Technical Note 1

Determination of the Inflow Design Flood for High Hazard Dams in Montana

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1 OVERVIEW

Technical Note 1 (TN1) provides guidance to engineers and professionals engaged in the analysis and design of spillways for high hazard dams in Montana. TN1 describes hydrologic modeling methods to determine the **minimum** required Inflow Design Flood (IDF) for high hazard dams in Montana, which are regulated by the Montana Dam Safety Program (MT DSP), but the guidance can also be applied to dams not regulated by the MT DSP.

TN1 provides an explanation of the Montana Dam Safety rules with respect to determining the minimum required IDF flood frequency for high hazard dams and is organized to lead the user through a logical sequence of steps when computing an IDF. The user is led through a discussion of analysis methods, tools, and references to assist the user in estimating the various elements of a hydrologic model, such as basin characteristics, routing techniques, precipitation estimates, and routing the IDF through a spillway. TN1 also describes methods to verify the IDF is reasonable and provides a level of protection commensurate with the estimated consequences that would occur should the dam fail. The appendices of TN1 provide information such as design standards and flowcharts to assist the user in analyzing a spillway, developing model input factors, and provides an example of a spillway capacity analysis.

An objective of TN1 is to provide guidance that will lead to a best estimate of the IDF, but the best estimate must be balanced against the uncertainty inherent in hydrologic modeling. To mitigate uncertainty and to ensure risk to the downstream community is commensurate with the statutory requirement, a factor of safety must be incorporated into the design. Uncertainty mitigation can be incorporated into the analysis process by selecting conservative hydrologic modeling parameters that result in a greater IDF peak runoff rate and volume, but this “compounding conservativism” can result in an IDF that is significantly larger than necessary, thus is not recommended.

TN1 is not a regulatory requirement on how hydrologic analyses must be performed - other methods that are appropriately documented and justified may be used, but the guidance and methods described in TN1 are accepted by the MT DSP. We welcome comments on TN1, and contact information is provided in Section 2. We hope you find TN1 helpful and that it describes reasonable methods to be used when computing IDFs for high hazard dams in Montana.

2 INTRODUCTION

The **Montana Dam Safety Program (MT DSP)** is pleased to provide this **Technical Note 1 (TN1), Determination of the Inflow Design Flood for High Hazard Dams in Montana**. We hope this publication is helpful in providing technical guidance to professionals engaged in the analysis and design of spillways for high hazard dams in Montana. Our intent is to make available relevant and up-to-date information, references and procedures, much of which is unique to Montana, for use in computing IDFs that are commensurate to the estimated loss of life in the event of a dam failure.

This is the first revision to Technical Note 1. It is a goal of the MT DSP to offer the best guidance we possibly can, and we welcome and encourage your feedback on the contents of TN1. TN1 will continue to be revised as updated as new information and analytical methods become available. Please send your comments to:

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2.1 Purpose

The purpose of TN1 is to provide guidance on hydrologic methods and procedures to determine the minimum required Inflow Design Flood (IDF) for high hazard dams in Montana. A principal objective of TN1 is to provide a framework for consistency in computing IDFs. TN1 is not a regulatory document, and the references and procedures provided can be modified to suit the needs of each individual project; however, regardless of the methods and procedures employed, the basis for determining the IDF must be fully documented and justified.

2.2 Target Audience

TN1 is intended for use by professionals experienced in hydrologic analyses. Regardless of the guidance provided in TN1, hydrologic analyses require professional judgment, and it is a requirement that users of TN1 have experience and expertise in computing IDFs. DNRC and the Dam Safety Program are not responsible for the use or interpretation of TN1 contents.

3 DESIGN STANDARDS AND PROCEDURES

Montana Dam Safety Administrative Rules govern the design, operation and maintenance of high hazard dams (dam safety rules are part of the Administrative Rules of Montana (ARM)). Dam safety rules for Montana dams came about through a growing awareness of the potential danger presented by dams and the need for regular maintenance and operation procedures.

3.1 Minimum Required Inflow Design Flood (IDF)

Central to the determination of the IDF is the determination of Loss of Life (LOL), which is the basis for determining the minimum flood frequency that a high hazard dam must safely pass.

3.1.1 Estimated Loss of Life Determination

One of the most subjective and misunderstood components of dam safety is the determination of LOL from a dam failure, yet LOL is the deciding factor in determining the minimum size of the IDF for high hazard dams in Montana. Because of the varying approaches to calculating LOL, the Montana Dam Safety Program has provided **Technical Note 2 (TN2), Estimating Loss of Life for Determining the Minimum Inflow Design Flood Recurrence Interval**, which describes the methodology for estimating LOL that is acceptable to the MT DSP.

3.1.2 Inflow Design Flood Determination

The procedure to compute the minimum required IDF recurrence interval (annual exceedance probability, or frequency) for high hazard dams depends on whether the LOL is greater than, equal to, or less than 5. For an LOL less than or equal to 5, the IDF frequency is determined from the corresponding frequency precipitation depth. An LOL greater than 5 requires the IDF frequency to be computed based on a precipitation depth that is interpolated between the 5,000-year frequency precipitation depth and the PMP.

The IDF is usually computed by applying a “design” precipitation depth (with a frequency that is equal to the required IDF frequency) to a rainfall-runoff model, but other methods may be acceptable with adequate justification. Methods to determine the “design” precipitation depth are described in Section 7. The precipitation event with a given frequency rarely produces a flood of the same frequency, but this is a common assumption in rainfall-runoff modeling. Even if two precipitation storms have identical total rainfall depths, they rarely produce equal runoff hydrographs because of differences in the temporal and spatial distribution of precipitation and variability in other basin parameters. Therefore, the user should verify rainfall-runoff model results, to the extent possible, with independent statistical estimates of flood frequency; Section 10 discusses this issue in greater detail.

For some dams, the IDF may be determined by conducting statistical analyses on stream gage data, but two conditions have to be met: 1) the reservoir must exist on a stream that has a nearby gage with a long-term record; and 2) the gage record must be long enough to produce a statistically valid estimate of the IDF recurrence interval discharge.

For dams with LOL less than or equal to 0.5, the IDF can be developed for the 500-year flood by first estimating the peak flood magnitude using the appropriate regional regression equation for an ungaged basin (Montana Streamstats) and comparing with the calculated spillway capacity. Note that this may be conservative as routing effects of the reservoir are ignored. Thus, if a dam with a LOL ≤ 0.5 does not pass the IDF, it may be worth developing a rainfall runoff model. Additional information on screening methods can be found in Section **Error! Reference source not found.** of this document.

3.1.2.1 IDF for LOL ≤ 5

If the estimated LOL is greater than zero and less than or equal to 0.5, the minimum IDF is the 500-year recurrence interval runoff event (i.e., the 500-year flood). (Yes, LOL can be a fraction! This is part of the reason for some of the misunderstanding surrounding LOL and the need for using the procedures in TN2.)

If the estimated LOL is greater than 0.5 and less than or equal to 5, the minimum IDF recurrence interval is determined by multiplying the LOL by 1,000. The general equation for the minimum IDF frequency for LOL ≤ 5 is:

Inflow Design Flood (IDF) Frequency = LOL x 1,000

3.1.2.2 IDF for LOL > 5

If the LOL is greater than 5 and less than 1,000, the IDF is computed from a precipitation depth that is between the 5,000-year depth and the probable maximum precipitation (PMP) depth. The equations to compute the appropriate design storm depth are defined in the Dam Safety Administrative Rules of Montana and are provided below:

$$P_S = P_{5,000}(10^{rd})$$
$$r = -0.304 + (0.435)log_{10}(LOL)$$
$$d = log_{10}(PMP) - log_{10}\left(P_{5,000}\right)$$

LOL = Loss of Life

PMP = Probable Maximum Precipitation

$P_{5,000}$ = 5,000-year recurrence interval precipitation depth

P_S = Design precipitation depth

For an LOL greater than or equal to 1,000, the IDF is the probable maximum flood, or PMF. The PMF is the runoff produced by the PMP.

4 BASIN CHARACTERISTICS

Basin characteristics are all of the physical features of a watershed that affect the volume and timing of rainfall-runoff hydrograph.

4.1 Area

The area of the drainage basin is all of the area from which rainfall-runoff will flow into the reservoir. For many basins, boundaries can be obtained from the USGS hydrologic unit maps for the hydrologic unit code (HUC) in which the study basin resides.

4.1.1 Subbasins

If the drainage basin has sub-areas with distinctly different characteristics (e.g., soils, terrain, land cover), or that contain separate tributaries leading to a main drainage, it may be appropriate to divide the basin into subbasins. Each subbasin is treated as a separate rainfall-runoff unit, but the connection of each subbasin to the reservoir, either directly connected to the reservoir or "routed" through successive downstream basins to the reservoir, must be done in such a way as to represent, as closely as possible, the actual rainfall-runoff flow paths and conditions. A list of compelling reasons to subdivide a study basin into subbasis is presented below:

1. The study area is greater than 500 mi^2.
2. To stay within the lower or upper limits of basin area size for the USGS regional regression equations if regression equations are used for pseudo-calibration.
3. There is no rationale for simplification (such as for design vs. evaluation of an existing spillway).
4. There is a reservoir upstream from the study reservoir that must be accounted for in the study.
5. There is a stream gage site upstream from the study reservoir that will be useful in developing basin parameters and for model verification.
6. The flow characteristics in different parts of the study basin are needed for some other reason (e.g., flood study).
7. It is not reasonable to assume uniform precipitation over the study basin, such as when the study area includes mountainous and prairie areas.
8. The study basin has areas that are distinctly “non-typical”:
    a. The study basin includes both mountainous and prairie areas.
    b. The study basin has a unique shape.
    c. There are obvious groupings of consistent basin characteristics across the study area, such as soil type, large areas of rock outcrops, or vegetative cover.

4.2 Soils

Accurate identification of soil types in the basin is essential to accurately predicting runoff. In most cases it is impractical to conduct comprehensive physical soil sampling and testing to determine soil types, and the modeler must rely upon publicly available soil surveys. Much of Montana, including state, federal and private lands, has been “mapped” by the USDA Natural Resources Conservation Service (NRCS). However, some areas, such as U.S. Forest Service lands and private land within and adjacent to US Forest Service lands, were not mapped by the NRCS. In such areas, the U.S. Forest Service may have unpublished data, and modelers are encouraged contact the appropriate National Forest office to determine what data, if any, are available. If soil data are not available, soil sampling and testing, including infiltration tests, are warranted as the basis for estimating soil parameters.

4.2.1 NRCS Web Soil Survey

The NRCS provides a *Web Soil Survey (WSS)* site to assist in determining soil properties for a specified "area of interest" which, in the case of a hydrologic model, is the drainage basin being modeled. The *WSS* includes available soils data for most federal land in its inventory. Soils data can be downloaded from the WSS in a geodatabase format, and there is an ArcMap extension (Soil Data Viewer) that can be used to create soil-based thematic maps and perform geo-spatial analysis of soil characteristics, such as hydrologic soil group.

4.2.2 NRCS Soil Surveys

NRCS Soil Surveys are published for most areas throughout Montana by county and are available through the Web Soil Survey. However, some soil surveys, such as the Beaverhead National Forest Area Soil Survey, are archived in pdf format.

4.3 Land Use and Ground Cover

While soil is important in determining infiltration rates and water holding capacity, it is equally important to determine the land cover types in the study basin. These factors greatly influence precipitation loss parameters, as in determining an appropriate NRCS curve number (see Section 4.5.2.1). Commonly, areas with thick vegetation or healthy forests have larger "initial abstractions" delayed runoff responses, when compared to basins with poor vegetative cover. Conversely, urbanization and human development produce impervious areas that reduce or prevent infiltration, resulting in a shorter response time and more runoff than would occur in a natural setting.

4.3.1 Percent Impervious Area

If a basin contains areas that prevent infiltration, such as frozen ground, open water bodies, pavement, rock, etc., these surfaces are accounted for in the model as "percent impervious area". Greater percentages of impervious areas produce higher runoff volumes. Percent imperious area can be used as a calibration parameter when verifying model results.

There are two "types" of impervious area: directly connected impervious areas (DCIA), which has a direct hydraulic connection to the reservoir, and isolated impervious areas (IA), which is impervious area that does not have a direct hydraulic connection to the reservoir. In basins with

DCIA, the percent of the basin area that is DCIA is what is entered into HEC-HMS as “percent impervious area”. For basins with IA (not DCIA), the impervious area is accounted for in the computation of the area weighted loss parameter (e.g., NRCS CN or infiltration rate).

4.3.2 Hydrologic Condition and Antecedent Moisture Condition

According to the NRCS, hydrologic condition refers to the combination of factors that affect infiltration and runoff, including vegetation density and canopy of vegetated areas and surface roughness. Poor hydrologic condition indicates impaired infiltration and increased runoff, while good hydrologic condition is the opposite. Antecedent moisture condition is a measure of the soil moisture just prior to a storm event. Estimating hydrologic condition and antecedent moisture have a significant impact on the IDF produced using a rainfall-runoff model, and the modeler must exercise judgement to select values that represent the conditions that are likely to exist when the “design storm” occurs.

