in such results. Such uncertainties need to be honestly represented and considered throughout the risk assessment process.

If the initial characterization is insufficient to define and quantify the hydrologic dam safety issues, further analyses are required. These studies may become necessary to address reservoir routing effects of upstream dams, reduce uncertainty in the flood estimates, verify statistical results before committing large capital expenditures to dam safety modifications, etc. When planning the next study, the goal is to achieve a balance between the amount of hydrologic analysis needed to address the issues and the level of effort required to conduct the study. Generally, studies progress from those requiring a low level of effort to those requiring a higher level of effort. As the studies get more detailed, the results should become more precise and contain less uncertainty.

Since each study site is different, no single approach can be identified to address all hydrologic issues. The method chosen should consider climatic and hydrologic parameters, drainage area size, amount of upstream regulation, data availability, and level of confidence needed in the results. The previous chapter of this report described the methods available in Reclamation to develop hydrologic hazard curves.

Hydrologic studies usually will proceed from the initial characterization to an analysis using either the GRADEX or Australian Rainfall-Runoff Method. The advantages of these approaches are that they use regional rainfall data in developing flood estimates and yield flood hydrographs. Both approaches work best on small drainages (less than 2,000 mi²), and the Australian Rainfall-Runoff Method works better if snowmelt floods are important.

The SEFM and CASC2D are more sophisticated hydrologic modeling techniques used to address hydrologic issues driven by large potential dam failure consequences. These approaches are more suitable for final design studies. They also allow for better quantification of hydrologic uncertainty. These approaches require a lot of data and are generally more costly than the others.

When multiple methods have been used to determine the hydrologic hazard, sound physical and scientific reasoning for weighting or combining results is needed. Clearly, a measure of judgment is required to ensure that appropriate information is included in the dam safety decisionmaking process. Reclamation develops various weighting schemes to evaluate the results of multiple analyses and uses a team of hydrologists to select the one that best characterizes the hydrologic conditions for the site of interest. The selection is based on the experiences of the team members and the assumptions used in each of the analyses. The A.R. Bowman case study that follows illustrates this approach.

6. Case Studies

Three case studies are presented to illustrate the use of several methods. The study sites are Los Banos, Fresno, and A.R. Bowman Dams. Each study begins with an initial characterization. Different solution techniques were used to answer followup questions for each of the following case studies.

6.1 Los Banos Dam

Los Banos Dam is an earthfill structure with an uncontrolled spillway. It was completed between 1964 and 1965. This dam is located 7 miles southwest of the town of Los Banos, California, on Los Banos Creek. The drainage basin upstream from the dam has a total area of approximately 156 mi². The spillway crest elevation is 353.5 feet and the dam crest elevation is 384.0 feet. The spillway discharge capacity at the dam crest elevation is 11,800 ft³/s. The total capacity of the dam at water surface elevation 353.5 feet is 34,600 acre-feet. The active capacity between elevations 296 and 353.5 feet is 26,300 acre-feet. The current PMFs for Los Banos Dam were computed in 1996 (Bureau of Reclamation, 1996a). The results of these PMF studies are summarized in table 6-1.

Table 6-1.—Probable maximum flood summary for Los Banos Dam, California

Flood Type	HMR	Peak Inflow (ft ³ /s)	Volume (acre-feet)	
			With 100-year antecedent storm event	Without 100-year antecedent storm event
General storm	58	75,800	146,700 (7 day)	138,000 (4.3 days)
Thunderstorm	58	75,200	27,400 (24 hr)	—

The PMFs were computed using a standard storm arrangement. Routing results indicate that Los Banos Dam would be overtopped by a flood with a magnitude about 37.6 percent of the general storm PMF (Bureau of Reclamation, 1996b). The general storm PMF would overtop the dam by 6.2 feet for 30 hours. The thunderstorm PMF does not overtop the dam.

Because Los Banos Dam is overtopped by the general storm PMF, flood probabilities are needed to understand and quantify the risk of overtopping and hydrologic hazard for dam safety. A flood frequency and volume frequency analysis was completed using the proposed hydrologic hazard curve procedures.

6.1.1 Example of Los Banos Hydrologic Hazard Curves Using Proposed Procedures

A recent flood study, including preliminary paleoflood data, peak-flow frequency, and hydrographs, was completed for a risk analysis (Weghorst and Klinger, 2002). The streamflow and paleoflood data from that study are summarized below and then used to compute a hydrologic hazard curve.

Because streamflow gage records for Los Banos Creek at Los Banos Dam were limited, the records from the gaging station on Orestimba Creek, near Newman, California (U.S. Geological Survey [USGS] station No. 11274500), were used in this example. This gage is located just north of Los Banos Dam, has a drainage area of about 134 mi², and provides 71 years (1932 to 2002) of unregulated peak discharge records. The peak discharge data were adjusted by the square root of the drainage area ratio (Cudworth, 1989); this factor was $(156/134)^{0.5} = 1.078$.

There are several sources of regional and reconnaissance-level paleoflood data that may be applied to estimates of flood frequency at Los Banos Dam including reconnaissance-level data on Los Banos Creek and supporting data from site-specific soil stratigraphic information on the Cantua Stream Group, located about 55 miles south of Los Banos Creek (Weghorst and Klinger, 2002). The hydrometeorologic setting for the Cantua Stream Group and Los Banos Creek, in general, appears to be very similar to each other; however, no field verification was performed to confirm this. Each of the basins is located on the eastern flank of the central Coast Ranges, and the drainage basin areas are close enough in size that differences in their runoff characteristics would be negligible given similarities in rock types, basin aspect and slope, average elevation, and vegetation and land use. Therefore, the paleoflood peak discharge bounds and age estimates from the Cantua Stream Group are believed to be applicable to Los Banos Creek (Weghorst and Klinger, 2002). Paleoflood bounds were established simply by scaling (Cudworth, 1989) the peak discharges for paleoflood data on the Cantua Stream Group to the Los Banos basin. It appears that peak discharges in the range of 42,000 to 60,000 ft³/s on Los Banos Creek have not been exceeded in the last 1,800 – 2,800 years.

A hydrologic hazard curve for Los Banos Dam was constructed using the proposed techniques outlined in section 0. Peak flow and paleoflood data and a PMF general storm hydrograph were used to estimate the hazard curve (figure 6-1). Estimated peak flows and volumes can exceed the PMF peak and volume at this site (table 6-1 and table 6-2). Therefore, the PMF should be considered as an upper limit for design and risk analysis (Bureau of Reclamation, 2002).

Hydrographs for these floods were then routed through Los Banos Reservoir and spillway. The results from this initial hydrologic hazard curve characterization and flood hydrograph routing indicate that Los Banos Dam may potentially be overtopped by a flood with a return period of about 2,800 years (based on peak flow). Using these results, Los Banos Dam does not meet Reclamation hydrologic hazard criteria for overtopping because it does not pass a 10,000-year flood (at a minimum). Because this dam does not meet Reclamation criteria, additional studies for Los Banos are required to further assess the flood risk, estimate the need for any structural modifications, and determine suitable probability-based design hydrographs for any potential modification or design alternatives. Currently, a detailed flood hydrology study is in progress to further assess the flood risk at Los Banos Dam.

6.2 A.R. Bowman Dam

The case study for A.R. Bowman Dam is an example of a project where several hydrologic studies have been completed. The dam is an earthfill structure with an uncontrolled spillway. It was completed in 1961. This dam is located about 20 miles upstream from Prineville, Oregon,

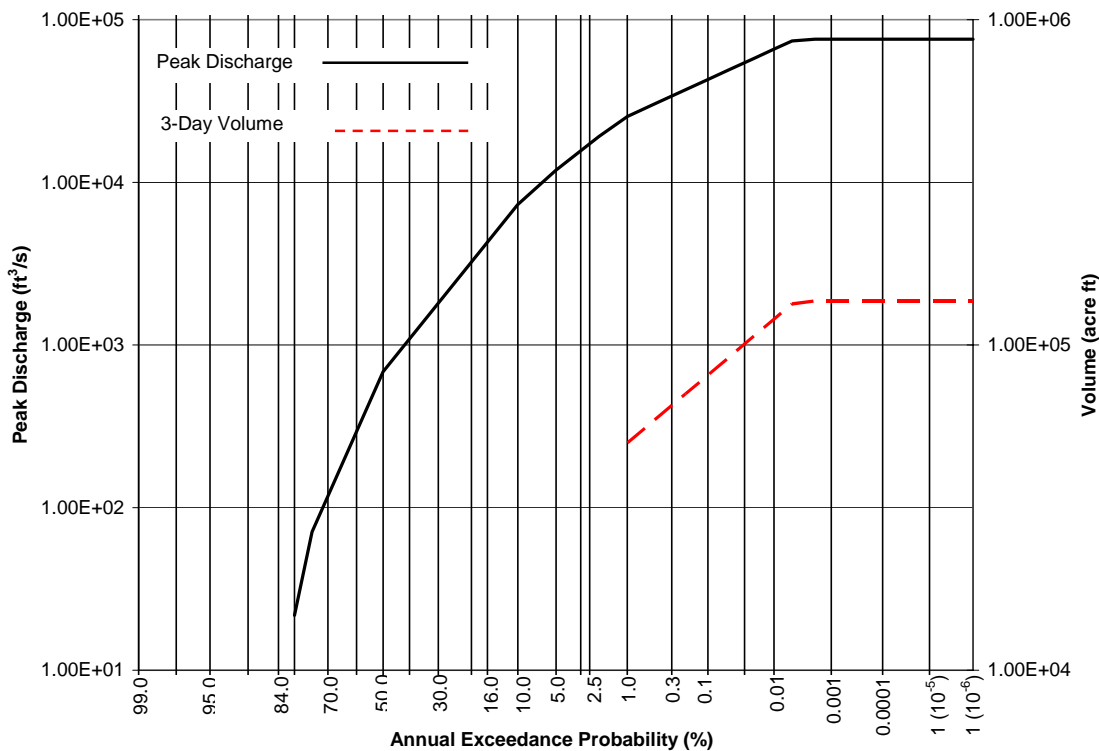


Figure 6-1.—Example of hydrologic hazard curve for Los Banos Dam, California.

