CHAPTER II

SELECTING AND ACCOMMODATING INFLOW DESIGN FLOODS FOR DAMS

Table of Contents

CHAPTER II

SELECTING AND ACCOMODATING INFLOW DESIGN FLOODS FOR DAMS

2-1 Purpose and Scope

The purpose of this chapter of the Guidelines is to provide technical guidance for determining the appropriate Inflow Design Flood (IDF) to be used in the review of spillway and appurtenant structure designs and to conform to the provisions of the Federal Guidelines for Dam Safety.

This chapter is not intended to provide a complete manual of all procedures used for estimating inflow design floods for spillways, because the selection of procedures is dependent upon available hydrologic data and individual watershed characteristics. All studies submitted to the Commission should be performed by a competent engineer experienced in hydrology and hydraulics, and should contain a summary of the design assumptions, design analyses, and methodologies used to evaluate the inflow design flood.

2-2 Definition of Terms

This section contains definitions of some specialized technical terms used in this chapter:

Flood Routing - A process for determining progressively over time the amplitude of a flood wave as it moves past a dam and continues downstream to successive points along a river or stream.

Freeboard - Vertical distance between a specified stillwater reservoir surface elevation and the top of the dam, without camber.

Hazard Potential - The hazard potential of dams describes the potential for adverse incremental consequences in event of failure or mis-operation. Hazard classification does not indicate the structural integrity of the dam itself, but rather the effects if a failure should occur. The hazard potential assigned to a dam is based on consideration of the effects of a failure during both normal and flood flow conditions.

Hydrograph - A graphical representation of the streamflow stage or discharge as a function of time at a particular point on a watercourse.

Inflow Design Flood (IDF) - The flood flow above which the incremental increase in water surface elevation due to failure of a dam or other water impounding structure is no longer considered to present an unacceptable threat to downstream life or property. The IDF of a dam or other water impounding structure flood hydrograph is used in the design of a dam and its appurtenant works particularly for sizing the spillway and outlet works, and for determining maximum height of a dam, freeboard, and temporary storage requirements.

Maximum Wind - The most severe wind for generating waves that is reasonably possible at a particular reservoir. The determination will generally include results of meteorologic studies which combine wind velocity, duration, direction, and seasonable distribution characteristics in a realistic manner.

One Percent Flood - A flood that has 1% probability of being equaled or exceeded in a given year. This is often referred to as a 100 year flood.

Outlet Works - A dam appurtenance that provides release of water (generally controlled) from a reservoir.

Probable Maximum Flood (PMF) - The flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the drainage basin under study. This is the upper limit for determining the IDF.

Probable Maximum Precipitation (PMP) - Theoretically, the greatest depth of precipitation for a given duration that is physically possible over a given size storm area at a particular geographical location during a certain time of the year.

Reservoir Regulation Procedure (Rule Curve) - Compilation of operating procedures that govern reservoir storage and releases.

Spillway - A gated or ungated hydraulic structure used to discharge water from a reservoir. Definitions of specific types of spillways follow:

- **Service Spillway**. A spillway that is designed to provide continuous or frequent regulated or unregulated releases from a reservoir without significant damage to either the dam or its appurtenant structures.
- **Auxiliary Spillway**. Any secondary spillway which is designed to be operated very infrequently; possibly, in anticipation of some degree of structural damage or erosion to the spillway would occur during operation.

- **Emergency Spillway**. A spillway that is designed to provide additional protection against overtopping of dams and is intended for use under extreme flood conditions or mis-operation or malfunction of the service spillway.

- **Spillway Capacity -** The maximum outflow flood which a dam can safely pass.

Stillwater Level - The elevation that a water surface would assume if all wave action were absent.

Wave Runup - Vertical height above the stillwater level to which water from a specific wave will run up the face of a structure or embankment.

Wind Setup - The vertical rise of the stillwater level at the face of a structure or embankment caused by wind stresses on the surface of the water.

2-3 Determination of the Inflow Design Flood

The Commission's Order No. 122, issued January 21, 1981, states that the adequacy of a spillway must be evaluated by considering the hazard potential which would result from failure of the project works during flood flows. If failure of the project works would present a threat to human life or would cause significant property damage, the project works must be designed to either withstand overtopping or the loading condition that would occur during a flood up to the probable maximum flood, or to the point where a failure would no longer constitute a hazard to downstream life and/or property. In the alternative, the capacity of the spillway must be adequate to prevent the reservoir from rising to an elevation that would endanger the safety of the project works.

The **Inflow Design Flood (IDF)** is the flood flow above which the incremental increase in water surface elevation due to failure of a dam or other water impounding structure is no longer considered to present an unacceptable threat to downstream life and property.

The procedures used to determine whether or not the failure of a project would constitute a threat to human life or could cause significant property damage vary with the physical characteristics and location of the project.