Hydrologic condition of a basin can typically be estimated from aerial imagery and land cover maps based on the factors listed above. However, soil moisture constantly changes, and predicting – with certainty – the water holding capacity of a basin when the design storm occurs is not possible. If a basin has not experienced rainfall for a period of time and is relatively dry, the capacity of the soil to “abstract” rainfall is greater than if the soil is relatively wet. Conversely, for relatively wet, or saturated, soils, the capacity of the soil to “abstract” rainfall is greatly reduced, resulting in a more severe rainfall runoff hydrograph, including a greater total runoff volume. Hydrologic condition and antecedent moisture condition are typically accounted for in a hydrologic model by infiltration variables, which are discussed in Section 4.5. The rainfall-runoff modeler should evaluate a range of antecedent moisture conditions to understand the sensitivity to antecedent conditions.

4.4 Unit Hydrograph Determination

The most common method to define the shape of a runoff hydrograph for a given basin is to determine its unit hydrograph. First introduced by Sherman (1932), the unit hydrograph is defined as a hydrograph of 1 inch of rainfall excess (effective precipitation) or direct runoff (the portion of rainfall that is available for runoff after losses) for a specified rainfall duration. It is also assumed that the rainfall is distributed uniformly over the entire basin. The unit hydrograph is considered to be a function of basin characteristics. The theory of a unit hydrograph is that runoff is linearly related to rainfall excess, and "individual" hydrographs are obtained by multiplying the ordinates of the unit hydrograph by successive rainfall excess increments. Therefore, runoff from complex storms of any duration with sequential increments varying in depth can be estimated by superimposing and adding a series of individual hydrographs. Many hydrology references are available to provide additional discussion and detail on the theory and application of unit hydrographs.

4.4.1 Synthetic Unit Hydrograph Development

Most applications in dam safety will involve developing a synthetic (or dimensionless) unit hydrograph on an ungaged basin. Many methods have been developed for estimating synthetic unit hydrographs, but because of the work accomplished by the USGS specifically for Montana, this technical note will focus on two related methods, and only include a brief discussion of other common methods.

4.4.1.1 Montana Procedures (USGS WSP 2420)

Water-Supply Paper 2420, Procedures for Estimating Unit Hydrographs for Large Floods at Ungaged Sites in Montana (U.S. Geological Survey, 1996) presents procedures for determining unit hydrograph parameters that were derived from streamflow and rainfall records in Montana. The USGS study was initiated because of the subjectivity in determining unit hydrographs in dam safety applications, and the MT DSP recognizes *WSP 2420* as a reliable reference for developing unit hydrographs for dams in Montana. *WSP 2420* presents procedures for determining unit hydrograph parameters for two methods: the Clark unit hydrograph and the dimensionless unit hydrograph.

Clark (1945) developed a method to define unit hydrographs using three components: the time of concentration (Tc); a basin storage coefficient (R); and a time-area curve. Using regression analysis of rainfall and runoff gage data, the USGS developed equations to calculate the basin specific Tc and R for use with the Clark unit hydrograph. Although the time-area curve may be developed independently, it is most common to utilize the default time-area curve from Figure 3 of *WSP 2420* (which is also the default time-area relationship in HEC-HMS).

The dimensionless unit hydrograph is a versatile tool when comparing drainage basins with varying characteristics. In the USGS study, unit hydrographs were made dimensionless by dividing the discharge at each time step by the "standard lag" time, as defined by Snyder (1938).

This is the same procedure described in the Flood Hydrology Manual (Bureau of Reclamation, 1989). The procedures described in *WSP 2420* to calculate unit hydrographs are too extensive for this technical note; however, a spreadsheet is available through the MT DSP that will perform the calculations.

To properly apply *WSP 2420* to dam safety applications, it is necessary for the user to be familiar with the concepts and procedures presented in *WSP 2420*. While the parameters for determining unit hydrographs were developed from hydrologic data obtained throughout Montana, there is scatter in the data used to derive the different regression equations. The user must understand the reliability of the equations and the range of values in which the equations are valid prior to application to a specific basin. *WSP 2420* fully explains the limitations and design considerations of the study. Several considerations and limitations include:

- The methods are intended for use at ungaged sites where no site-specific information is available.
- Unit hydrograph characteristics were generally derived from general storm (or long duration) rainfall events (as opposed to local storm (thunderstorm) events). These general storms are some of the largest storms recorded in Montana and are typically the controlling storm type for larger basins.
- The equations developed to estimate lag time are not valid for use in other dimensionless unit hydrograph methods, such as the methods described in the Flood Hydrology Manual (Bureau of Reclamation, 1989).
- The unit hydrograph duration needs to be small enough to adequately define the shape of the hydrograph peak (i.e., accurately capture the peak runoff rate). Rules for calculating the unit duration are given in the study.
- The methods are applicable to basins small enough that variations in areal runoff do not affect the hydrograph shape. If the calculated unit duration is greater than 7 hours, the basin may require subdividing. Regardless of unit duration, basins over 500 square miles are to be subdivided.
- Equations for adjusting the peak and shape of dimensionless unit hydrographs are invalid when the desired dimensionless peak discharge is less than 8.0.
- The study estimation methods are valid only within the range of variables shown in the tables. Figures 15-19 of *WSP 2420* should be reviewed to ensure that the basin parameters are within the range of data used in *WSP 2420*

4.4.1.2 Other Methods

In addition to the procedures described in *WSP 2420*, there are a number of other synthetic and dimensionless unit hydrograph methods that are commonly applied to basins throughout Montana, but the most common alternate method is likely the NRCS dimensionless unit hydrograph. Of note, the modeler must be aware that the default "peaking factor" of the NRCS dimensionless unit hydrograph is 484, which is not appropriate for all basins. Additional information on the NRCS dimensionless unit hydrograph, including information on selecting an appropriate peaking factor, is available in the Unit Hydrograph (UHG) Technical Manual available through NOAA.

4.5 Rainfall Losses

Not all rainfall is effective precipitation, or that portion of rainfall that becomes surface runoff. Understanding how rainfall is intercepted, is stored in ground depressions, and infiltrates is key to determining the volume and timing of runoff. Rainfall losses are typically divided into two major components, initial abstraction (interception and depression storage) and infiltration. While other parameters affect the shape of the rainfall-runoff hydrograph (e.g., the slope and timing of the rising and falling limbs of the hydrograph), rainfall losses directly control the volume of the rainfall-runoff hydrograph. For some dam and reservoir systems, such as when the reservoir size is large relative to the total basin area, volume, rather than peak inflow rate, is the more significant parameter in successfully passing (or storing) the IDF.

When developing a rainfall-runoff model to compute an IDF, rainfall losses must be represented as accurately as possible, and the modeler must understand that a basin's capacity to retain precipitation through initial abstraction and infiltration vary both spatially and temporally. Modelers must exercise judgement to select values to best represent the conditions likely to exist when the "design storm" is most likely to occur. For example, as was observed in 2011, severe flooding (i.e. design events) often occurs in response to a series of storms that pre-wet, or even saturate, a basin. For this reason, it's not likely that the antecedent conditions of a basin will be dry, or even "normal" (i.e, antecedent moisture condition II), when an extreme storm occurs, it's far more likely that antecedent conditions will be such that a basin's capacity to retain precipitation is greatly diminished. The following sections provide guidance on some of the methods and assumptions for determining rainfall losses.

4.5.1 Initial Abstraction

As defined by the NRCS, initial abstraction is the portion of rainfall that is lost before it becomes effective precipitation (or surface runoff). Initial abstraction includes interception (rainfall that is held by vegetation or structures before reaching the ground), infiltration during the early portion of the storm, and depression storage (surface storage areas that trap and retain rainfall). Most hydrology texts provide a general description of the mechanics of the component processes for initial abstraction, and initial abstraction can be estimated by several different methods.

The most common equation for determining the initial abstraction for a basin that is characterized as having "average" runoff potential is empirical and used in conjunction with the NRCS curve number method (see Section 4.5.2.1), which is

$$I_a = 0.2S$$

Where I_a = initial abstraction, inches

S = potential maximum retention after runoff begins, inches = $1000/CN - 10$

CN = soil complex curve number

As previously stated, the assumption that initial abstraction is equal to 0.2S is for a basin of "average" runoff potential, or antecedent runoff condition (ARC) II, as opposed to a "dry" basin (ARC I) or ARC III for "wetter" conditions. Adjusting initial abstraction to account for antecedent conditions can be accomplished in one of three ways. First, the modeler can select the applicable curve number for the basin's average land cover **and** ARC III conditions, the modeler can use a smaller storage coefficient (i.e., <0.2), or a combination (i.e., ARC III and a smaller storage coefficient). Since 2000, there has been much discussion and debate about the use of 0.2 as the storage coefficient, including **when**, **or** even **if**, 0.2 is a reasonable estimate for initial abstraction. More information on application of the NRCS method to estimate initial abstraction is provided in the NEH Part 630 Hydrology, Chapter 10 – Estimation of Direct Runoff from Storm Rainfall:

Note that when using the NRCS curve number method (refer to section 4.5.2.1) in HEC-HMS, if the initial abstraction field is left blank, HEC-HMS automatically calculates the initial abstraction as equal to 0.2S.

4.5.2 Infiltration

For this application, infiltration refers to rainfall lost to the soil after initial abstraction, once surface runoff has started. Infiltration rates vary with soil texture. In general terms, infiltration rates in coarse grained or sandy soils are much higher than fine grained or clayey soils. Infiltration rates for a given soil are not linear over time when water is continuously ponded at the surface. Given enough time with water ponded over the soil column, infiltration will approach a steady, constant rate. Infiltration rates can be estimated by several different methods, including those briefly described below:

4.5.2.1 NRCS Curve Number

The USDA Natural Resources Conservation Service (NRCS) developed the curve number model through empirical research on a broad range of soil types and cover combinations. It is probably the most popular method for computing rainfall-runoff hydrographs because of its ease of application and extensive database of parameters. The curve number model divides rainfall into three components: rainfall excess (runoff), initial abstraction, and retention. Through a series of computational relationships between all three components, the NRCS developed hydrologic soil-cover complexes. Each complex was assigned a curve number, or CN, which indicates the runoff potential of a soil complex during periods when the soil is not frozen. High curve numbers indicate high potential for runoff. The highest possible CN is 100, which means that all rainfall becomes runoff and there are no abstractions.

In order to use the CN method, the user has to know the soil type and land cover types for the basin being considered. The user also has to determine the hydrologic condition of the cover vegetation (whether it is thick or sparse) and assume an antecedent soil moisture condition. Reference tables for assigning curve numbers are available in NEH Part 630 Hydrology, Chapter 10 – Estimation of Direct Runoff from Storm Rainfall:

While this method is empirically-based and has been shown to be less accurate than some physically-based infiltration models, it is one of the most widely used methods. It is a relatively easy method to estimate rainfall losses, especially over large basins. To the extent possible, the user should verify the results of a rainfall-runoff model with an independent analysis of peak discharge values and adjust CN values appropriately, as described in Section 10 of this technical note.

Note that curve numbers can give a quick estimate of direct runoff volume from a basin using the following NRCS equation:

$$R = \frac{(P - 0.2S')^2}{(P + 0.8S')}$$

Where R = Rainfall excess, inches (which can be converted to volume by multiplying by the basin area)

P = total precipitation in inches

S' = Maximum storage volume = (1000 – CN)/10

CN = curve number

4.5.2.2 Constant Loss Rate

The constant loss rate for infiltration is used when the assumption is that the soil is saturated and infiltration has reached a minimum, steady value. Various tables and data sources are available with estimated constant loss rate values for different soil types. The Technical Reference Manual for HEC-HMS has such a table with ranges of constant loss values for soil types used for CN estimations, the Flood Hydrology Manual presents a table of "ultimate" infiltration rates based on hydrologic soil group, and the saturated soil hydraulic conductivity is generally available for soils mapped by the NRCS and are available through the Web Soil Survey.

4.5.2.3 Other Methods

Other infiltration models may be appropriate and, when properly used, can provide accurate estimates of rainfall infiltration. General input parameters are available for other infiltration models, but most require site specific infiltration data, which is typically expensive to obtain. References for the following infiltration models are included in the HEC-HMS Technical Reference Manual.

Horton Model – Derived empirically but based on physical relationships, the Horton Model is an exponential relationship that defines infiltration rate over time assuming water is continuously ponded over the soil. Infiltration declines exponentially until it asymptotically approaches a minimum constant rate.

Green-Ampt Model – Also called the delta function model, it was derived as both a physical and empirical model that assumes water is constantly ponded on the soil surface and that as the water moves downward in the soil, there is a sharp interface where wet soil meets dry soil. Infiltration is affected by soil hydraulic conductivity, wetting front suction head and porosity.