Table 6-2.—Peak and volume (3-day) estimates at Los Banos Dam, California, for specified probabilities

Annual exceedance probability	Return period	Peak discharge estimate (ft³/s)	3-day volume estimate (acre-feet)
0.01	100	25,400	50,100
0.005	200	30,100	58,500
0.002	500	37,000	70,800
0.001	1,000	42,800	80,800
0.0005	2,000	49,100	91,700
0.0002	5,000	58,300	107,200
0.0001	10,000	65,900	120,000
0.00005	20,000	74,100	133,700
0.00002	50,000	75,800	136,500

on the Crooked River. The drainage basin upstream of the dam has a total area of approximately 2,635 mi². The spillway crest elevation is 3234.8 feet, and the dam crest elevation is 3264.0 feet. The spillway capacity at the dam crest elevation is 11,500 ft³/s. The total capacity

of the dam at water surface elevation 3257.9 feet is 233,100 acre-feet. The current PMFs for A.R. Bowman Dam were computed in 1994. The results of these studies are summarized in table 6-3.

Table 6-3.—Probable maximum flood summary A.R. Bowman Dam, Oregon

Flood type	HMR	Peak inflow (ft ³ /s)	Volume (acre-feet)	
			With 100-year antecedent conditions	Without 100-year antecedent conditions
February General storm	57	255,000	770,000 (15 day)	138,000 (4.3 days)
June General storm	57	83,300	185,600 (15 day)	185,600 (15 day)
August General storm	57	75,000	—	112,600 (15 day)

Routings of the PMF hydrographs indicated that the February general storm event, with a starting water surface elevation of 3211.17 feet, would overtop the dam by 18.6 feet. A.R. Bowman Dam would only pass approximately 10 to 25 percent of the February general storm PMF without overtopping. The June and August PMFs did not overtop the dam.

Because A.R. Bowman Dam is overtopped by the February general storm PMF, flood probabilities are needed to understand and quantify the risk of overtopping and hydrologic hazard for dam safety. Several investigations have been undertaken since about 1991 to determine PMF design modification alternatives, including overtopping protection and parapet walls. In addition, detailed probabilistic flood studies using alternative approaches (stochastic event flood modeling and paleoflood studies) were completed at A.R. Bowman Dam to better estimate the flood risk.

Three methods for estimating the hydrologic hazard are discussed in this section: an example using the proposed procedures for the initial characterization, results from a SEFM, and peak-flow frequency with detailed paleoflood data. Flood risk results from the three methods and implications for potential modifications at A.R. Bowman Dam are then discussed.

6.2.1 Example of A.R. Bowman Hydrologic Hazard Curves Using Proposed Procedures

Peak flow data at A.R. Bowman Dam were obtained from three USGS gaging stations:

- Crooked River at Post, Oregon (USGS station No. 14079500)
- Crooked River above Prineville Reservoir near Post, Oregon (USGS station No. 14079800)
- Crooked River near Prineville, Oregon (USGS station No. 14078050)

These gages are the closest to A.R. Bowman Dam; the first two are located upstream from the reservoir, and the gage near Prineville is located downstream from the dam. The combined

gages provide 38 years (1909 to 1972, with many missing years) of unregulated peak discharge records. The peak discharge data for the upstream gages were adjusted by the square root of the drainage area ratio (Cudworth, 1989). The December 1964 flood was treated as a historic peak with a return period of approximately 140 years (largest since about 1860).

The reconnaissance-level paleoflood data that may be applied to estimate flood frequency at A.R. Bowman Dam are based on the detailed paleoflood data presented below. Information from the paleohydrologic bound with the largest discharge was used and was dated based on the Mount Mazama volcanic eruption 7,600 years ago. Based on this information, it appears that a peak discharge between 27,000 and 36,000 ft³/s on the Crooked River has not been exceeded in the last 7,600 to 10,000 years.

A hydrologic hazard curve for A.R. Bowman Dam was constructed using the techniques outlined in section 5. This example calculation was completed after the results of the SEFM and paleoflood study were completed. This example represents the first step that might be performed in future flood studies for Reclamation dam safety. Peak flow and paleoflood data and a general storm PMF hydrograph were used to estimate the hazard curve (figure 6-2 and table 6-4).

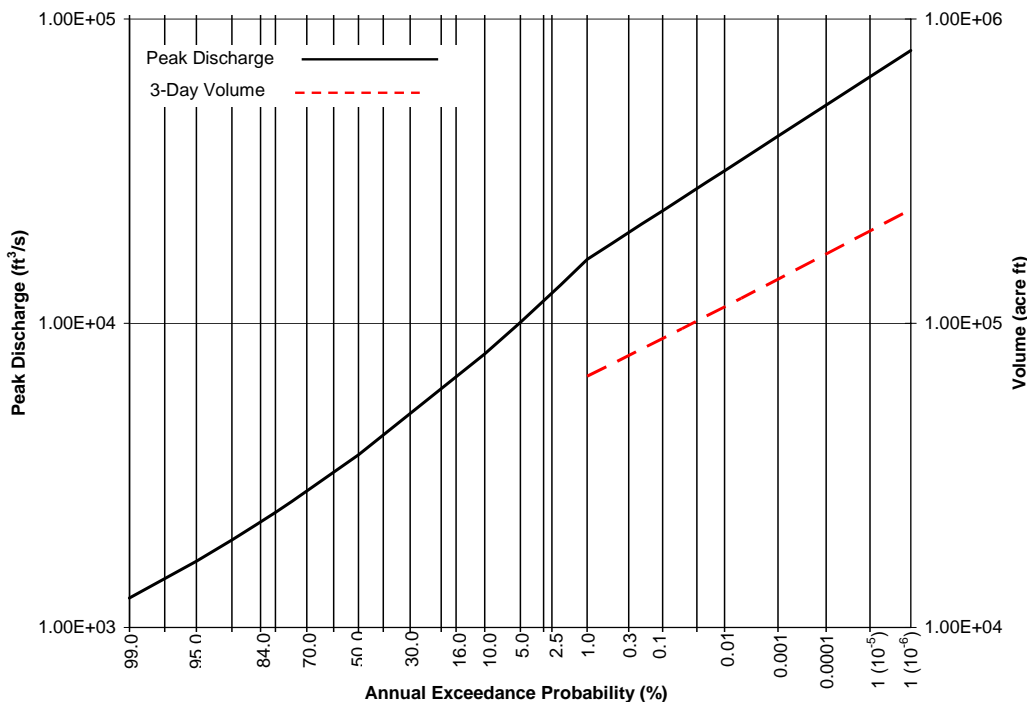


Figure 6-2.—Example of a hydrologic hazard curve for A.R. Bowman Dam, Oregon.

6.2.2 A.R. Bowman Hydrologic Hazard Estimates Based on a Stochastic Event Flood Model

MGS Engineering Consultants, Inc., was contracted in November 1997 (Schaefer and Barker, 1997) to perform a hydrologic stochastic analysis to develop magnitude frequency curves for flood peak discharge, runoff volume, and maximum reservoir elevation. The SEFM was developed using a deterministic rainfall-runoff model (HEC-1) and treated input parameters

Table 6-4.—Peak and volume (3-day) estimates at
A.R. Bowman Dam, Oregon, for specified probabilities

Annual exceedance probability	Return period	Peak discharge estimate (ft ³ /s)	3-day volume estimate (acre-feet)
0.01	100	16,000	65,100
0.005	200	18,100	71,300
0.002	500	21,000	79,700
0.001	1,000	23,300	86,300
0.0005	2,000	25,700	93,000
0.0002	5,000	29,100	102,100
0.0001	10,000	31,700	109,200
0.00005	20,000	34,500	116,600
0.00002	50,000	38,400	126,600
0.00001	100,000	41,500	134,500
0.000001	1,000,000	52,700	162,400
1E-07	10,000,000	65,500	193,200
1E-08	100,000,000	80,300	227,300

that were selected by Monte Carlo sampling procedures as a probabilistic distribution of values rather than fixed values. The SEFM is described in section 4.5. After the initial modeling runs and results at A.R. Bowman Dam were made, the precipitation depth-area relationships were refined (Schaefer and Barker, 1998) and new flood frequency relationships were developed.

The results from the SEFM include peak flow, volume, and reservoir elevation frequency curves. These graphs are shown below and include polynomial equations fitted to model output that are applicable in the range of AEP from 10^{-2} to 10^{-5} (Schaefer and Barker, 1998). From these results, the dam may be potentially overtopped with a maximum reservoir elevation probability equal to 0.0005 (2,000-year return period) (figure 6-4).