Analyses of dam failures are complex with many historical dam failures not completely understood. The principal uncertainties in determining outflow from a dam failure involve the mode and degree of failure. These uncertainties can be circumvented in situations where it can be shown that the complete and sudden removal of the dam would not endanger human life or cause extensive property damage. Otherwise, reasonable failure postulations and sensitivity analyses such as those suggested in Appendix II-A should be used. Although a study using the breach parameters suggested in Appendix II-A of this chapter may indicate that a hazard does not exist, a hazard could exist for a more extensive mode of failure. If it is judged that a more extensive mode of failure is possible, then an analysis should be done to determine whether remedial action is required. The possibility of more extensive modes of failure should particularly be considered when failure is due to overtopping.

2-3.1 Hazard Evaluation

A properly designed, constructed, and operated dam can be expected to improve the safety of downstream developments during floods. However, the impoundment of water by a dam can create a potential hazard to downstream developments greater than that which would exist without the dam because of the potential for dam failure. There are several potential causes of dam failure, including hydrologic, geologic, seismic, and structural. This chapter of the Guidelines is limited to the selection of the IDF for the hydrologic design of a dam to reduce the likelihood of failure from a flood occurrence to an acceptable level.

2-3.1.1 General

Once a dam is constructed, the downstream hydrologic regime may change, particularly during flood events. The change in hydrologic regime could alter land use patterns to encroach on a flood plain that would otherwise not be developed without the dam. Consequently, **evaluation of the consequences of dam failure must be based on the dam being in place, and must compare the impacts of with-failure and without-failure conditions on existing development and known future development. Comparisons between existing downstream conditions with and without the dam are not relevant.**

2-3.1.2 Defining the Hazard Potential

The hazard potential of a dam pertains to the potential for loss of human life or property damage in the area downstream of the dam in the event of failure or incorrect operation of a dam. Hazard potential does not refer to the structural integrity of the dam itself, but rather the effects if a failure should occur.

The hazard potential classification assigned to a dam (see Chapter 1 of these Guidelines) should be based on the worst-case failure condition. That is, the classification is based on failure consequences resulting from the failure condition that will result in the greatest potential for loss of life and property damage. For example, a failure during normal operating conditions may result in the released water being confined to the river channel, indicating a low hazard potential. However, if the dam was to fail during a flood flow condition, and the result would be a probable loss of life, the dam would have high hazard potential classification.

In many cases, the hazard potential classification can be determined by field investigations and a review of available data, including aerial photos and topographic maps. However, when the hazard potential classification is not apparent from a field reconnaissance, detailed studies, including dam break analyses, are required for various flood flow conditions to evaluate the incremental effects of a failure of a dam in order to identify the flood level above which the consequences of failure become acceptable--that is, the flood flow condition above which the additional incremental increase in elevation due to failure of a dam is no longer considered to present an unacceptable threat to downstream life and property.

The selection of the appropriate IDF for a dam is related to the hazard classification for the dam. The IDF for a dam having a low hazard potential is selected primarily to protect against loss of the dam and its benefits should a failure occur. The IDF for high and significant hazard potential dams is the maximum flood above which there are no significant incremental impacts on downstream life and/or property.

2-3.1.3 Evaluating the Consequences of Dam Failure

The possible consequences resulting from a dam failure include loss of human life; economic, social, and environmental impacts; damage to national security installations; and political and legal ramifications. Estimates of the potential for loss of human life and the economic impacts of damage resulting from dam failure are the usual bases for defining hazard potential. Social and environmental impacts, damage to national security installations, and political and legal ramifications are not easily evaluated, and are more susceptible to subjective or qualitative evaluation. Therefore, these other considerations do not usually affect decisions on hazard potential. Because their actual impacts cannot be clearly defined, particularly in economic terms, their consideration as factors for determining the hazard potential rating must be on a case-by-case basis, as determined by the Regional Engineer in consultation with the Director or Deputy Director, D2SI.

The following factors should be evaluated regarding potential for loss of human life when estimating the potential for fatalities resulting from dam failure:

- The number and location of habitable structures within the potential area inundated by dam failure. The presence of public facilities within the potential area inundated by dam failure that would attract people on a temporary basis (e.g., improved campgrounds, organized or unorganized recreation areas, State or national parks, etc.) requires special consideration.

- Type of flow conditions based on water depths, temperatures and velocities, rate of rise of the flood wave, duration of flood flow, and special hazardous conditions such as the presence of surface waves, debris flow or terrain conditions which may increase potential for loss of lives.

The evaluation of the economic impacts of failure should consider damages to residences; commercial property; industrial property; public utilities and facilities including transmission lines and substations; transportation systems; agricultural buildings, lands, and equipment; dams; and loss of production and other benefits from project operation.

In summary, in most situations the investigation of the impacts of failure on downstream life and property is sufficient in itself to determine the appropriate hazard potential rating and to select the appropriate IDF for a project. However, in determining the appropriate IDF for a project, there could be circumstances beyond loss of life and property damage, which would dictate using a more conservative hazard potential rating and IDF. For example, the reservoir of a dam that would normally be considered to have a low hazard potential based on insignificant incremental increases (in elevation) due to a failure may be known to contain extensive toxic sediments. If released, those toxic sediments would be detrimental to the eco-system. Therefore, a low hazard potential rating would not be appropriate. Instead, a higher standard should be used for selecting the hazard potential rating and IDF.