5 CHANNEL ROUTING METHODS

If there is a component to the hydrologic model that requires conveyance of runoff through an established channel, such as a stream, river, or prismatic channel, the model will require a specified method to route the hydrograph. Channel routing is necessary when a rainfall-runoff model contains one or more subbasins, where the runoff hydrograph from the "upper" subbasin(s) must be conveyed, or routed, through the "lower" subbasin(s) before reaching the reservoir. Attenuation of the hydrograph (reduction of peak discharge and increasing the duration of the hydrograph curve) occurs because of energy loss and storage as the hydrograph is translated downstream.

As described below, several computational methods are available to route a hydrograph, and each method has its limitations and conditions under which it is either an appropriate choice or not applicable. Before selection of a channel routing method, the modeler must understand these limitations and determine which channel routing methods are applicable to the study basin. Each of these models solves, in various degrees of simplification, the fundamental equations of open channel flow: the continuity and momentum equations. In-depth discussion of channel routing methods are beyond the scope of this technical note, but the HEC-HMS Technical Reference Manual provides detailed discussions on channel routing methods and includes a table (presented below) that lists the routing methods available in HEC-HMS and the conditions under which each method is suitable for use (refer to Applicability and Limitations of Routing Models in Chapter 8 Modeling Channel Flow of the HEC-HMS Technical Reference Manual). Following the table is a brief discussion on the most common channel routing methods.

Table 19. Guidelines for selecting routing model.

If this is true...	...then consider this model
No observed hydrograph data available for calibration	Kinematic wave; Muskingum-Cunge
Significant backwater will influence discharge hydrograph	Modified Puls
Flood wave will go out of bank, into floodplain	Modified Puls, Muskingum-Cunge with 8-point cross section
Channel slope > 0.002 and $\frac{TS_o u_o}{d_o} \geq 171$	Any
Channel slopes from 0.002 to 0.0004 and $\frac{TS_o u_o}{d_o} \geq 171$	Muskingum-Cunge; modified Puls; Muskingum
Channel slope < 0.0004 and $TS_o \left(\frac{g}{d_o}\right)^{1/2} \geq 30$	Muskingum-Cunge
Channel slope < 0.0004 and $TS_o \left(\frac{g}{d_o}\right)^{1/2} < 30$	None

Figure 1: Guidelines for Selecting a Channel Routing Method

Muskingum-Cunge Routing – This method combines the conservation of mass and an approximation of diffusion in the conservation of momentum equation. It is sometimes referred to as a variable coefficient method because the routing parameters are recalculated every time step based on channel properties and the flow depth. It represents attenuation of flood waves and can be used in reaches with a mild slope. The user has a choice of five types of cross sections to enter into the routing procedure: rectangular, trapezoidal, triangular, circular or an eight-point section. The eight-point section makes this method useful for natural channels and is based on an average stream and floodplain cross section for the selected reach. Reaches should be determined on the basis of each having similar cross-sectional properties. Manning's 'n' values for channel and left and right overbanks are to be input by the user. Manning's 'n' values should be average numbers for each reach. Note that depending on the channel geometry, a wetting flow (i.e., minimal baseflow) is sometimes necessary to maintain numerical stability during the computation process.

Modified Puls Routing – Also called storage or level pool routing, this method uses a relationship between water storage in a stream reach and discharge from the reach to route flow. It is through calculated storage and release of detained water that attenuation is determined. It can be used to calculate backwater from channel constrictions, as long as the backwater is contained in the stream reach (i.e., backwater does not rise to a level at which it can bypass the constriction). The process requires input of a stage-discharge function for each reach. This can be obtained from hydrograph analysis or by constructing a HEC-RAS analysis of the reach to determine a stage-discharge relationship. The user also determines an initial flow condition by one of three options: initial discharge, outflow equals inflow, or an elevation-discharge relationship.

Kinematic Wave Routing – This process ignores the inertial and pressure forces of the full unsteady flow equations. It is better suited for relatively steep streams and channels that can be approximated by regular shapes and slopes, such as man-made channels. Five options are provided for determining the channel cross sectional shape: circle (for open channel flow in circular channels – not for pressure flow in a pipe), deep (for shapes where depth is approximately equal to width), rectangle, trapezoid, and triangle.

6 BASEFLOW & SNOWMELT

When construction a hydrologic model to determine an IDF and evaluate the capacity of a reservoir and dam system to safely pass the IDF, the modeler must consider the need to also account for "baseflow". Baseflow is the "normal" flowrate through a water course, consisting of runoff from groundwater or antecedent storms. In addition, for the purposes of evaluating the hydrologic capacity of a reservoir and dam system, it may also be necessary to account for the potential for snowmelt inflow to occur concurrently with the design storm.

While most extreme floods in Montana are in response to a rainfall event, many of Montana's drainages have the potential for snowmelt to contribute significant water volume and, in some cases, significantly increase the peak flow rate. Spring and early summer storms may occur on snowpack, creating rain-on-snow events that have the potential to be severe. Mostly notably, the June 1964 storm along the northern Rocky Mountain front that caused nearly 20 fatalities and several dam failures was a rain-on-snow event. Given the right meteorological conditions, high elevation watersheds have the potential for significant runoff from only snowmelt.

However, effective and accurate modeling of snowmelt is difficult and tends to be very data intensive. A simplified contribution of reservoir inflow from snow melt can be developed in several ways. One way is to assume the 100-year peak flood magnitude (computed by other methods, such as USGS regional regression equations or from regression analysis of nearby stream gages) is largely a snowmelt event, which is not an unreasonable assumption for many drainages in Montana, and to include a steady-state snowmelt hydrograph with ordinates equal to the 100-year flood value.

Another way is to estimate an average snowmelt discharge using data obtained from NRCS SNOTEL stations, which are remote, high mountain snow data collection locations, for providing snowpack runoff data. SNOTEL stands for SNOwpack TELemetry. In addition to providing data that may be used to determine average or typical snowmelt runoff rates for a basin, nearby SNOTEL stations may provide data that can be used to confirm that a spillway is sized adequately to handle the largest snowmelt event of record. For some small, high mountain dams, this maximum snowmelt rate may be larger than the spillway design inflow calculated from precipitation events.

When evaluating the need to account for the effect of baseflow or snowmelt on the hydrologic capacity of a reservoir and dam system, there are several factors that should be considered:

- The principal reason to include baseflow and/or snowmelt is to account for the antecedent inflow conditions that are likely to exist when the design storm occurs.
- The volume of the baseflow or snowmelt hydrograph is often more significant than the peak of the baseflow or snowmelt hydrograph; this is usually the case for small reservoirs.
- Snowmelt hydrographs are typically “flat” (i.e., the flow rate is relatively steady over time) and have a relatively long duration (e.g., several days to weeks).

7 PRECIPITATION ESTIMATES FOR MONTANA

For use in rainfall-runoff modeling, precipitation is the catalyst for runoff. Four components of precipitation need to be determined to define how runoff will occur: depth, duration, temporal distribution, and spatial distribution. In response to the effort by DNRC to base an IDF on the estimated Loss of Life from dam failure, and move away from a strict PMF standard or standards based on reservoir size, the USGS developed methods to derive precipitation depth and temporal distributions (i.e., storm pattern) for storms in Montana with recurrence intervals of up to 5,000 years. For less frequent storms, rainfall depths are linearly interpolated between the 5,000-year storm and the probable maximum precipitation (PMP).

7.1 Storm Depth and Duration Determination (*USGS WRIR 97-4004*)

For dams having LOL less than or equal to 5 (determined in accordance with TN-2), and subsequently a design storm recurrence interval of between 500 and 5,000 years, the rainfall storm depth is calculated using methods developed by the U.S. Geological Survey (USGS) for extreme precipitation events. The USGS published *Water-Resources Investigations Report (WRIR) 97-4004, Regional Analysis of Annual Precipitation Maxima in Montana* (1997), which provides methods for determining precipitation depths in Montana for extreme storms.

The study utilized statistical procedures to combine record lengths of multiple rainfall gages to develop depth estimates for storms having exceedance probabilities up to one-in-five thousand (a recurrence interval of up to 5,000 years). Dimensionless precipitation frequency curves for homogeneous meteorological regions in Montana were developed based on physiography and climate. Basin specific precipitation depths can be computed for storm durations of 2, 6, and 24-hours.

The study presents a series of regression equations to compute an at-site "mean storm depth", which in turn is used to compute the at-site storm depth for a selected recurrence interval and storm duration. The regression equations use mean annual precipitation and latitude and longitude as variables, and the publication includes a map and list of all the precipitation stations (including the precipitation data) used to develop these regression equations. When applying *WRIR 97-4004*, only the available in *WRIR 97-4004* can be used and it is important that the user review the map of precipitation stations to determine if any of the precipitation stations used to develop the regression equation are in, or adjacent to, the study basin. If so, then use of the precipitation data for nearby precipitation gage(s) is often a more appropriate estimate for storm depths than what is achieved through application of the *WRIR 97-4004* regression equations. The user is encouraged to follow the publication examples in *WRIR 97-4004* to determine storm depths for various applications.

7.1.1 Temporal Distribution (*USGS WRIR 98-4100*)

Following WRIR 97-4004, the USGS published *Water Resources Investigations Report 98-4100, Characteristics of Extreme Storms in Montana and Methods for Constructing Synthetic Storm Hyetographs (1998)*, which identifies temporal distributions of incremental storm depths for 2-, 6-, and 24-hour duration storms, and their application to analyses or design. The study uses the same homogeneous regions as *WRIR 97-4004. WRIR 98-4100* describes how, starting from a total storm depth, incremental depths are calculated and distributed to construct hyetographs. Incremental depths are temporally distributed based on historical occurrences within a specified region and the level of conservatism desired. For dam safety applications, the median (0.5) exceedance probabilities are acceptable for both analysis of existing, and design of new, reservoir and dam systems; this topic was discussed during the Extreme Storm Working Group meetings, and the Group concluded that use of dimensionless depths associated with a 0.2 exceedance probability likely compound conservativism and result in an overly conservative estimate of the total precipitation depth. *WRIR 98-4100* offers examples of hyetograph construction for various applications across the state.

7.1.2 Areal Distribution & Areal Reduction Factors (ARFs)

Specific spatial, or areal, distribution recommendations are not provided in *WRIR 97-4004* or *WRIR 98-4100*. The spatial distribution of rainfall is left to the user and, for most rainfall runoff models, rainfall is assumed to be uniformly distributed over a basin, such as for a study area that includes both mountainous and prairie areas. Where the modeler has reason to believe rainfall will not be uniformly distributed over a basin, then the basin should be subdivided into basins that are likely to experience uniformly distributed rainfall.

It is important for the modeler to understand that the precipitation depths obtained through *WRIR 97-4004* are "point" precipitation depths. For basins of approximately 10 mi^2 and larger, the point precipitation must be converted to an "areal" precipitation depth by application of an areal reduction factor (ARF); note the ARF is applied to the total precipitation depth **or** the incremental hyetograph depths – **not both**. *WRIR 98-4100* provides depth-area reduction curves that are adapted from NOAA Atlas-2 (1973). These curves should be used to compute a basin-wide areal reduction factor for rainfall depth values derived from *WRIR 97-4004*.

7.2 Probable Maximum Precipitation (HMR 55A AND 57)

The National Oceanic and Atmospheric Administration (NOAA) conducted a nationwide study of precipitation events, mainly to address the need to develop data in support of the growing concern over dam safety issues in the late 1970's. Because of the potentially catastrophic consequences of dam failures, NOAA developed methods to estimate rainfall events that potentially create the most severe precipitation possible, given optimum meteorological conditions. These storms are defined as probable maximum precipitation, or PMP, events.

NOAA conducted PMP studies for specific regions throughout the United States, and these studies are available through the Hydrometeorological Design Studies Center. The PMP studies are called Hydrometeorological Reports, or HMRs. Two HMRs cover the state of Montana: HMR No. 55A (1988), which was developed for the area of the United States between the Continental Divide and the 103rd Meridian (the longitudinal line just east of the Montana-North Dakota border); and HMR No. 57 (1994), which is for the Pacific Northwest States of Idaho, Oregon, Montana and Washington between the Continental Divide and the Cascade Mountains.

Since issuance of the HMR series, there has been much debate regarding the accuracy of the documents; however, there is no question that the HMR series is now dated. The Federal Advisory Committee on Water Information's Subcommittee on Hydrology is examining the need for updated PMP documents, but funding for these studies has ceased. In the interim, confidence in the HMR publications is diminishing in the dam safety community, and there is a trend for individual states to conduct state specific PMP studies.

In Montana, a state specific PMP study is not available, and for new or existing dams with a large population at risk (and, hence a large potential for Loss of Life) or severe economic or environmental consequences, the need for site specific PMP studies must be considered. In such cases, the need, and scope and methods, for a site specific PMP study shall be coordinated with the MT DSP. When a site specific PMP study is available, the results of the site specific PMP study are considered to be more reliable than the PMP computed using the applicable HMR.