Peak discharge estimate:

$$Q_p = 2322 \text{Log}^2(\text{AEP}) - 2241 \text{Log}(\text{AEP}) + 950$$

Flood Runoff Volume Estimate:

$$Q_v = 8385 \text{Log}^2(\text{AEP}) - 59700 \text{Log}(\text{AEP}) - 39490$$

Maximum reservoir elevation estimate:

$$ELEV_{\max} = -0.3993 \text{Log}^4(\text{AEP}) - 6.193 \text{Log}^3(\text{AEP}) - 33.67 \text{Log}^2(\text{AEP}) - 89.81 \text{Log}(\text{AEP}) + 3159.2$$

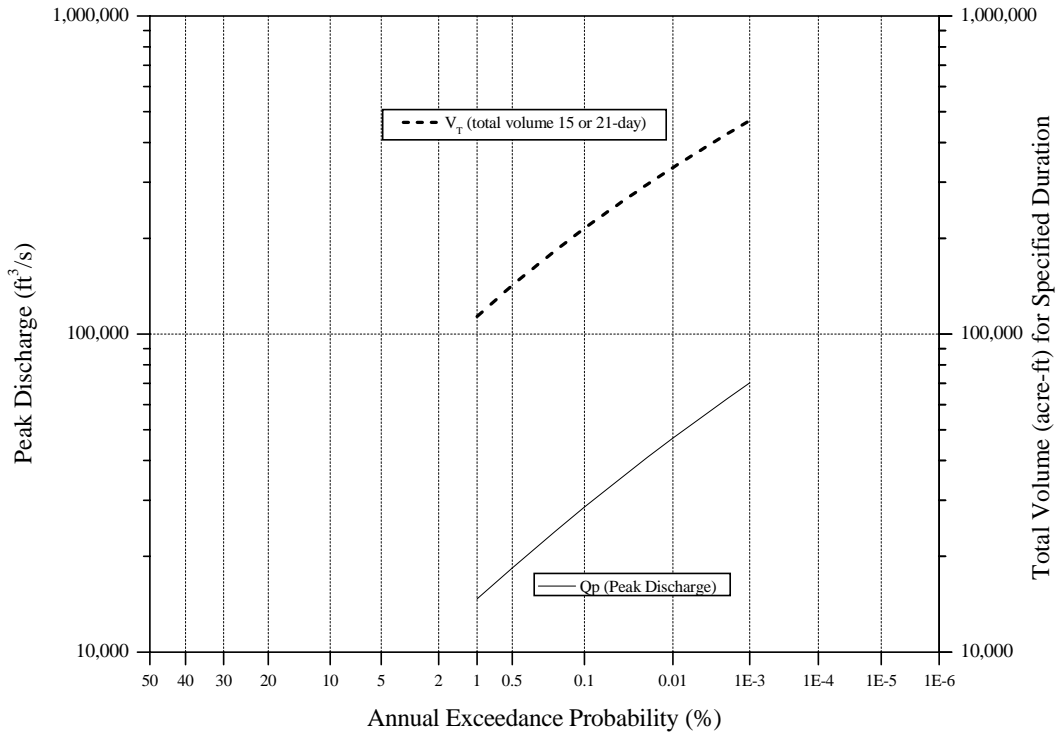


Figure 6-3.—Peak discharge and total volume frequency curves from the SEFM for A.R. Bowman Dam, Oregon.

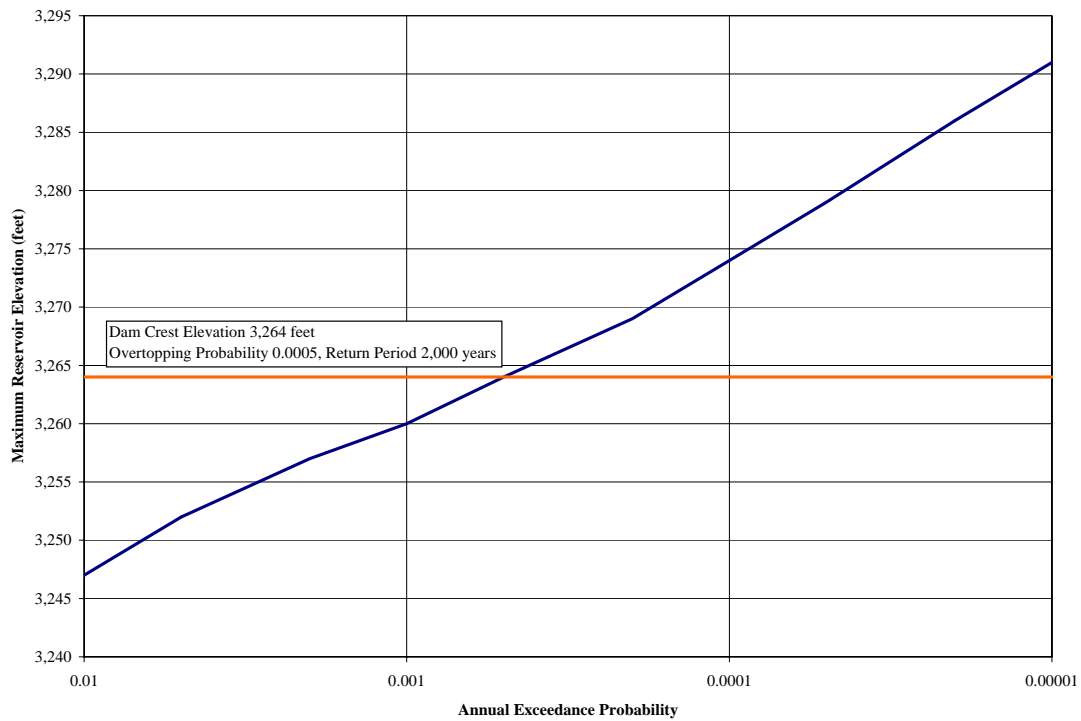


Figure 6-4.—Maximum reservoir elevation frequency curve from the SEFM for A.R. Bowman Dam, Oregon.

6.2.3 A.R. Bowman Hydrologic Hazard Estimates Using Bayesian Statistical Estimation

To gain information on the magnitude of low probability floods for risk analysis, the Dam Safety Office, in 1995, requested a paleoflood study on the Crooked River at A.R. Bowman Dam. The paleoflood study was an alternative approach to the SEFM. The paleoflood report has not been published because of changing priorities in the Dam Safety Office.

For the Crooked River paleoflood study, two reaches downstream from A.R. Bowman Dam were selected for detailed hydraulic modeling. These reaches are about 2 and 35 miles downstream from the dam. At these study reaches, there is geomorphic, stratigraphic, and botanic evidence to limit the paleostage of floods throughout the Holocene epoch (the past 10,000 years). In one of the downstream reaches, it is possible to reconstruct the magnitude of the December 1861 flood based on the presence of driftwood piles. In addition to these downstream study reaches, a third study reach was identified just upstream from Prineville Reservoir, where the stage of paleofloods during the late Holocene epoch can be related to the peak discharge estimates from the December 1964 flood.

Stratigraphy and soils were described at eight sites along the Crooked River, including the three study reaches. At these sites, the soils were described in detail and material was collected for radiocarbon dating. In total, there are 54 radiocarbon ages for the Crooked River paleoflood study. In addition to these radiocarbon ages, the Mazama ash forms an important stratigraphic datum along the river downstream from A.R. Bowman Dam. The calibrated ages of radiocarbon samples associated with the Mazama ash throughout the western U.S. yield an age of about 7,650 years. The hydraulic geometry of the Crooked River channel is remarkably consistent at all the study sites. This implies long-term stability of the Crooked River channel and of the mechanisms that produce large floods on the Crooked River.

The paleoflood data that have been collected and analyzed to date are summarized in table 6-5.

Table 6-5.—Detailed paleoflood estimates at A.R. Bowman Dam, Oregon

Type	Preferred age (years before present)	Age range (years)	Preferred discharge (ft ³ /s)	Discharge range	
				Low (ft ³ /s)	High (ft ³ /s)
1862 historical bound	140	—	15,000	13,000	18,000
Paleohydrologic bound	1,100	980 – 1,210	20,700	18,000	25,000
Paleohydrologic bound	3,150	2,650 – 3,650	25,000	22,000	30,000
Paleohydrologic bound	3,500	3,350 – 5,050	27,000	25,000	32,500
Paleohydrologic bound	9,000	7,600 – 10,000	30,000	27,000	36,000
Historical flood	140	—	20,700	18,000	25,000
Paleoflood	1,100	980 – 1,210	20,700	18,000	25,000

The paleoflood data were combined with peak flows from three gaging stations. A frequency analysis was performed using FLDFRQ3 (O’Connell, 1999; O’Connell et al., 2002). The peak-flow frequency results for a particular distribution (log-Pearson) are shown in figure 6-5 and summarized in table 6-6. Based on this peak-discharge frequency curve and scaling model hydrographs, the dam might be overtopped at exceedance probabilities greater than 0.0001 (more frequently than the 10,000-year return period).