Regardless of the source of PMP estimates, where PMP is a consideration in determining the design storm depth to compute an IDF, the PMP should be developed by professionals experienced in PMP and PMF determinations.

8 RESERVOIR AND DAM CHARACTERISTICS

The reservoir stage-storage capacity and stage-discharge capacity of the dam (i.e., outlet works and spillway) are needed to route the IDF through the reservoir and dam system. Unless the reservoir has a relatively small volume in relation to the runoff volume from the upstream drainage area, the reservoir pool is typically effective, at least to some degree, in attenuating the IDF hydrograph (i.e., the peak discharge past the dam is less than the peak inflow).

The term “spillway” in this section refers to a reservoir’s principal and auxiliary spillways, which are designed to safely convey the IDF past the dam. The capacity of the low-level outlet is typically ignored in the flood-routing analysis since the discharge capacity of low-level outlet works is typically much smaller than the spillway capacity. Additionally, for low-level outlets that require human intervention to operate, flood events often occur unexpectedly, and flood waters can impair access to the dam, making it difficult to operate the gates. There may be cases, however, where including the low-level outlet capacity when routing the IDF through the dam is justified.

8.1 Resevoir Stage-Area/Volume

For routing the IDF through the reservoir and dam system, the modeler must have an accurate representation of the stage-storage capacity of the reservoir above the principal spillway crest elevation.

Often, reservoir volume information is provided in design documents, such as design reports or drawings, describing the design and construction of the dam. When using record data, one issue to be aware of when using record data is that for many dams (particularly older dams), the record documentation does not identify the vertical datum from which elevations are referenced. In many cases, elevations are with respect to a NGVD29 or a local (or arbitrary) datum, which are often shifted from the NAVD88 datum by only a few feet – making it difficult to identify the difference in elevation datum between different data sources.

Regardless of whether or not storage information is available, computation of reservoir volume above the spillway crest is easily accomplished using GIS tools and publicly available mapping data. Volumes can be computed by interpolating an elevation-area relationship using USGS 7.5 minute quadrangle topographic maps or digital elevation models available through the USGS 3DEP. The method involves the following steps:

1. Determine the area for elevation contours from the spillway crest elevation (e.g., often the blue shaded (water) area shown on quad maps) through several contours above the dam crest elevation. Assume the dam’s location does not change.
2. Develop the stage-storage table using the average-end-area method to compute the incremental volume between contours. Compute the cumulative storage volume with increasing elevation.

3. From the stage-storage table, plot the contour elevations versus area and/or volume.

8.2 Dam Crest Elevation and Width

If reliable dam crest data are not available, the elevation of the low point on the dam crest, and the width of the dam crest, can be measured easily in the field. It is important to obtain the low-point elevation of the crest to understand when overtopping will occur. The crest width is used to estimate flow over the dam if overtopping occurs during model routing. Generally, overtopping flow can be calculated using the broad-crested weir equation with a weir coefficient (C) consistent with the dam crest geometry, surface materials, and overtopping depth (C changes with depth).

8.3 Outlet Conduit Discharge

For most small to medium sized earthen dams, the outlet capacity is much smaller than the spillway capacity and it is usually appropriate to neglect discharge through the low-level outlet when routing the IDF through the reservoir and spillway. This is particularly true for dams that require intervention to open the low-level outlet gate(s) or valve(s).

8.4 Spillway Stage-Discharge

In most cases, the spillway is used only during runoff events. Spillways are typically open channels with relatively prismatic cross sections, or weir structures that have various crest configurations (such as broad-crested or ogee). Stage-discharge information is often included in design documents, such as design reports or drawings, describing the design and construction of the dam. However, in many cases these records no longer exist, and the modeler must develop the stage-discharge relationship.

For simple spillways, the stage-discharge relationship can be developed using the appropriate weir equation (sharp-crested or broad-crested weir), or they can be automatically computed in HEC-HMS. However, it is often more appropriate to develop the stage-discharge curve from of a surface water profile model over a range of flow rates, such as can be developed using HEC-RAS. Using HEC-RAS, the geometry layout will include the approach flow path from the reservoir to the spillway crest, then through the spillway to a point where the tailwater does not affect the discharge capacity of the spillway.

9 MODELING SURFACE RUNOFF AND FLOOD ROUTING

As indicated previously in this technical note, the most common runoff and flood routing model currently in use is HEC-HMS. HEC-HMS is made available to the public at no cost through the U.S. Army Corps of Engineers and is relatively intuitive, user-friendly, and provides reliable results. The software is versatile in that it can be used to compute rainfall-runoff hydrographs and to route hydrographs through reservoirs and spillways. Training in the use of HEC-HMS is beyond the scope of this document, and it is up to the user to understand the applicability and limitations of the model. The Corps of Engineers does not provide technical support to the public, but they do provide a Quick Start Guide, User's Manual, and Technical Reference Manual, and training courses on the use of HEC-HMS are offered through private companies and professional organizations.

It is worth noting that with the release of HEC-RAS v. 5.0, HEC-RAS now has the ability to apply an effective precipitation hyetograph to a terrain model grid and perform two-dimensional (2D) rainfall runoff routing (effective precipitation is the average depth of rainfall that runs off the basin). Discussion of this type of analysis to produce an IDF hydrograph is beyond the current scope of TN-1, but such an approach offers several advantages over the traditional unit hydrograph and dimensionless unit hydrograph methods for developing IDFs.

9.1 Phased Approach

Modeling flood routing for evaluating spillway adequacy can be a detailed and complex effort. When the cost for conducting such analyses is considered, there can be financial benefit to the owner in evaluating methods to improve the efficiency in the evaluation process. In other words, if there is no reduction in the construction cost for a dam and spillway system based on an optimized IDF, then there is no benefit to the owner in paying for a more detailed study. There are a number of reasons where the site characteristics may warrant a lesser level of analysis, such as there is a need to waste material, there is a need for fill material, or if a greater level of analysis is not likely to appreciably reduce the IDF or the size of the spillway. Conversely, if construction costs can be reduced by optimizing the hydrologic analysis, then there is benefit to the owner in pursuing a greater level of analysis. For this reason, this section offers suggestions for a phased approach and study phasing when determining IDFs.

If detailed information is available and building a detailed model is relatively easy, then it is always better to build a detailed model. However, if making conservative assumptions, such sizing a spillway to pass the peak inflow rate of the IDF (e.g., design the spillway based on the discharge computed using the USGS Regional Regression Equations), can demonstrate an existing dam and spillway have adequate hydraulic capacity or results in a cost-effective design of a dam and spillway system, then no further analysis may be necessary. When conservative assumptions yield unfavorable or undesirable results, a more detailed analysis is necessary. The following steps outline an initial approach for computing an IDF and assessing spillway adequacy:

1. Determine the minimum acceptable recurrence interval of the IDF and the design precipitation:
    a. Design storm depth for the required return interval. For dams with estimated large LOL, or that are suspected to have sufficient capacity to pass the PMF, it may be more efficient to evaluate at the PMF first. If the reservoir passes the PMF, then there is no need to estimate the LOL downstream or calculate the IDF.
2. Conservative assumptions in computing the IDF:
    a. Use a larger design storm depth than is required. For example, if the minimum acceptable IDF is based on an LOL of 0.5, consider basing the IDF on a larger LOL, such as an LOL of 1 or 2. If the dam and reservoir system can safety pass a significantly larger flood than the minimum acceptable IDF, then there may be little value in expending the effort necessary to refine the rainfall-runoff model parameters. Similarly, consistent with the guidance in TN-2, basing the IDF on a larger LOL allows the designer and owner to understand potential need for spillway improvements should the estimated LOL increase.
    b. Assume no initial abstraction (Ia) and little to no rainfall losses. Note that when applying the NRCS Curve Number method in HEC-HMS, if the initial abstraction field is left **blank**, HEC-HMS will compute initial abstraction using the equation Ia = 0.2S; if you are following this approach, make sure you input a zero.
3. Conservative assumptions when evaluating spillway adequacy:
    a. Compare the total runoff volume to the storage volume between the spillway crest and the dam crest. If the entire runoff volume can be stored without overtopping the dam, no further analysis is necessary.

b. Compare spillway capacity to the IDF peak inflow rate, without taking into account routing effects of the reservoir. If spillway capacity at the lowest dam crest elevation exceeds the IDF peak inflow rate, the spillway has adequate hydraulic capacity and no further analysis is necessary. This is a particularly useful initial comparison when evaluating the adequacy (i.e., hydraulic capacity) of an existing spillway or if the IDF is the 500-year flood and the peak inflow rate can be calculated with the USGS regional regression equations.

4. If the initial assessment using any of the above simplifying assumptions produces inadequate or undesirable results, progressively greater levels of detail must be incorporated into model.

10 MODEL VERIFICATION

10.1 Uncertainty

Streamflow gages exist on many waterways but are more common on larger rivers. Most small streams are not gaged. Studies have been conducted by the USGS and other agencies that produced regression relationships to help predict flow frequency and magnitude on ungaged streams. While the regression equations developed for ungaged streams are reasonably accurate, they are based on gage data from regional streams, which have limited record lengths, usually no more than about one hundred years. So when spillway hydrologic analyses are conducted for runoff events that exceed a return interval of 500 years and greater the uncertainty associated with the flow estimates can be very significant.

Parameter estimates for basin characteristics tend to be general in nature, especially in larger basins, and are based on the best available data or empirical relationships. Parameter selection, such as soil and ground cover type, infiltration rates, and unit hydrograph parameters introduce uncertainty to the model.

The same is true for meteorological data, where gage data is likely not available for watersheds being considered, and where gage records, if available, are relatively short in length compared to the return intervals required for spillway analyses. User decisions for temporal and spatial distribution are subjective, though there are references for Montana that provide guidance based on statistical analyses of actual gage data. Regardless, rainfall and snowmelt input to a hydrologic model will invariably introduce additional uncertainty.

But any uncertainty introduced into a model is balanced against what should be considered reasonable when using the best available data. The methods included in this technical note provide references to data that are considered to be the best available for Montana. The goal of “reasonable” is reached when model verification is accomplished to within standard error bounds of recorded gage data or regression equations. Variability of model results will still exist among different users and according to differing objectives for specific spillway analyses, but hopefully the procedures for verification mentioned in this section will produce somewhat consistent methodologies.

A hydrologic model is only as “good” (accurate) as the model input. A crucial step in the modeling process is some form of verification to assess the reasonableness of the model and to develop confidence in the computed IDF. In dam safety, the consequences of an inappropriately small IDF can be severe. Underestimating the IDF can lead to dam failure, which can be devastating to the downstream public, the owner, and the engineer of record. However, overestimating the IDF may result in a design for which there is inadequate funding, and necessary repairs/reconstruction may not occur, which also increases the risk to the downstream public. Putting effort into verifying the model may cost more in engineering analysis, but usually is a relatively small amount if done in a logical and professional manner, especially compared to the construction cost of oversized dam features or the tragedy that can result from a dam failure.

Ideally, a rainfall-runoff model would be validated by comparing the model results to streamflow and precipitation data obtained at the site of interest for several large storm events. With such data, the rainfall-runoff model could be calibrated using data from one large storm and validated using data from another large storm. Practically, however, that kind of at-site data is rarely available when computing hydrology for dam safety evaluations. For most dam safety evaluations, model validation is the process by which the engineer verifies model results against independent flood-frequency estimates and “pseudo-calibrates” the model such that model results are reasonably consistent with results from those independent methods. A verified model is one that is demonstrated to produce reasonably acceptable peak runoff rates for floods having 100-year and 500-year recurrence intervals and, on that basis, is determined to be acceptable for computing the IDF. After computation of the IDF, the IDF is evaluated for reasonableness by comparing the maximum inflow rate of the IDF against measured historical peak stream discharge rates throughout the region (i.e., an envelope curve – refer to Figure 2). Thus, in this way, the IDF is determined to be validated.

Definitions for key terms in the model verification process, and the model verification workflow, are presented below.

10.2 Definitions

10.2.1 Verification

The process in which the flood frequency results from a hydrologic model (total runoff volumes and/or peak runoff rates) are compared to flood frequency estimates developed from an independent method (or methods). For example, comparing peak runoff rates from the rainfall runoff model for the 500-year flood against the USGS regional regression equation for the 500-year peak flood magnitude at the same location.

10.2.2 Calibration

The process in which the parameters of a hydrologic model are adjusted to replicate a measured (or observed) event. For example, adjusting loss rates down or up such that the model results for a 50-year storm match gage data for a measured 50-year event on the same drainage.

10.2.3 Pseudo-Calibration

The process in which the parameters of a hydrologic model are adjusted to reasonably approximate a range of flood frequency values obtained independently from the rainfall-runoff model (refer to Section 10.3 Verification), such as the 100- and 500-year flood magnitudes. For example, adjusting model parameters based on comparing model results with the 100-year and 500-year flood magnitude estimates computed using the USGS regional regression equations.