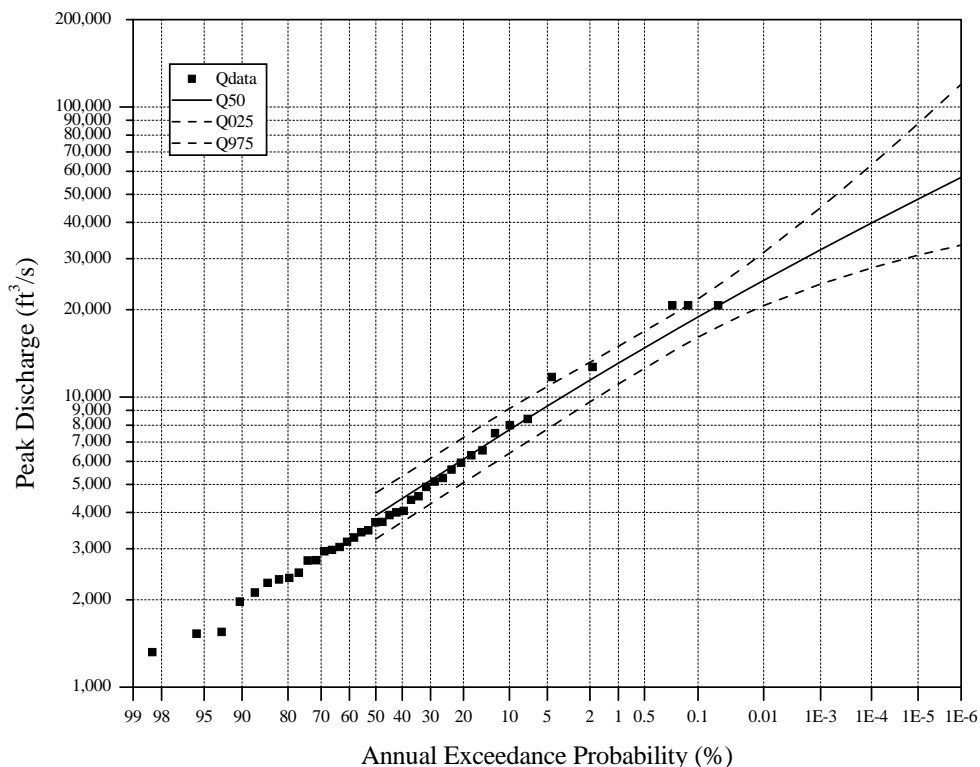


Figure 6-5.—Peak-discharge frequency curve based on paleoflood data for A.R. Bowman Dam, Oregon.

Table 6-6.—Peak discharge estimates based on paleoflood data at A.R. Bowman Dam, Oregon, for specified probabilities

Annual Exceedance Probability	Return Period	Median (50%) LP3 Model Peak Discharge Estimate (ft ³ /s)	2.5% LP3 Model Peak Discharge Estimate (ft ³ /s)	97.5% LP3 Model Peak Discharge Estimate (ft ³ /s)
0.01	100	13,100	11,100	15,000
0.002	500	17,100	14,600	19,500
0.001	1,000	18,900	16,100	21,800
0.0005	2,000	20,700	17,500	24,400
0.0002	5,000	23,300	19,300	28,200
0.0001	10,000	25,200	20,700	31,500
0.00001	100,000	32,200	24,500	44,900
0.000001	1,000,000	39,800	27,900	63,200
0.0000001	10,000,000	48,200	30,800	87,500
0.00000001	100,000,000	57,100	33,400	119,700

6.2.4 Combined Hydrologic Hazard Estimates for Risk Analysis and Dam Safety Implications

Both the SEFM and paleoflood study indicate that A.R. Bowman Dam potentially might not meet Reclamation hydrologic hazard criteria for overtopping because it does not pass the 10,000-year flood (at a minimum). The results from the three approaches (initial characterization, the SEFM, and paleoflood study) were combined to provide a single estimate of the hydrologic hazard curve for risk analysis. The weighting schemes are simple numerical weights given to each model result. Three weighting schemes were analyzed for A.R. Bowman: (1) enveloping all results (figure 6-6), (2) weighting the SEFM and paleoflood peak flow curves equally (50 percent each) (figure 6-7), and (3) choosing a “best estimate” between the SEFM and paleoflood peak flow curves equal to the upper 97.5 percent confidence limit from the Bayesian analysis (figure 6-8). The third option was selected to represent the peak discharge flood frequency based on results and decisions made in 1999 for a risk analysis. A “combined” hydrologic hazard curve is shown in figure 6-9, which also includes a volume frequency analysis based on results of the SEFM.

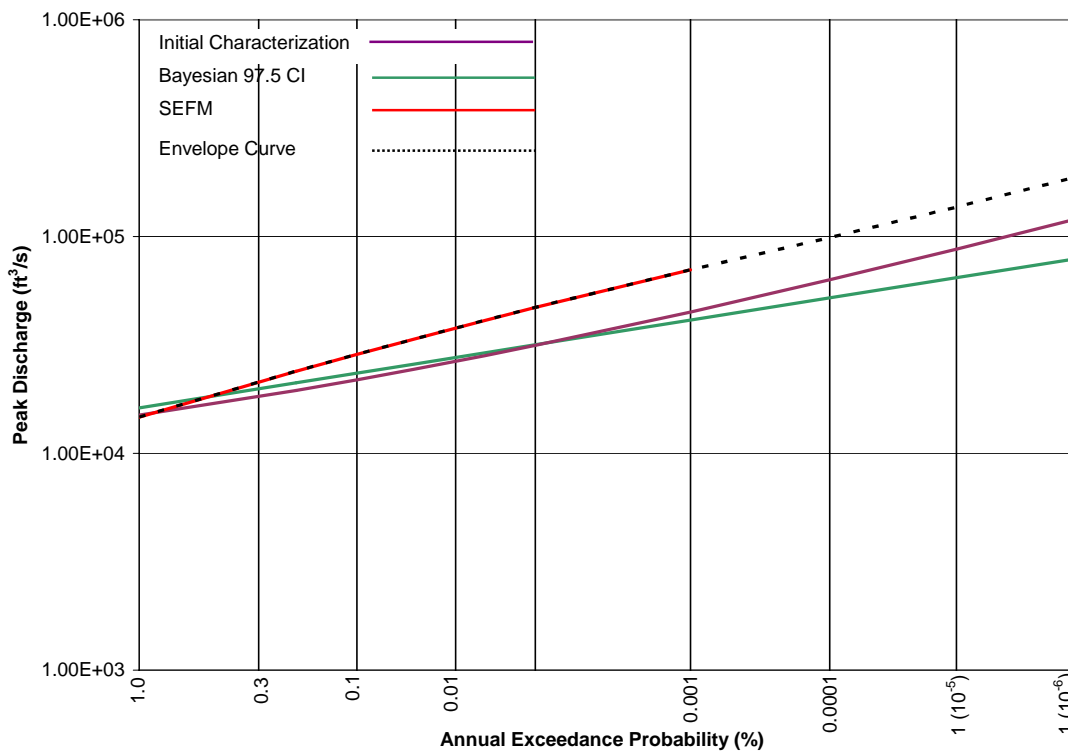


Figure 6-6.—Weighted peak-discharge frequency curve based on enveloping results for A.R. Bowman Dam, Oregon.

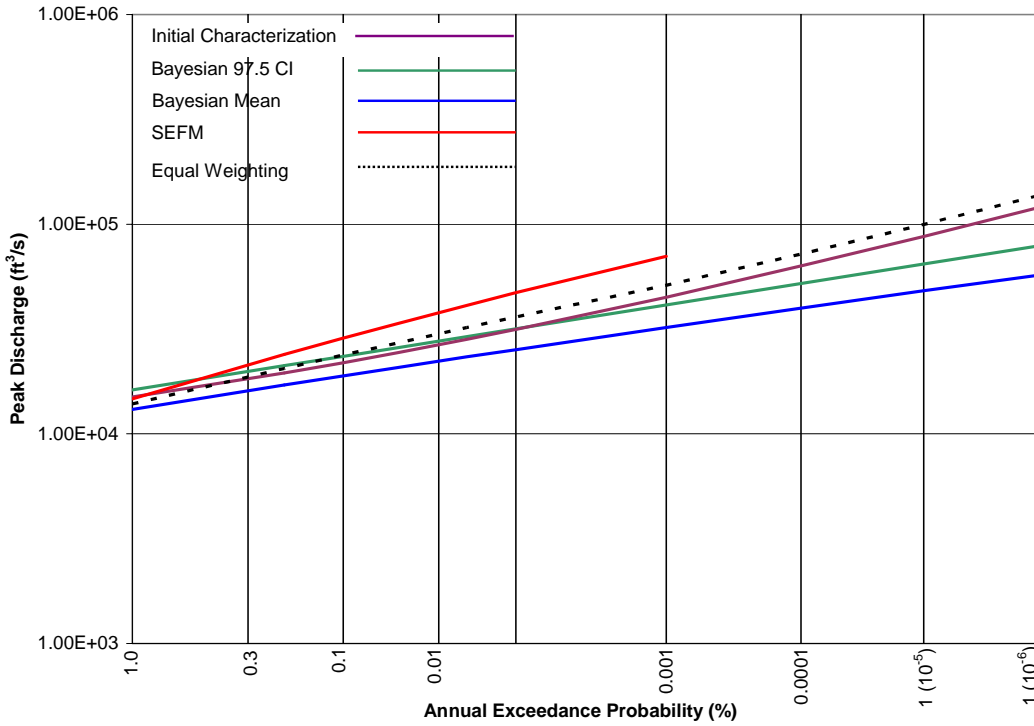


Figure 6-7.—Equal (50 percent) weighted peak-discharge frequency curve for A.R. Bowman Dam, Oregon.

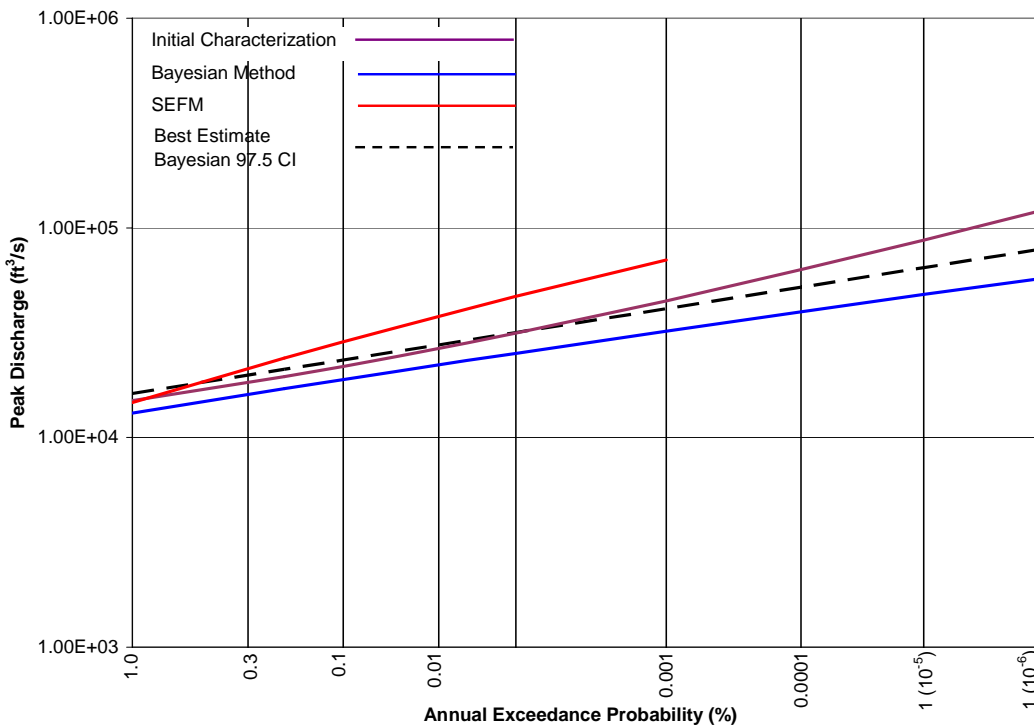


Figure 6-8.—Best estimate (Bayesian 97.5 percent) weighted peak-discharge frequency curve for A.R. Bowman Dam, Oregon.

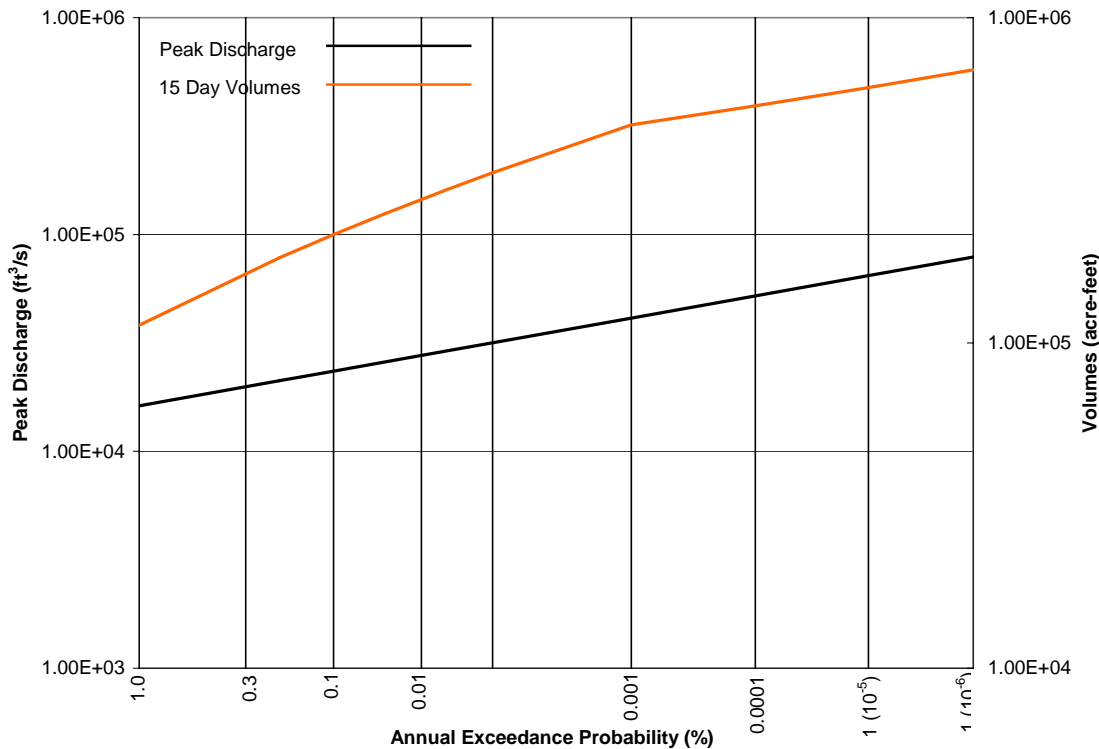


Figure 6-9.—Final combined hydrologic hazard curve for A.R. Bowman Dam, Oregon.

Because A.R. Bowman Dam might not meet Reclamation criteria, additional engineering studies are ongoing to estimate the need for any potential structural modifications at this site. Probability-based design hydrographs for any potential modification or design alternatives were selected based on a combination of the weighting results shown above. Corrective action studies are ongoing.

6.3 Fresno Dam

Fresno Dam is located on the Milk River in north-central Montana, approximately 14 miles west of the town of Havre (figure 6-10). The compacted earthfill dam was constructed from 1937 to 1939. It is approximately 110 feet high. Previous studies have indicated that the dam could safely pass Reclamation’s 1985 PMF hydrograph. Normally, Reclamation would not have conducted additional hydrologic studies for this dam; however, project beneficiaries wanted to examine the possibility of enlarging the dam.

6.3.1 Example of Hydrologic Hazard Curves Using Proposed Procedures

A hydrologic hazard curve for Fresno Dam was constructed using the techniques outlined in section 0.1. Peak flow and paleoflood data and a general storm PMF hydrograph were used to estimate the hazard curve (figure 6-11 and table 6-7).

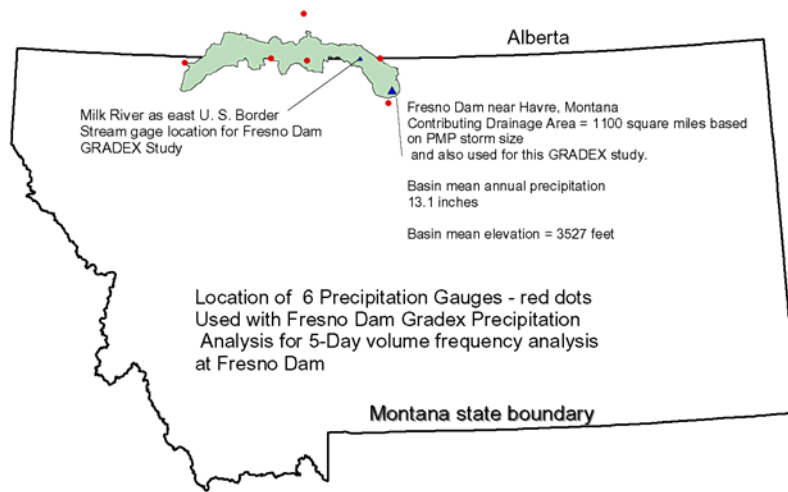


Figure 6-10.—Site and watershed depiction of Fresno Dam, Montana.

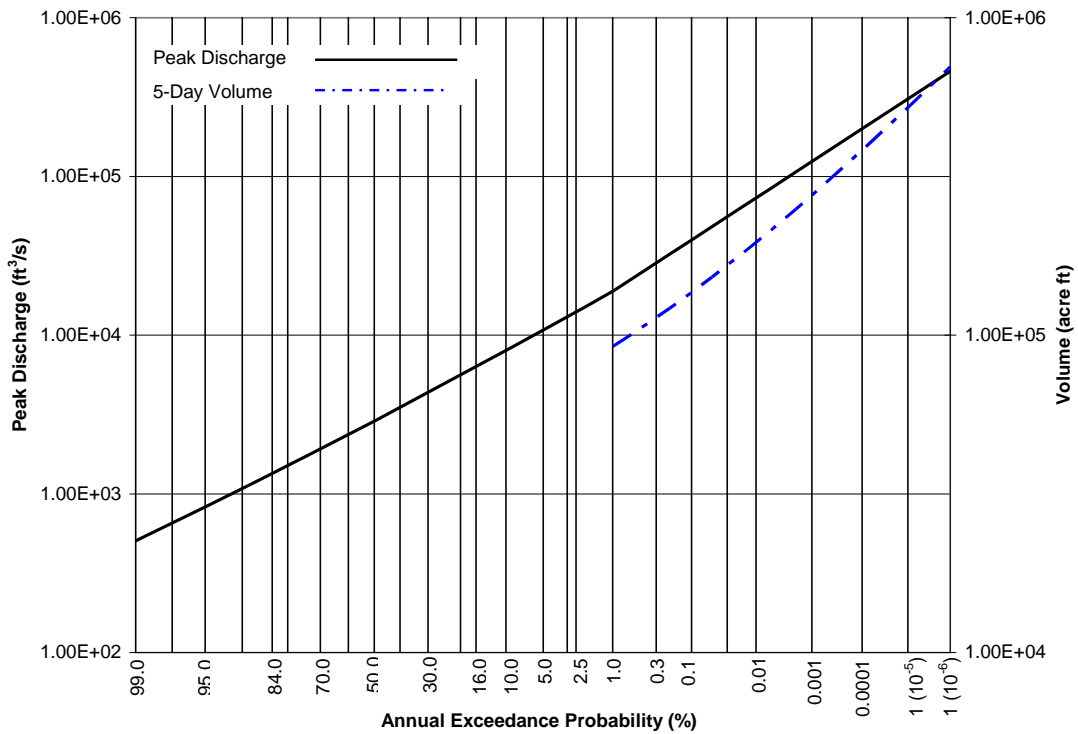


Figure 6-11.—Example of hydrologic hazard curve for Fresno Dam, Montana.