10.2.4 Simulation

The process of using a validated model to produce flood frequency hydrographs and/or an IDF hydrograph.

10.3 Verification

For dam safety evaluations, the engineer must verify that the hydrologic model used to compute the IDF is producing reasonable results. Because IDFs are large, infrequent floods (i.e., $\geq$ 500-year recurrence interval), verification of the hydrologic model must be accomplished through comparison of model results for smaller floods (e.g., 100-year and 500-year recurrence intervals) that are, in turn, verified against results from other, independent methods for computing flood frequency.

In general, if the hydrologic model produces a result within one standard deviation or one standard error of prediction (SEP) above the value computed from one of the acceptable methods for verification, the hydrologic model can be considered verified. Statistical measures of reliability are not comparable among the accepted verification methods, however, and may not even be available for some methods. For situations where the one standard deviation or one SEP cannot be reliably calculated, such as when using local regression equations as the basis for verification, an acceptable reasonable upper bound for a modeled result would be twice the value of the result from the selected verification method. Regardless of the method employed to verify the reasonableness of the rainfall-runoff model results, the engineer must document and justify the basis for concluding the IDF is acceptable.

10.4 Pseudo-Calibration / Calibration

When the verification process shows the hydrologic model is not producing results that are reasonably consistent with other methods for estimating flood magnitudes, the rainfall-runoff model parameters must be adjusted to achieve an acceptable level of agreement. When making parameter adjustments, the goal is not necessarily to "calibrate" the model to achieve a specific flood magnitude. Rather, the goal is to "pseudo-calibrate" the model to produce acceptable flood magnitudes over a range of flood frequencies.

When pseudo-calibrating a rainfall-runoff model, the engineer must understand the implications associated with parameter adjustments. Specifically, adjustments to the loss parameters (initial abstraction and infiltration) add to, or subtract from, the total runoff volume, while adjustments to the routing parameters (e.g., time-of-concentration, basin storage coefficient) change the shape of the runoff hydrograph without (substantively) affecting runoff volume. A sensitivity analysis will determine which parameters have the greatest and least impacts on the model results, and the final parameter assignments should be maintained within published ranges and/or that reasonably represent the modeled basin's characteristics.

A common objective when pseudo-calibrating a rainfall-runoff model is to determine the combination of basin parameters that produces results that closely approximates both the

100- and 500-year flood magnitudes as computed/estimated through an independent method. When the model is pseudo-calibrated and reasonably predicts these two events (within the values computed using the independent method and reasonable upper bounds, such as one standard deviation or SEP above the values), the model is generally believed to produce a reliable estimate of the IDF peak inflow rate.

As stated previously, the rainfall runoff response during an extreme flood, such as an IDF, is often disproportionately more severe than more frequent events. Because of the nonlinearity of the rainfall-runoff response phenomena, a model that is calibrated to a specific event will not compute the peak runoff rate for a different flood frequency to the same level of accuracy. For this reason, when pseudo-calibrating a rainfall runoff model, the modeler will have to assign parameters that produce "acceptable" results over a range of flood frequencies.

Another aspect of the non-linearity of a basin's response to increasing precipitation depth is the slope of the flood frequency curve decreases with increasing return interval. Because of this, when pseudo-calibrating a rainfall runoff model against a relatively frequent storm (e.g., the 100-year flood), the modeler may apply an initial abstraction or assign a loss rate at the high end of the acceptable range. After the model is pseudo-calibrated for the more frequent storm(s), the initial abstraction can be removed, or the loss rate decreased, for the simulation used to compute the IDF.

10.5 Model Verification and Confidence

Since analysis of gage data and the use of regression equations have associated uncertainty, and "best estimates" for parameters have been incorporated throughout the development of the model, it is necessary that the model be verified to develop a sense of confidence that the magnitude of IDF is consistent with the downstream hazard (i.e., LOL), and that the dam and spillway (either existing or proposed) provide a commensurate level of protection as required by Montana Rule.

The model results are typically considered to be "reasonably conservative" if they are within a confidence band between the calculated mean value and the mean value plus one standard deviation of the streamflow estimates (whether from gauged data or from the USGS regression equations). Additionally, it is important to verify that the model will continue to yield reasonably conservative results when it is used to estimate flows beyond the 500-year flood. This may best be accomplished by plotting the model results for the 100-year and 500-year peak inflows on the flood-frequency graph that contains the historic (or regression) streamflow estimates. The slope of the line between the 100-year and 500-year model result points should be equal to or greater than the slope between the corresponding streamflow estimate points. If the slope is too flat, it is very likely that the model will underestimate peak flows for larger events.

In addition to verification of model results by comparison to independent methods for computing flood frequency, the reasonableness of an IDF can be verified through comparison of the IDF hydrograph peak inflow rate to the appropriate envelope curve of peak discharge vs. drainage area as shown in Figure 4 of USGS Scientific Investigations Report 2015-5019, Montana StreamStats, Chapter F. Figure 4 shows the peak discharge of record at stream gaging sites in each flood region of Montana, together with enveloping curves plotted above the largest discharges. Figure 4 also shows envelope curves for floods throughout the United States and a regional regression line relating the 100-year recurrence interval to drainage area for sites within the region. An IDF with a recurrence interval of 500 years can be expected to plot near the envelope curve for the appropriate region in Montana, but below the envelope curve for the largest floods in the United States. An IDF with a recurrence interval greater than 500 years can be expected to plot above the envelope curve for Montana, but probably not above the envelope curve for the United States. If the IDF plots below the envelope curve(s) for Montana, the modeler should revise the model parameters to increase the peak runoff rate such that the IDF plots above the appropriate envelope curve.

Figure 2 is a graph of the 100-year, 500-year, and IDF peak inflow rates obtained from a rainfall-runoff model for a basin in southwestern Montana. Also shown on the graph are the upper bands for one-half, one, and two standard deviations above the USGS Regional Regression Equation results, and the envelope curve value for the specific basin area size, as well as the peak inflow rate for the IDF computed using three alternate rainfall-runoff model parameter sets. Of interest, while the rainfall-runoff model result for the 500-year peak inflow rate is in very good agreement with the USGS Regional Regression Equation result, the peak inflow rate computed for the IDF (5,000-year flood in this example) using those same basin parameters is less than 80% that of the envelope curve value for the southwest region of Montana. For this application and the floods of records in the southwest region of Montana, it is difficult to accept that the 5,000-year flood would fall below the envelope curve. Similarly, it is also difficult to accept that a dam designed based on this IDF hydrograph provides the **minimum** level of protection required by Montana's

Rules. Consequently, for the example shown, it is necessary to make further adjustments to the basin parameters to produce an IDF that is reasonable (i.e., plots above the envelope curve), refer to Figure 2.

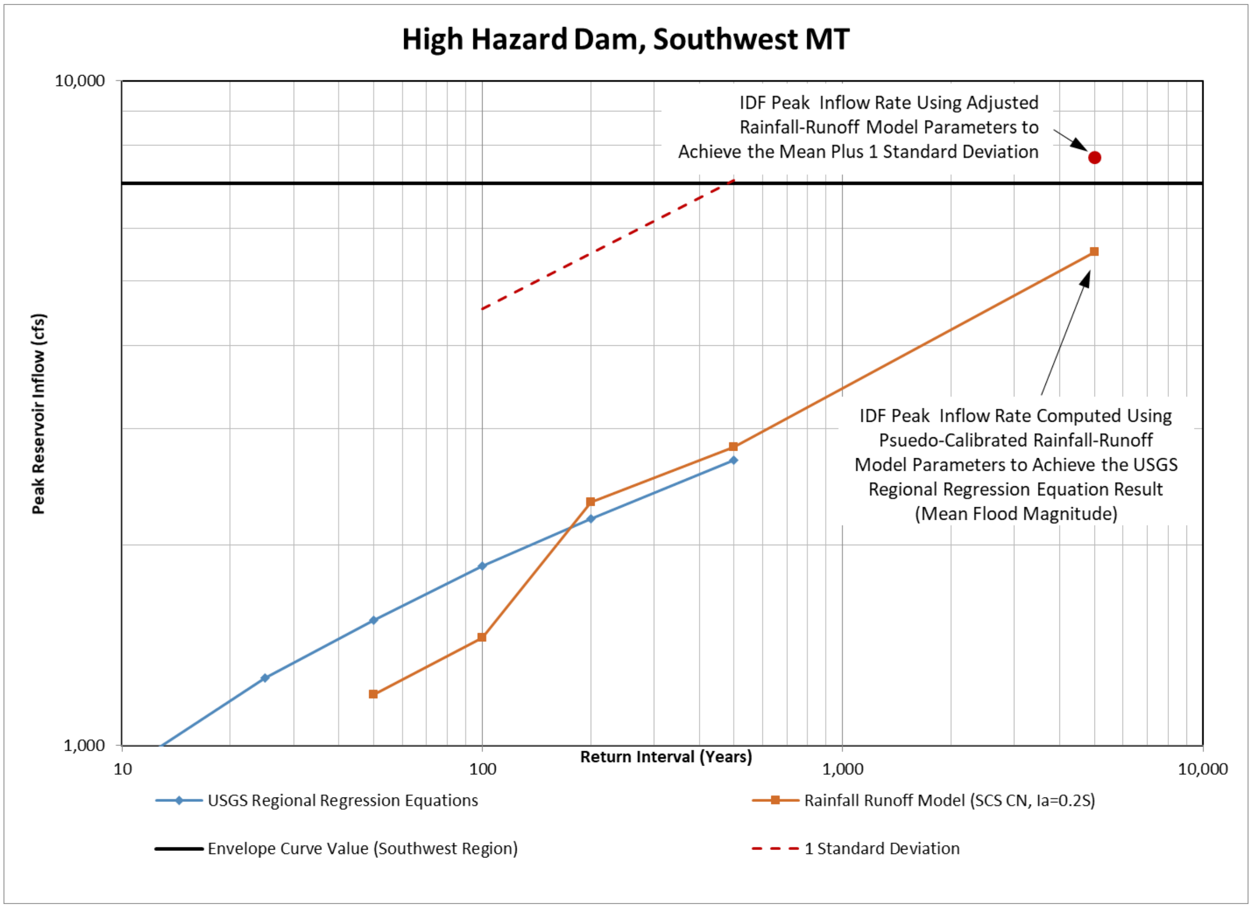

Figure 2: Flood Frequency Curve with Standard Deviation and Envelope Value

10.6 Validation Process Flow diagram

The flow diagram below (Figure 3) presents the verification, pseudo-calibration, and validation process.