Table 6-7.—Peak and volume (5-day) estimates at Fresno Dam, Montana, for specified probabilities

Annual Exceedance Probability	Return Period	Discharge Estimate	5-Day Volume Estimate (acre ft)
0.01	100	18,900	92,000
0.005	200	24,100	103,900
0.002	500	32,300	121,400
0.001	1000	39,700	136,100
0.0005	2000	48,200	152,300
0.0002	5000	61,500	176,000
0.0001	10000	73,100	195,900
0.00005	20000	86,400	217,600
0.00002	50000	106,600	249,500
0.00001	100000	124,200	276,000
0.000001	1000000	199,600	382,100
0.0000001	10000000	307,800	520,800
0.00000001	100000000	459,400	700,300

Peak flow data at Fresno Dam were obtained from the USGS gaging station, Milk River at Eastern U.S. Border Crossing (USGS station No. 06135000). The gage provides peak flow records from 1910 through 2002. No adjustments were performed on these data, as the associated drainage area and streamflow records were believed to accurately represent the conditions at Fresno Dam.

The reconnaissance-level paleoflood data that may be applied to estimate flood frequency at Fresno Dam are based on detailed chronology developed at archaeological sites on the Milk River. Based on this information, it appears that a peak discharge between 40,000 and 70,000 ft³/s on the Milk River has not been exceeded in the last 2,000 to 4,000 years.

Fresno Dam has a spillway discharge capacity of 51,360 ft³/s at the maximum water surface elevation of 2591 feet. Comparing this value with the peak-discharge flood frequency curve indicates that the spillway is capable of passing a flood with a return period of only slightly longer than 2,000 years. At the time of this study, the methods for determining the volume frequency relationship had not been developed, so hydrographs were not available for reservoir routing. Therefore, we decided to develop hydrographs using the GRADEX Method.

6.3.2 Example of Hydrologic Hazard Analysis Using the GRADEX Method

Daily flow records that approximate the daily inflows to Fresno Dam come from the stream gage on the Milk River at the Eastern U.S. Border Crossing (USGS station No. 06135000). This gage record has daily flow data from August 1909 through September 2001. A total of 86 years of daily flow data is available, with a few missing years. The contributing drainage area at the gage site is considered to be 2,525 mi², and the gage is located about 54 river miles upstream from Fresno Dam. The total possible contributing drainage area at Fresno Dam is considered to be 2,911 mi² for the rainfall-runoff modeling calculations in this study. These daily flow records were checked with available reservoir inflow records for concurrent years.

The information needed from the stream gage record is a set of independent flood volumes, representing the volume of flooding caused by rainfall for independent storms in the basin. The process starts by setting a threshold flow value and then counting the number of events that have continuous days of flow above that value. A threshold flow value was set and adjusted until about 86 independent flood events above this value were determined. The average duration of all of the independent flood events was then calculated. This average duration of independent flood events for the Milk River above Fresno Dam was found to be between 4 and 5 days, with a threshold value of 1,307 ft³/s. The value of 5 days was selected for further use in the study as the critical duration for both rainfall and flood events.

The GRADEX Method required that the number of multi-day rain events at each gage be equal to the number of common years of record for the selected gages. The number of multi-day rain events was made equal by selecting a threshold 5-day total rainfall for each gage such that exactly fifty 5-day rain totals were above the threshold value for each gage. Figure 6-10 displays the locations of the six precipitation gage stations selected for the GRADEX analysis at Fresno Dam. Table 6-8 lists the six stations used for the remainder of this analysis along with some pertinent information including the MAP.

Table 6-8.—Rainfall data description

Selected Rainfall Stations with a Period of Record Including 1950-2000 Used with Fresno Dam GRADEX Study for high return period volumes Fresno basin mean basin elevation = 3527 feet Fresno basin mean annual precip. = 13.1 inches					
Station	State/Prov.	Latitude	Longitude	Elevation (feet)	M.A.P. (inches)
Babb	MT	48.93	113.36	4300	18.27
Gold Butte	MT	48.98	111.4	3498	13.38
Kremlin	MT	48.52	110.1	2860	11.56
Simpson	MT	49	110.22	2815	10.24
Sweetgrass	MT	49	111.97	3466	13.98
Foremost	ALB	49.48	111.45	2899	14.68

Using the 5-day precipitation totals, 5-day flow volumes, drainage area, and other data described above, the GRADEX Method computations were made as outlined in Naghettini (1994) and Naghettini et al. (1996). For this study, a special situation arises with respect to selection of the contributing drainage area above Fresno Dam.

Based on all previous Reclamation IDF and PMF studies for Fresno Dam, the authors of this study believe that a single-storm precipitation pattern will not cover the entire basin. For this GRADEX study, a total contributing drainage area of 1,100 mi² was selected, based primarily on the use of the hypothetical elliptical patterns used with the PMP studies.

Table 6-9 and figure 6-12 display the summary results from the calculations for Fresno Dam. The volume of the PMF volume centered hydrograph from the 1985 Reclamation PMF study is plotted as a straight line on the frequency curve for comparison. It should be noted that the PMF is based on a 3-day rainfall, and the Fresno Dam frequency curve is based on 5-day total rainfall amounts. The GRADEX Method is intended to give flood volume data for very large return periods, in excess of 200 years. More common methods, such as a LP-III distribution fit to available streamflow data, should be used for the lower return periods, 100 years and less in this study.

Table 6-9.—Five-day volume frequency curve for Fresno Dam, Montana, based on GRADEX Method

Return Period (years)	Five-day Volume (acre-ft)	Upper Bound (acre-ft)	Lower Bound (acre-ft)
500	188,700	320,100	112,400
1,000	214,700	358,700	131,100
5,000	275,100	448,200	174,700
10,000	301,100	486,800	193,500
20,000	327,100	525,400	212,300
50,000	361,600	576,400	237,200
100,000	387,600	615,000	256,000
500,000	448,000	704,600	299,600
1,000,000	474,100	743,100	318,500

In most normal years, snowmelt may contribute a large amount to the total volume of inflows to Fresno Reservoir. Another assumption in the GRADEX Method is that the precipitation that is measured at the gages is in liquid form and ready for runoff, or if it is snow, the snow will melt during the critical storm-time period and add to the runoff volume. A simple scheme to separate purely snowmelt runoff from rain or rain-on-snow runoff was used with the 86 years of daily flows at the Milk River at Eastern U.S. Border Crossing gage. The 7-day maximum of purely snowmelt flow were then determined for each year and expressed as a constant flow for 7 days. The 86-year-long series of 7-day maximum snowmelt flows at the gage site was then analyzed using a standard LP-III analysis. The lower curve in figure 6-13 displays the frequency curve of maximum 7-day purely snowmelt flows. The curve is quite flat. This is expected for purely snowmelt driven volumes. There is a physical limit to how much snow can be melted with normal ranges of climate variables. The snowmelt volume frequency curves will become quite flat for larger return periods. The 7-day flows were then converted to volumes in acre-feet and a similar frequency curve was also plotted in figure 6-13.

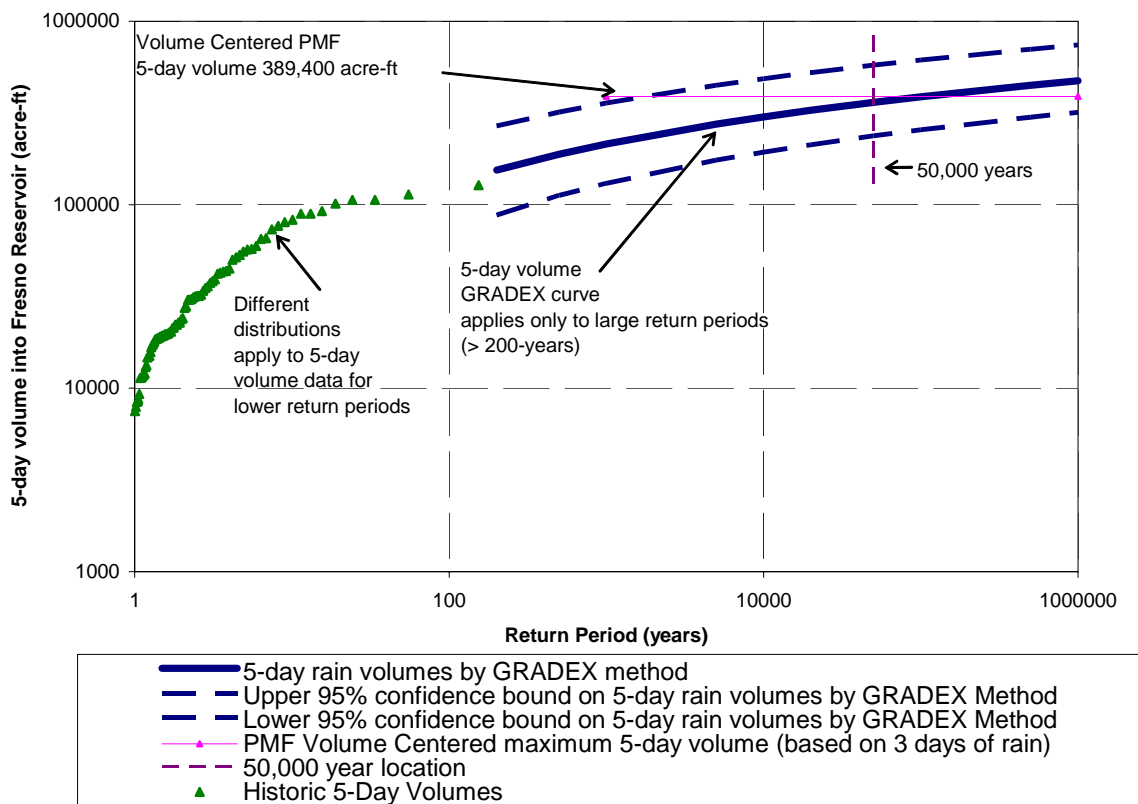


Figure 6-12.—Fresno Dam, Montana, 5-day rain-generated volumes by GRADEX Method.

The 5-day volumes of the GRADEX rain flood volumes from this study were then added to the plot. This is the blue curve near the top of figure 6-13. A combined probability curve was then computed and is shown as the red curve on the top of figure 6-13. The combined probability curve expresses the probability (or return period) of getting a particular total flood volume by combining a 7-day snowmelt with a 5-day rain flood volume. The combined probability curve very closely follows the 5-day rain flood volume curve for the large return periods.

Hourly inflow hydrographs were developed for Fresno Dam for the desired return periods by applying indexed rainfall amounts to the HEC-HMS models calibrated to the 1906 and 1964 rainfall flood events. The rainfall amounts for the 1964 and 1906 storm hyetographs were uniformly increased in each subbasin until the 5-day volume of the respective return periods, determined by the GRADEX Method, was achieved. Figure 6-14 displays the results for the 1964 flood computations. Figure 6-15 displays the results of increasing the 1964 precipitation amount to calculate hydrographs with the GRADEX volumes for specified return periods.

The peak versus volume relationship was developed using a LOWESS technique (Cleveland, 1979). The 86 years of peak and 5-day volume data from the USGS gage at the Milk River at Eastern Border were plotted. In addition, the 30 peak-flow and 5-day volumes generated from

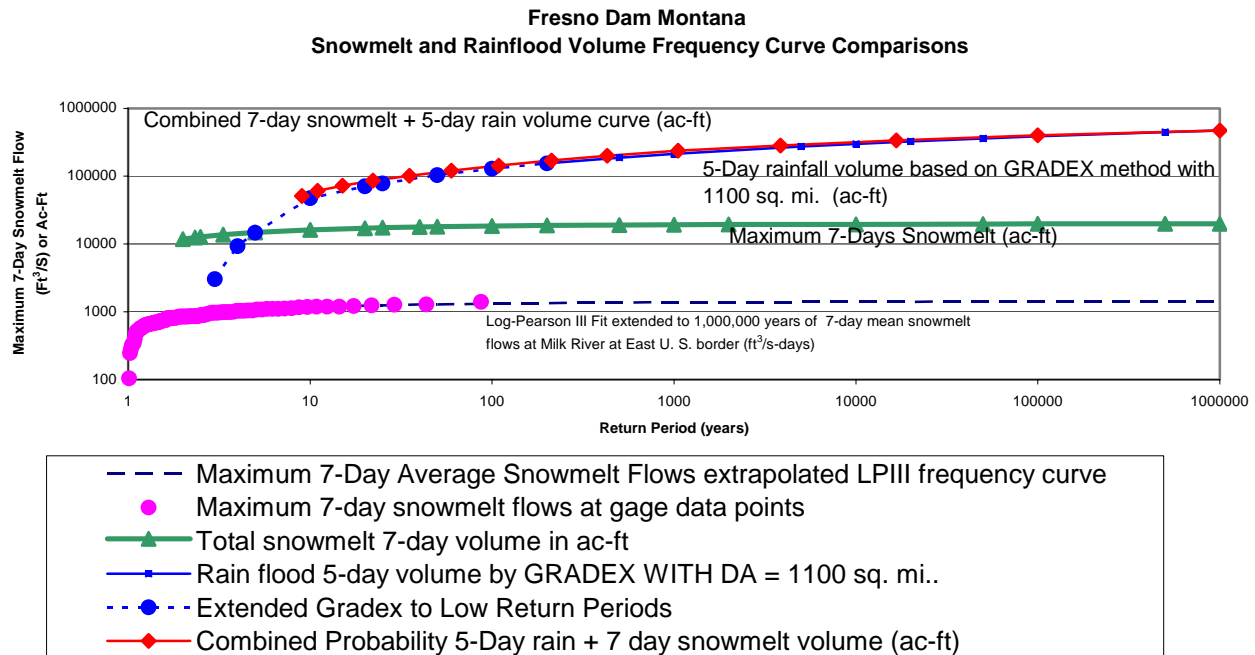


Figure 6-13.—Fresno Dam, Montana, snowmelt and rainflood volume frequency curve comparisons.

the HEC-HMS models originally calibrated to both the 1906 and 1964 flood events were run. In each run, the lag time was varied or the rainfall was increased in a systematic manner to cover a range of possible values for greater storm events. A LOWESS fit was done using the entire set of peak versus volume points and is shown in figure 6-16. This relationship was used to determine the peak discharge estimates that correspond to the selected 5-day return period volumes computed using the GRADEX Method.

The results of this calculation can be used to derive a peak-flow flood frequency curve. The information for the peak flow and volumes for the return periods of interest in this study is summarized in table 6-10. In this study, the reported paleohydrology information indicated a non-exceedance peak flood bound with an age between 2,000 and 4,000 years and a magnitude between 40,000 and 70,000 ft³/s. The paleohydrology information was in agreement with the GRADEX results.

For Fresno Dam, the results of the GRADEX Method were used for the risk assessment because flood volumes were needed to evaluate the safety of the dam. Since only peak flow data were available at the time of the first analysis, the GRADEX study was conducted. Hydrographs could then be routed through the reservoir to take advantage of the flood attenuation effects of the large surcharge storage in the reservoir.

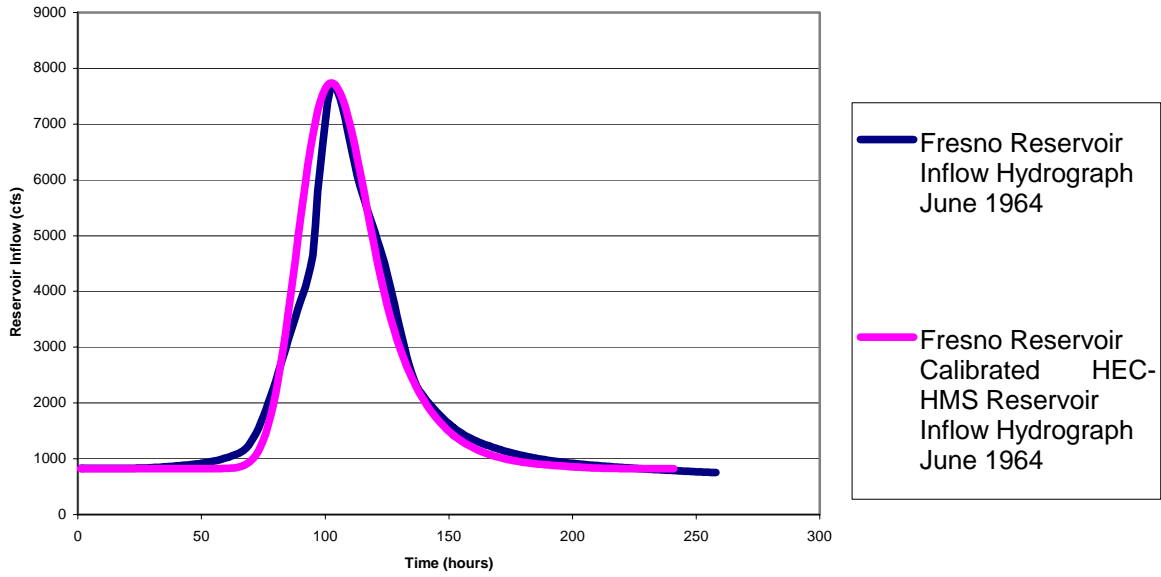


Figure 6-14.—Comparison of gage record and HEC-HMS calibration run.