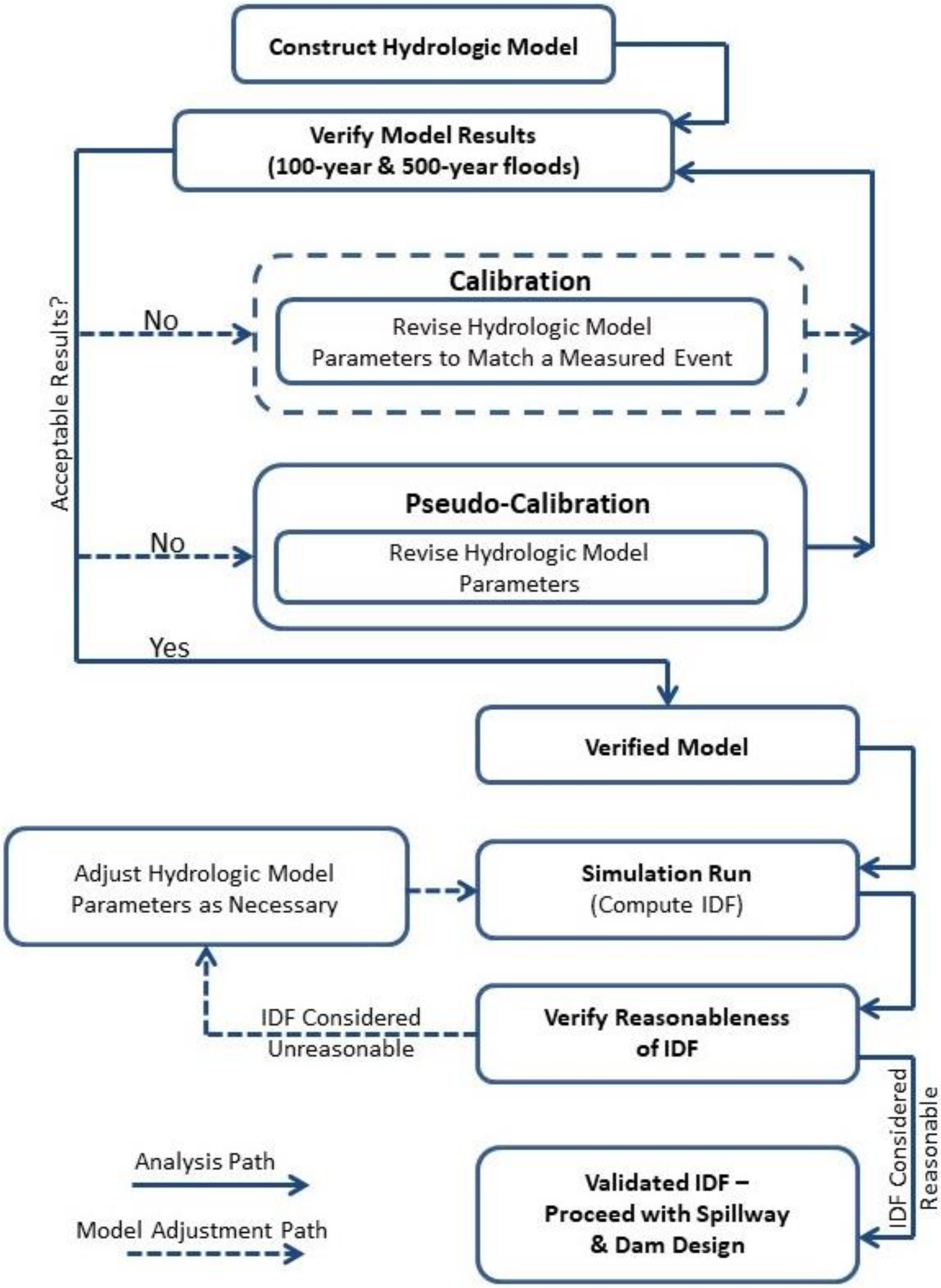

Figure 3: Hydrologic Model Validation & IDF Computation Process

10.7 Verification Methods Summary

Because streamflow records are limited, statistical analyses to determine storm frequency peak flows can only estimate up to approximately 100-year, or possibly 500-year, return interval peaks. While these storm peaks may be relatively small in comparison to spillway design peaks, they still represent large, infrequent storms. The response of a Montana drainage basin to a 100- or 500-year rainfall event may be similar to that of a larger event, especially if soil infiltration rates reach the point of saturation and most of the rainfall becomes direct runoff. It is reasonable, then, to use 100- or 500-year peak stream flows determined using other, independent methods for computing flood frequency to aid in the verification of the reasonableness of hydrologic models for a spillway IDF. Appendix C includes a model validation checklist and below is a list of acceptable independent model verification methods:

- USGS regional regression equations described in USGS Scientific Investigations Report 2015-5019, Montana StreamStats, Chapter F.
- Transferring flood-frequency estimates from a gaging station on the same stream to an ungaged site upstream or downstream using methods described in USGS Scientific Investigations Report 2015-5019, Montana StreamStats, Chapter F.
- Local regression equations for flood frequency developed from surrounding nearby USGS stream gages having similar physiographic characteristics. Because local regression equations are developed using Ordinary Least Squares (OLS) regression methods that do not account for differing record lengths at gaging stations or for the degree of inter-station correlation among gages, USGS regional regression equations developed from Generalized Least Squares (GLS) regression are generally preferred over local regression equations. Nevertheless, it is recognized that local regression equations may provide more reliable estimates for flood frequency than those from USGS regional equations in localized areas where physiographic and runoff characteristics at nearby gages differ from those prevalent in the rest of the region.

10.7.1 Model Parameters

The primary model parameters that may be utilized to pseudo-calibrate the runoff peaks from the model are those associated with the unit hydrograph and basin losses. Although both will affect the runoff peak, changes made to the basin losses will also affect the runoff volume. Generally, it is more conservative to calibrate by adjusting the unit hydrograph since this will only affect the runoff peak without affecting the total runoff volume. If necessary, parameters that affect the total runoff volume may be adjusted, but the modeler must maintain an awareness of the total effective precipitation. The total effective precipitation is an important IDF characteristic when routing the IDF through the reservoir – too little volume will route through the reservoir and too much volume will overtop the dam – the modeler must assign parameters that achieve a total runoff volume that seem appropriate.

Unit Hydrograph Parameters: Tc and R (Clark Unit Hydrograph) and tp and qp (Dimensionless Unit Hydrograph) should be estimated using the equations given in *WSP 2420* (Section 3.2.2.1). These two parameters may be utilized to pseudo-calibrate the model but should remain within one standard deviation of the estimated value.

Basin Infiltration Parameters: The parameters available to pseudo-calibrate the model are the "percent impervious area", "initial abstractions", and the loss rate (typically the curve number or ultimate infiltration rate). Each of these parameters has a range of "reasonable" values that may be estimated utilizing published guidelines (Section 3.3). The model should be run with various combinations of basin infiltration parameters during the process of verification.

10.7.2 Gage Data

If the drainage basin upstream of a reservoir considered for spillway analysis has one or more streamflow gages near the reservoir or nearby in the basin, the recorded data could be used to determine a desired return period peak flow by a direct frequency analysis or analysis by transposition.

Probably the most common distribution used to conduct a frequency analysis of streamflow gage data is Log Pearson Type III. This distribution has been found to fit well with streamflow data and has wide application in hydrologic studies. Use of Log Pearson Type III analysis was popularized with the publication of Bulletin 17B by the Interagency Advisory Committee on Water Data. Methods described in Bulletin 17B have become the standard for conducting streamflow frequency analyses; Bulletin 17B has been superseded by Bulletin 17C. The USACE has issued the program HEC-SSP (Statistical Software Package) that allows users to compute flood frequency curves for gaged sites (including links to directly access USGS stream gage data) in accordance with the methods described in both Bulletins 17B and 17C.

Techniques to transposition flows from an ungaged site (i.e., the location of the reservoir) on a gaged stream are included in USGS Scientific Investigations Report 2015-5019, Montana StreamStats, Chapter F.

10.7.3 StreamStats Program

The StreamStats Program is a Web application that incorporates a Geographic Information System (GIS) to provide users with access to an assortment of analytical tools that are useful for a variety of water-resources planning and management purposes, and for engineering and design purposes. In StreamStats, users can select USGS data-collection station locations shown on a map and obtain previously published information for the stations, including descriptive information, and previously published basin characteristics and streamflow statistics. Currently, StreamStats provides additional tools that allow users to select sites on ungaged streams and do the following:

- obtain the drainage-basin boundary,
- compute selected basin characteristics,
- estimate selected streamflow statistics using regression equations,
- download a shapefile of the drainage-basin boundary, as well as any computed basin characteristics and flow statistics,
- edit the delineated basin boundary,
- modify the basin characteristics that are used as explanatory variables in the regression equations and get new estimates of streamflow statistics,
- print the map,
- measure distances between user-selected points on the map, and
- obtain plots of the elevation profile between user-selected points on the map.

In conjunction with the national StreamStats program, the USGS has issued Montana StreamStats, SIR 2015-5019 (USGS, 2015), which includes Chapter F (Methods for estimating peak-flow frequencies at ungaged sites in Montana based on data through water year 2011) and Chapter G (Methods for estimating streamflow characteristics at ungaged sites in western Montana based on data through water year 2009). Using the methods described in SIR 2015-5019, the user can compute the instantaneous peak flood flow for flood frequencies up to the 500-year (0.2% exceedance probability) flood.

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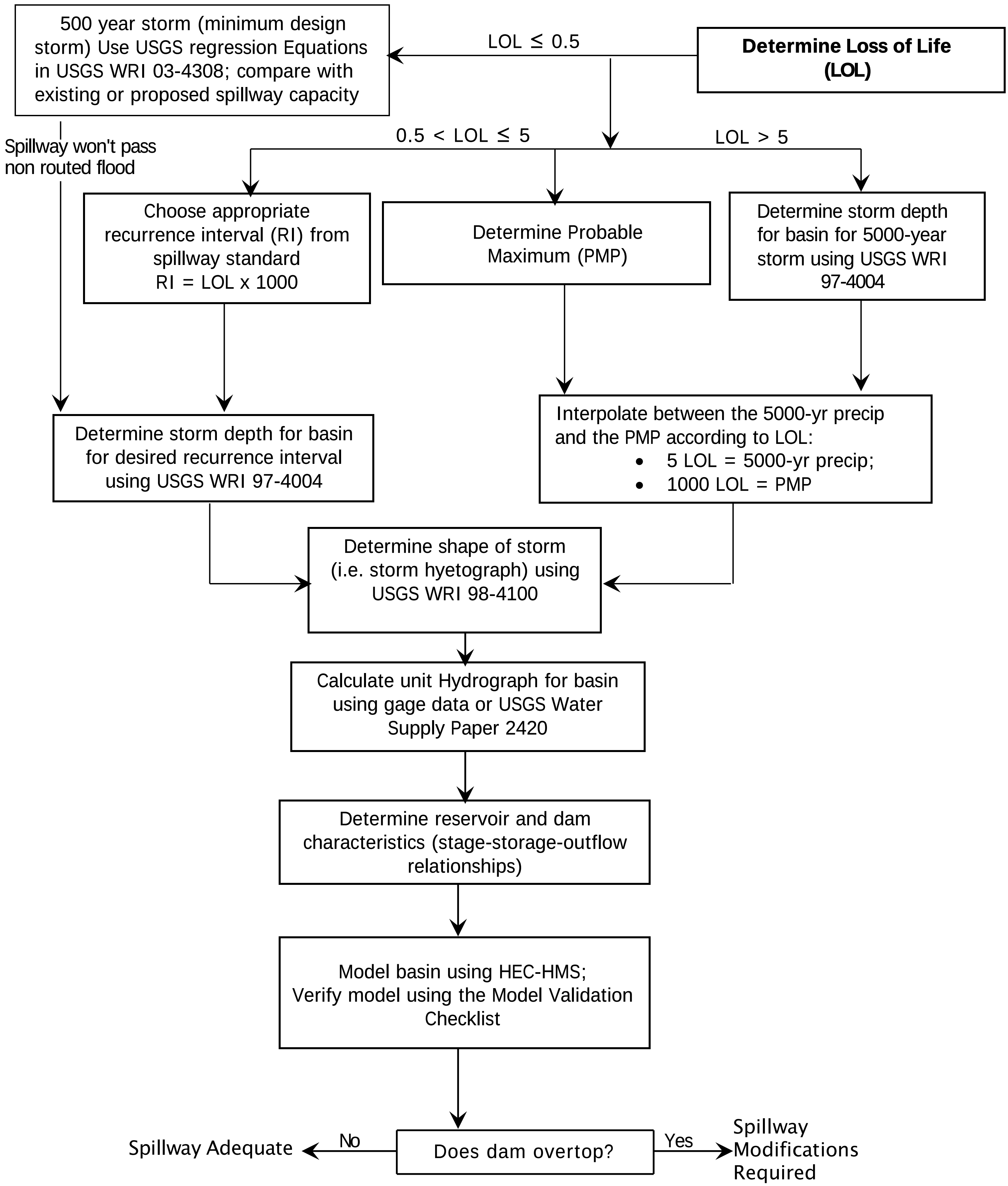
APPENDIX A: FLOWCHART PROCEDURE FOR DETERMINING COMPLIANCE WITH MONTANA SPILLWAY STANDARDS



Procedure for Determining Spillway Adequacy

APPENDIX B: MINIMUM INFLOW DESIGN FLOOD FOR HIGH HAZARD DAMS

MINIMUM INFLOW DESIGN FLOOD FOR HIGH HAZARD DAMS

MONTANA SPILLWAY STANDARDS COMMITTEE

TECHNICAL DECISIONS

May 1999

Introduction

Under current Montana Rule 36.14.502, the Probable Maximum Flood (PMF) or a fraction thereof is often used to calculate the inflow design flood for high hazard dams. It is required that the reservoir and spillway safely store and pass the design flood. Inflow design criteria are based on both reservoir volume and dam height. Some form of the PMF criteria are applied unless a reservoir is less than 100 acre-feet and the dam is less than 20 feet high.

Under the current rule, the required spillway size for existing dams is not related to potential loss of life downstream from the dam. For example, the criteria remains the same regardless if one loss of life or one hundred loss of life would result from spillway failure below a given dam.

A goal of the Montana Department of Natural Resources and Conservation (DNRC), Dam Safety Section, is to implement a hydrologic standard for spillways that is risk based, with dams having a large population below being held to a more stringent criteria than dams having a low population below. Therefore, during the spring of 1997, the Montana Spillway Standards Committee was formed to advise the Dam Safety Section in development of new rules for spillway conveyance requirements. The committee is made up of dam owners, downstream residents of dams, consulting engineers, and government employees. The purpose of this report is to document the recommendations made by this committee.

In support of this effort, the U.S. Geological Survey (USGS), with some financial assistance from DNRC, conducted a study to characterize extreme storms that have a very low probability of occurrence. As a result, we are now able to estimate precipitation depths, throughout Montana, with recurrence intervals as high as 5,000-years. After determination of rainfall depth, the resulting flood is routed through the reservoir to estimate required spillway size.

Discussed in this report are recommendations by the Montana Spillway Standards Committee regarding risk vs. recurrence interval for the design flood and loss of life assessment. This report only addresses spillway requirements and not hydrologic techniques or other structural aspects of dams. Hydrologic techniques used by DNRC to help assess the proposed spillway standards are presented in "Montana Spillway Standards - Hydrologic Analysis", DNRC, May 1999.

Risk vs. Recurrence Interval - Inflow Design Flood

B.C. Hydro has pioneered the development of quantified risk analysis for dam safety evaluations. B.C. Hydro owns and operates 61 dams and is responsible for the production and distribution of electrical energy for the province of British Columbia, Canada. B.C. Hydro has adopted a mathematical annual expected value of loss of life for each dam of 0.001.

The committee recommends that a probability value of .001 (0.1% annually) of loss of one life due to spillway failure is acceptable. The associated recurrence interval for annual probability of .001 is 1,000-year. Therefore, the estimated loss of life due to spillway failure is multiplied by 1,000 to determine the inflow design storm recurrence interval that must be routed in the reservoir and passed by the spillway. For example, if a spillway failure is estimated to result in the loss of 2.4 lives, the spillway and reservoir will need to route the 2,400-year recurrence interval rainfall to be considered safe. If the spillway just meets this criteria, there will be a .042% ($\{1/2400\}$x100) chance each year that the 2.4 lives will be lost, and during a 50 year period there will be a 2% ($1-\{1-.00042\}^{50}$) chance that these lives will be lost.

The committee found that once a loss of life can be estimated from spillway failure, an appropriate inflow design storm with an assigned recurrence interval or probability of occurrence must safely be stored in the reservoir and passed by the spillway.

Committee Suggested Criteria

If the estimated loss of life is greater than zero and less than or equal to 0.5, the committee determined that the spillway and reservoir must be able to store and pass the 500-year runoff event. In other words, the minimum standard inflow design flood for high hazard dams is the 500-year flood. A graphical display of spillway standards for five loss of life or less is shown in Figure 1.

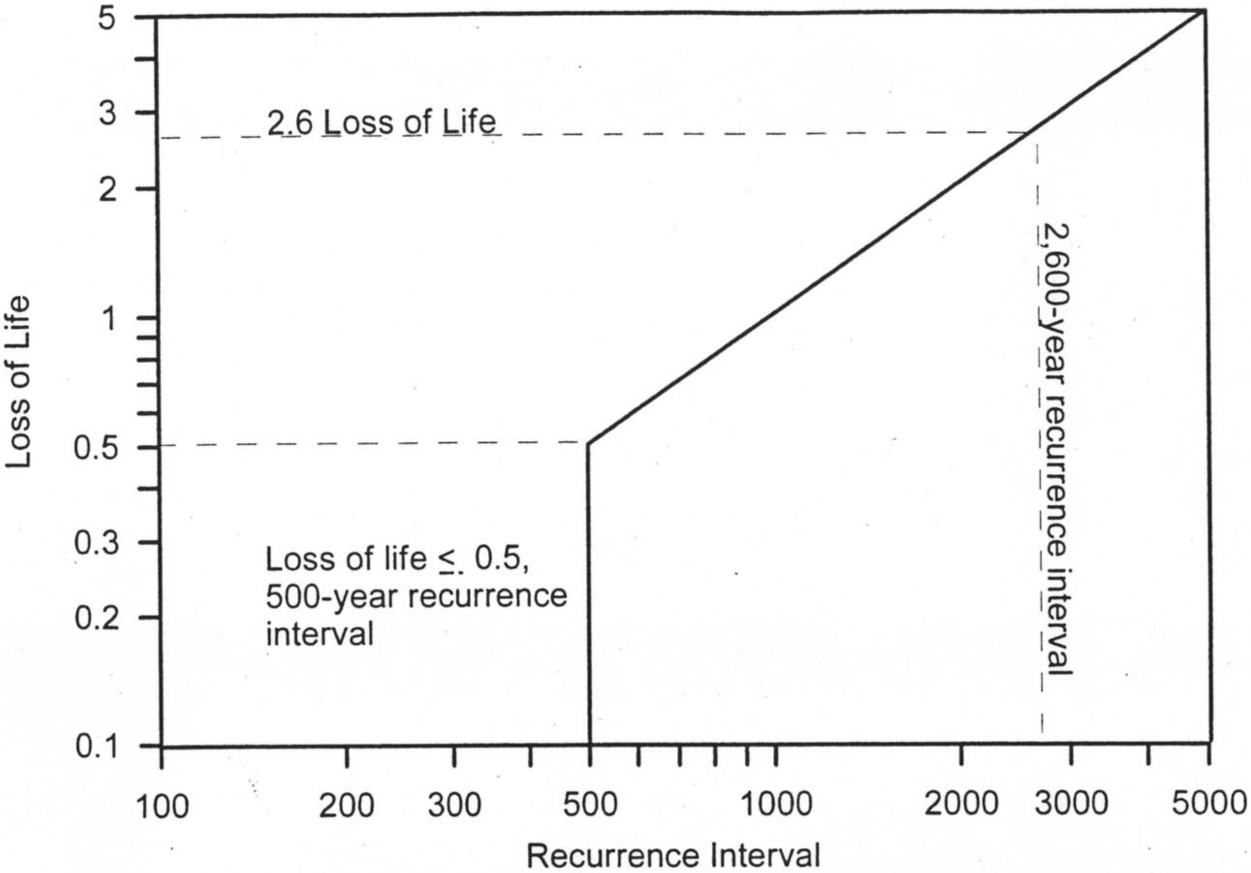


Figure 1. Minimum Standard Inflow Design Flood with an Estimated Loss of Life of Five or Less

If the loss of life is greater than 5, the probability-based approach will no longer be used and the inflow design flood is derived from the precipitation computed using equations 1 through 3. It needs to be emphasized that although no probability is associated with the standard above five loss of life, the standard does become more severe as expected loss of life increases below the dam. The results of the committee's decision on this portion of the standard are shown in Figure 2.

Eq. 1 $P_s = P_{5,000}\,(10^{rd})$

Eq. 2 $r = -0.304 + .435 \log_{10}(lol)$

Eq. 3 $d = \log_{10}(PMP) - \log_{10}(P_{5,000})$

lol = *loss of life* ($5 < lol < 1{,}000$)

PMP = Probable Maximum Precipitation

$P_{5,000}$ = *5,000-year recurrence interval precipitation*

P_s = *Design precipitation to meet spillway standard*

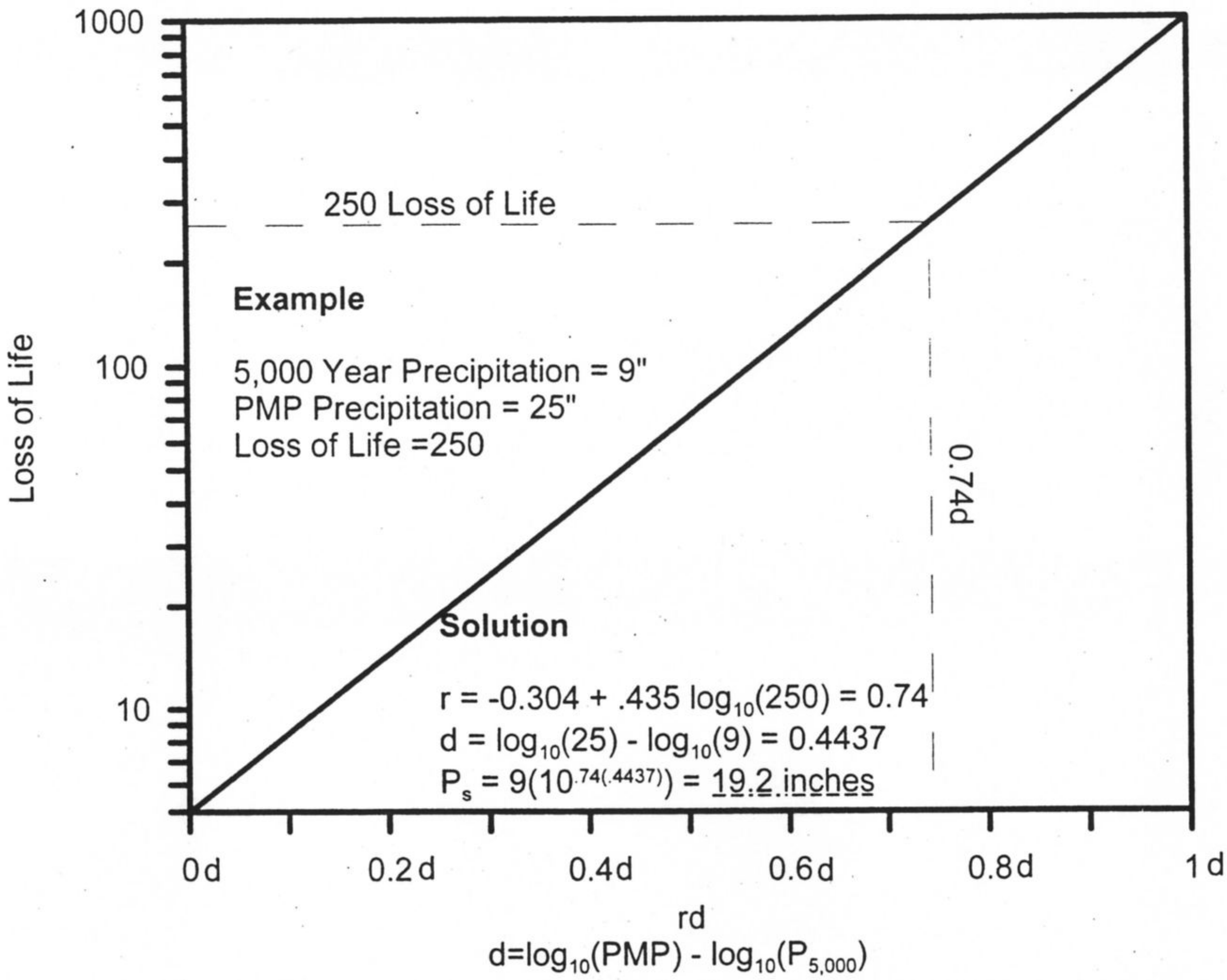


Figure 2. Minimum Standard Inflow Design Flood with an Estimated Loss of Life Between 5 and 1,000

The spillway standard changes for a loss of life greater than 5 not because the overall risk-based approach is no longer acceptable, but there is currently no acceptable way to estimate rainfalls with recurrence intervals greater than 5,000-year events.

For only the purposes of computing the risk factor, which will be discussed shortly, the PMP event is assumed a 1:1,000,000 event for interpolation.

If the loss of life is greater than or equal to 1,000, the reservoir and spillway must safely store and pass the runoff resulting from the PMP storm, often called the probable maximum flood or PMF.

Current design criteria provide for the minimum design storm to be reduced if it can be shown no further loss of life results from spillway failure. This criteria will be kept under the new standard.

Compliance of Existing Dams

For existing dams, the Montana Spillway Standards Committee has decided that it would be unreasonable to expect all existing spillways to come under compliance in a short period of time.

To assist the DNRC Dam Safety Section in prioritizing spillway construction or rehabilitation, the committee suggested using a risk factor developed using both the expected loss of life below the dam and the recurrence interval storm that the existing dam and spillway can safely store and pass. The risk factor is determined using Equation 4.

Eq. 4 $$RF = RI_{SW}/LOL$$

RF = *Risk Factor*

RI_{SW} = *Flood recurrence interval that the spillway can currently pass*

LOL = *Estimated loss of life associated with spillway failure*

If the RF value is 1,000 or above, the existing spillway meets or exceeds the standard and a larger spillway is not required. The committee has recommended for spillways significantly undersized (RF value below 100) that immediate action be taken. These actions may include reservoir level restrictions or breaching. These dams have a more than 1% probability each year of loss of life due to spillway failure.

For RF values between 100 and 500, proof of diligent effort towards correcting the problem will be required for issuance of operation permits.

For RF values between 500 and 1,000, dams will be evaluated on a case-by-case basis. The committee felt that these dams are sufficiently close to meeting the new standard, and the need for repair is not urgent if the spillway is in good condition.

Loss of Life Assessment

The committee examined several methods to estimate loss of life, caused by spillway failure, downstream from a dam. Many methods base the loss of life (LOL) from the population at risk (PAR). They found the best current method to come from "Predicting Loss of Life in Case of Dam Failure and Flash Flood," Michael L. DeKay and Gary H. McClelland, Risk Analysis, Vol. 13, No. 2, 1993. In the future, new and better methods may become available. These methods should then be used.

The DeKay and McClelland method is based on an empirical approach that incorporates warning time, PAR, and the force of the flood. It is suggested by DeKay and McClelland that no one who is more than 3 hours travel time below the dam be included in the PAR. It is also strongly recommended that the PAR not be lumped into smaller reaches and zones of inundations since this leads to overestimates of LOL. The following equations are given by DeKay and McClelland to determine loss of life.