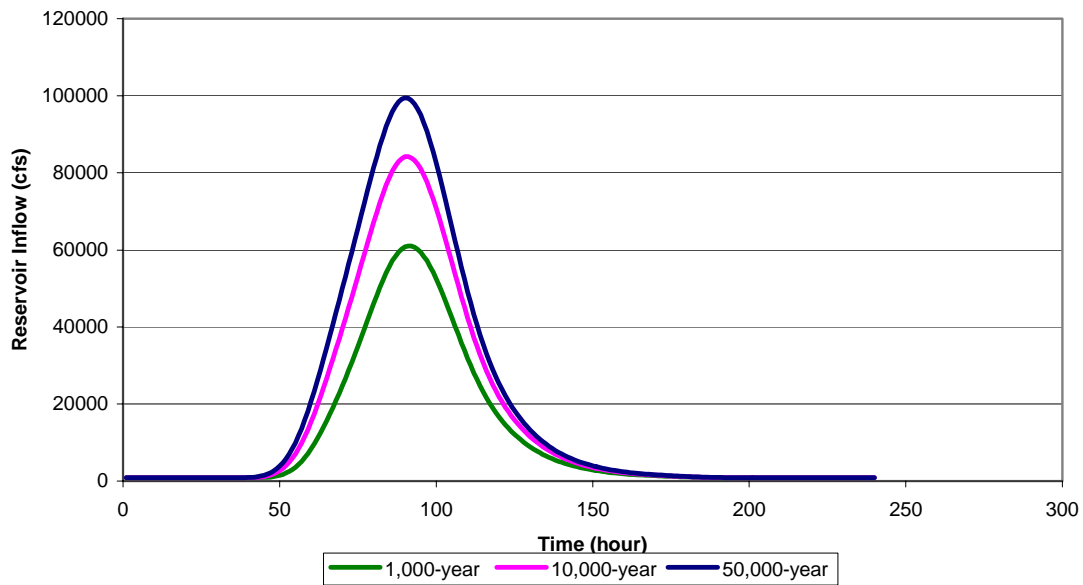


Figure 6-15.—Fresno Dam, Montana, 1,000-, 10,000-, and 50,000-year hydrographs. Hydrographs from 1964 precipitation pattern with volume from GRADEX Method.

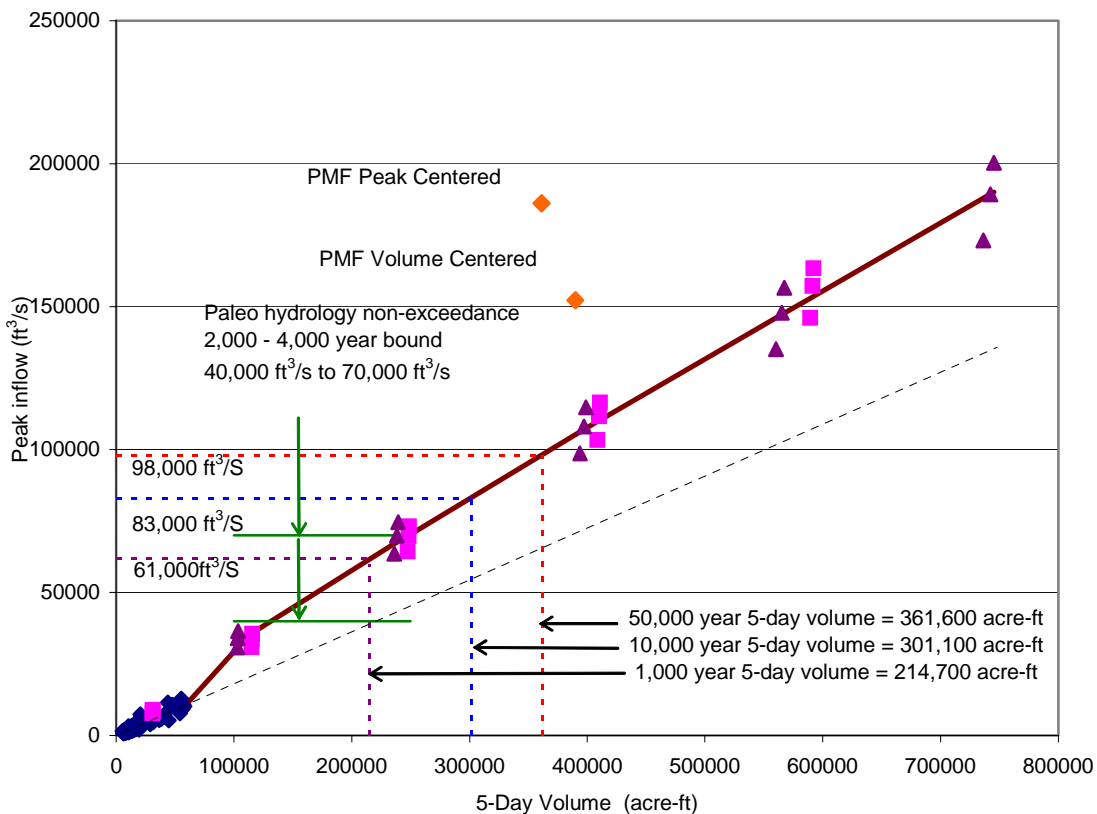


Figure 6-16.—Fresno Dam, Montana, 5-day volumes versus peaks.

Table 6-10.—Fresno Dam, Montana, GRADEX Method results

Return Period (year)	Peak Inflow (cfs)	5-Day Volume (acre-ft)
1,000	61,000	214,700
10,000	83,000	301,100
50,000	98,000	361,600

7. Summary

This report summarizes Reclamation’s Flood Hydrology Group’s approach toward developing hydrologic hazard curves for use in evaluating dam safety issues. The procedure relies on extracting information from existing studies to the fullest extent possible. The initial characterization of hydrologic risk can usually be accomplished with minimal effort. Initial

hydrologic hazard curves display peak flow and volume relationships with AEPs in the range of 0.99 to 0.00000001. The procedures and analysis techniques defined in this report allow for the possibility, and even plausibility, that peak discharge and volume estimates may exceed the PMF. This is a function of the uncertainty and inconsistency among and between analysis techniques. Therefore, in these cases, the PMF is believed to represent the upper limit to hydrologic risk.

This report recommends that the approach for developing hydrologic hazard curves consider the dam safety decision criteria, potential dam failure mode, and dam characteristics, available hydrologic data, possible analysis techniques, resources available for analysis, and tolerable level of uncertainty. Dam safety decision criteria determine the probabilistic range of floods needed to address hydrologic issues. The potential dam failure mode and dam characteristics impact the type of hydrologic information needed to assess the problem. This report further recommends that the approach chosen to answer specific hydrologic issues consider a tolerable level of uncertainty. Reducing the uncertainty in the estimates may require additional data collection and use of more sophisticated solution techniques.

Reclamation currently uses a combination of seven hydrologic methods to develop hydrologic hazard curves. These general techniques include: flood frequency analysis with historical and paleoflood data, hydrograph scaling and volumes, the GRADEX Method, the Australian Rainfall-Runoff Method, stochastic event-based precipitation runoff modeling with the SEFM, stochastic rainfall-runoff modeling with CASC2D, and the PMF. Each method is described within this report. It is believed that increasing the level of effort and the sophistication of analysis techniques will increase the level of confidence associated with the results.

The amount of effort expended on analyzing a hydrologic hazard depends on the nature of the problem and the potential cost of the solution. Reclamation suggests a staged approach toward evaluating a hydrologic safety issue. Initially, very little effort is expended to determine the magnitude of the hydrologic hazard. Reclamation attempts to make use of all available studies for the site of interest in the initial characterization. Often, the PMF study is the only hydrologic study available before the start of a probabilistic investigation. When other hydrologic studies have been performed, available data will be used to decrease uncertainty in results as well as to provide an overall assessment of hydrologic risk.

Dam safety evaluations usually begin with an initial characterization of hydrologic risk. If detailed studies have been conducted for the site of interest, they are summarized and presented to the risk assessment team. About two-thirds of Reclamation's dams can safely accommodate the PMF; when the PMF is selected as the IDF, no additional work may be required unless other hydraulic issues need evaluation. Additional hydrologic work begins with a flood frequency analysis developed for peak flows and volumes. It is believed that this type of information is sufficient to address hydrologic issues and make dam safety decisions at about 80 percent of the remaining dams. For the sites that still have potential safety problems, scopes of work can be developed for studies to address the specific hydrologic issues. These studies require more work and more sophisticated solution techniques than the initial flood frequency analysis.

When planning more detailed studies, it is recommended that the goal be to achieve a balance between the amount of hydrologic analysis needed to address the issues and the level of effort

required to conduct the study. As the studies get more detailed, the results should become more precise and contain less uncertainty. An example is a hydrologic study that proceeds from the initial characterization to an analysis using the GRADEX Method, the Australian Rainfall-Runoff Method, a stochastic rainfall-runoff model, or any other statistical estimation technique available.

When multiple methods are used, alternative hazard curves are developed by weighting results from the individual analyses. A team of hydrologists evaluates the alternatives and selects the one most representative of the site for use in the risk assessment. Selection of the final hydrologic hazard curve depends on the experience of the hydrologists and the assumptions that went into each analysis.

Three case studies, Los Banos, Fresno, and A.R. Bowman Dams, have been presented to illustrate the variety of methods available. These sites were chosen to demonstrate the use of the initial characterization of the flood hazard and more detailed followup studies, where available. The A.R. Bowman example shows how multiple studies were combined into a single-flood hazard curve for use in risk assessment.

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MISSION STATEMENTS

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to Indian tribes and our commitments to island communities.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.