Eq. 5

$$LOL_{HF} = \frac{PAR_{HF}}{1+13.277(PAR_{HF}^{0.440})e^{(2.982(WT_{HF})-3.790)}}$$

Eq. 6

$$LOL_{LF} = \frac{PAR_{LF}}{1+13.277(PAR_{LF}^{0.440})e^{(0.759(WT_{LF}))}}$$

LOL_{HF} = *Loss of Life in High Force Area*

LOL_{LF} = *Loss of Life in Low Force Area*

PAR_{HF} = *Population at Risk in High Force Area*

PAR_{LF} = *Population at Risk in Low Force Area*

WT_{HF} = *Warning time for High Force Area*

WT_{LF} = *Warning time for Low Force Area*

In most cases the committee recommends that 0 warning time be used. This assumption can be modified if sufficient evidence can be shown that a warning time exists.

The force of flood will often be estimated at site. In general, if the flood will occur in a narrow canyon area, the flood will likely be high force. If the area downstream of the dam is flat and wide, it is likely that much of the areas within the inundation zone will be classified low force.

If more specific information is needed regarding the flood force, the document "Downstream Hazard Classification Guidelines," ACER Technical Memorandum No. 11, U.S. Bureau of Reclamation, 1988 may be useful. Shown in Figures 3-5 are plots of velocity vs. depth for houses, mobile homes, and passenger vehicles. The various zones are high danger, low danger, and judgment.

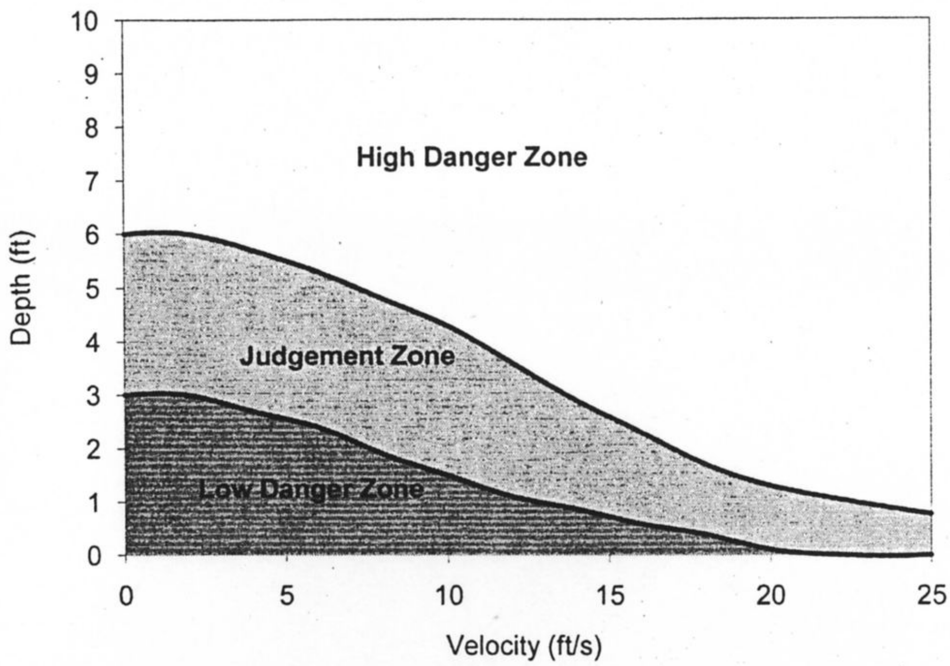


Figure 3. - Depth-Velocity Flood Danger Level Relationships for Houses Built on Foundations

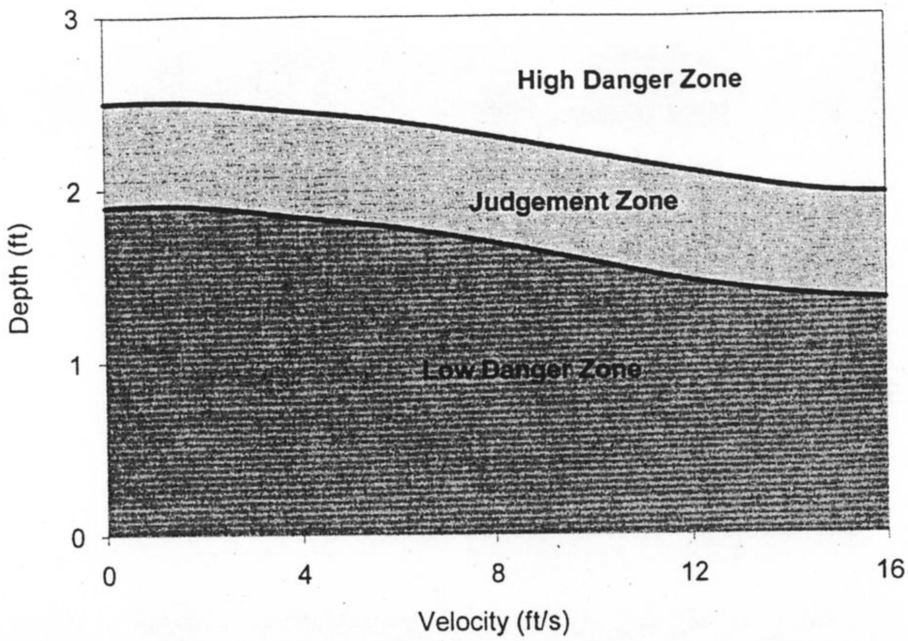


Figure 4. - Depth-Velocity Flood Danger Level Relationship for Mobile Homes

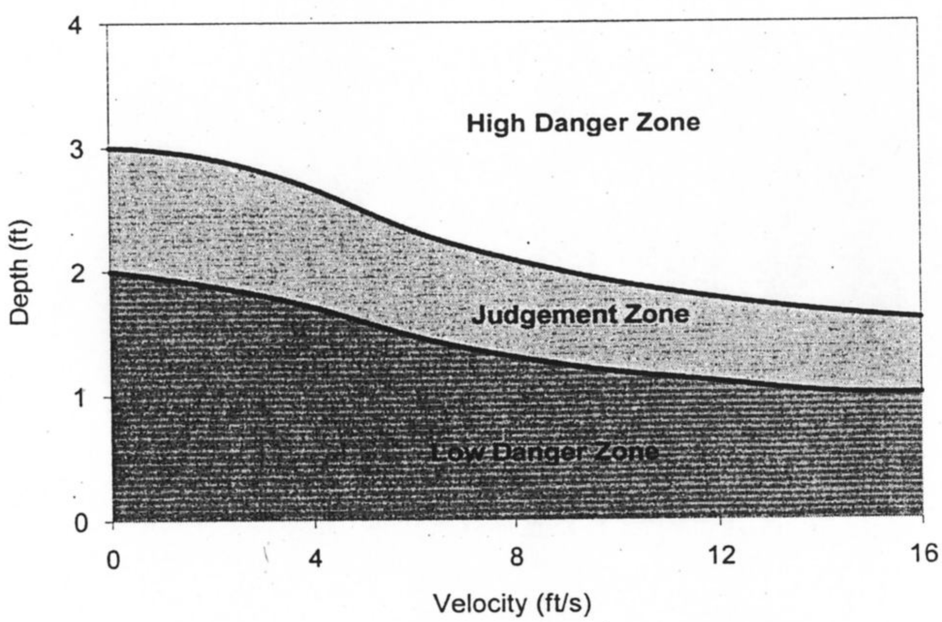


Figure 5. - Depth-Velocity Flood Danger Relationships for Passenger Vehicles

The estimation of PAR in the high force area and the subsequent use of the high force equation has the most significant effect on the LOL. Special attention will be focused on defining the area influenced by the high force wave and the PAR in the area.

Conclusion

The design precipitation used to determine spillway size under the proposed rule is very different from the design precipitation used under the current rule. The proposed rule requires the design precipitation to be based on population level downstream of the dam whereas the current rule requires the design precipitation to be based on the height and volume of water associated with the dam.

Since the population at risk is closely related to whether the force of water is high or low after dam failure, the proposed rule does still indirectly incorporate height of dam within the criteria. The current rule only considers that some population is at risk below the dam. The level of population at risk plays a small to nonexistent role in the spillway conveyance criteria.

For these reasons, dams with large populations at risk downstream, will often have more stringent spillway requirements under the proposed rules. On the other hand, dams with small populations at risk will often have less stringent spillway requirements under the proposed rules. The Montana Spillway Standards Committee has decided that the risk of life downstream should be the overriding factor in determining spillway capacity.

APPENDIX C: MODEL VALIDATION CHECKLIST

Model Validation Checklist

✓#1 Actual Event

Compare modeled results with measured or estimated flows for an actual storm event at nearby streamflow or crest stage gages with similar basin characteristics, using measured precipitation at applicable rain gages.

- If rain gage data are not available, storm characteristics may be estimated using NEXRAD data available through NOAA, or on the PRISM Climate Group website.
- If nearby streamflow gages do not have similar characteristics, or gage data are not available, request USGS to conduct an indirect measurement to estimate the maximum streamflow during the event
- Interview local eyewitnesses for information on measured flows, depth of water in spillways, starting water surface elevations peak of storm, lag following rainfall, etc.
- Consider antecedent conditions preceding storm – include this in model. A series of storms commonly precede a large event.
- Part of the process is to understand the most significant gage parameters that are being used in the validation or calibration (i.e. rainfall or snowmelt).

✓#2 Flood Frequency Estimates on Nearby Streamflow gages

Compare modeled results with 25-, 100-, and 500-year flood frequency estimates for nearby streamflow gages that have similar basin characteristics (Table 1-6 in USGS Scientific Investigations Report 2015-5019, Montana StreamStats, Chapter F).

✓#3 USGS Regional Regression Equations

Compare the rainfall-runoff model results with results from the USGS regional regression equations. USGS regional regression equations may also be applied at the sub-basin level. Use the USGS regional regression equations described in USGS Scientific Investigations Report 2015-5019, Montana StreamStats, Chapter C.

- Applicability of the regression equations in a region of interest must be evaluated first by comparing flood frequency estimates on nearby streamflow gages that were used in the equations with regional regression equation estimates for those gages. If significant deviation is present, regional regression equations should not be relied upon.
- Determine the size of drainage basins used in regional regression equations. If most basins are larger than the basin of interest, then the results will be biased towards bigger basins. Big drainages tend to attenuate more, and therefore the regional regression equations may underestimate flows and, hence, not be applicable to the study basin.
- Verify that regional basin parameters are within the range used to develop the equations

✓#4 Local Regression Equations

Compare the rainfall-runoff model results with results from local regression equations for flood frequency developed from surrounding nearby USGS stream gages having similar physiographic characteristics.

- Identify nearby gages with similar basin characteristics (Table 1-1in USGS SIR 2015-5019-F)
- Using MDT Dept. of Transportation (MDT) local regression equation spreadsheet (unpublished, available from MT DNRC Dam Safety with permission from MDT) and basin characteristics of nearby gages (i.e., area, elevation above 5000 feet, etc.), develop local regression equations for 100- and 500-year floods
- Using these regression equations, calculate 100 year and 500 year floods for basin of interest
- Compare with local regression results for flood frequency with the rainfall-runoff model results. If necessary, psuedo-calibrate the rainfall-runoff model basin parameters such that the rainfall-runoff model matches results from the local regression equations

✓#5. Conduct a Sensitivity Analysis / Document Awareness of Uncertainty

- Evaluate model sensitivity to subbasin definition, initial abstraction, loss method , unit hydrograph, and routing methods
- Document uncertainty by plotting the flood frequency curve with standard deviation or error bounds

✓#6 Reality Check / Reasonableness Test

- Compare the sum of individual modeled sub basin outflows to the modeled flow at lowest point of interest. Is attenuation occurring in a realistic manner?
- Determine velocities in the model reaches. Is the model matching what you would physically expect to find in the field?
- Look at drainage basin land cover when estimating initial abstraction. Examine photos – do the loss parameters assumed look right? More information on area soil characteristics can be found on the Natural Resources Conservation Service Web Soil Survey App.
- Do the flows look reasonable for the type of channel?
- Are Mannings n's in line with what would be seen during a large, out-of-bank flood event?
- If there are potholes, wetlands, or other depressions in the drainage area, it may be necessary to use the modified Puls method for routing.
- If baseflow is used, is it representative of what is normally in the stream during the time of year a large runoff event is most likely to occur? Note that a quick approximation is to use the 2 year flow.
- Is baseflow too large – is it impacting storage in the reservoir(s) and skewing results?

✓ #7. Envelope Curves

Calculate unit peak discharge (cfs/sq mi) and compare the IDF hydrograph peak inflow rate to the appropriate envelope curve of peak discharge vs. drainage area as shown in Figure 4 of USGS Scientific Investigations Report 2015-5019, Montana StreamStats, Chapter F.