2-3.1.4 Studies to Define the Consequences of Dam Failure

The degree of study required to sufficiently define the impacts of dam failure for selecting an appropriate IDF will vary with the extent of existing and potential downstream development, the size of reservoir (depth and storage volume), and type of dam. Evaluation of the river reach and areas impacted by a dam failure should proceed only until sufficient information is generated to reach a sound decision or there is a good understanding of the consequences of failure. In some cases, it may be apparent, from a field inspection or a review of aerial photographs, Flood Insurance Rating Maps, and recent topographic maps, that loss of life and extensive economic impacts attributable to dam failure would occur and be unacceptable. In other cases, detailed studies including dam break analyses will be required. It may also be necessary to perform field surveys to determine the basement and first floor elevations of potentially affected habitable structures (residential, commercial, etc.).

When conducting dam break studies, the consequences of the incremental increase due to failure under both normal (full reservoir with normal stream flow conditions prevailing) and flood flow conditions up to the point where a dam failure would no longer significantly increase the threat to life or property should be considered. For each flood condition, water surface elevations with and without dam failure, flood wave travel times, local velocity and rates of rise should be determined. This evaluation is known as an incremental hazard evaluation. Since dam break analyses and flood routing studies do not provide precise results, evaluation of the consequences of failure should be reasonably conservative.

The upper limit of flood magnitude to be considered in an IDF evaluation is the Probable Maximum Flood (PMF) (see Chapter VIII of these Guidelines). The lower limit for the IDF is typically the One Percent (100-year) flood as determined from historic river flow data or other accepted methods. However, smaller flood events may need to be analyzed for a hazard classification study.

The type of dam and the mechanism that could cause failure require careful consideration if a realistic breach is to be assumed. Special consideration should be given to the following factors:

- Size and shape of the breach,
- Time of breach formation,

- Hydraulic head, and
- Storage in the reservoir.
- Reservoir inflow

In addition, special cases where a dam failure could cause domino-like failure of downstream dams resulting in a cumulative flood wave large enough to cause a threat should be considered.

The area affected by dam failure is the additional area inundated by the incremental increase in flood levels over that which would occur by natural flooding with the dam in place. The area affected by a flood wave resulting from a theoretical dam breach is a function of the height of the flood wave and the length and width of the river at a particular location. An associated and important factor is the flood wave travel time. These elements are primarily a function of the rate and extent of dam failure, but also are functions of channel and floodplain geometry and roughness and channel slope.

The flood wave should be routed downstream to the point where the incremental effect of a failure will no longer constitute a threat to life or property. When routing a dam break flood through the downstream reaches, appropriate local inflows should be considered in the computations. Downstream concurrent inflows can be determined using one of the following approaches:

- Concurrent inflows can be based on historical records, if these records indicate that the tributaries contributing to the reservoir volume are characteristically in flood stage at the same time that flood inflows to the reservoir occur. Concurrent inflows based on historical records should be adjusted so they are compatible with the magnitude of the flood inflow computed for the dam under study.

- Concurrent inflows can be developed from flood studies for downstream reaches when they are available. However, if these concurrent floods represent inflows to a downstream reservoir, suitable adjustments must be made to properly distribute flows among the tributaries.

- Concurrent inflows may be assumed equal to the mean annual flood (approximately bankfull capacity) for the channel and tributaries downstream from the dam. The mean annual flood can be determined from flood flow frequency studies. As the distance downstream from the dam increases, engineering judgment may be required to adjust the concurrent inflows selected.

In general, the study should be terminated when the potential for loss of life and significant property damage caused by routing flood flows appears limited. This point could occur when:

- There are no habitable structures, and anticipated future development in the floodplain is limited,
- Flood flows are contained within a large downstream reservoir,

- Flood flows are confined within the downstream channel, or
- Flood flows enter a bay or ocean.

The failure of a dam during a particular flood may increase the area flooded and also alter the flow velocity and depth of flow as well as the rate of rise of flood flows. These changes in flood flows could also affect the amount of damage. To fully evaluate the hazard created by a dam, a range of flood magnitudes needs to be examined. Water surface profiles, flood wave travel times, and rates of rise should be determined for each condition.

The results of the downstream routing should be clearly shown on inundation maps with the breach wave travel time indicated at critical downstream locations. The inundation maps should be developed at a scale sufficient to identify downstream habitable structures within the impacted area. Guidance on inundation map requirement appears in Chapter VI of these Guidelines.

Dam break studies should be performed in accordance with one or more of the techniques presented in Appendix II-A and Chapter VI of these Guidelines.

The most widely used and recommended model for dam break analysis is the one-dimensional Corps of Engineers Hydrologic Engineering Center's River Analysis System (HEC-RAS) model. Older existing models that are no longer supported by the developer, such as National Weather Service (NWS) DAMBRK and FLDWAV, are still acceptable. However, they may not be used as part of a new or revised submission.

Other one-dimensional and two-dimensional models may be used if pre-agreed to by the Commission. Two-dimensional unsteady flow models, however, must be able to properly route flows through control structures and river-crossings where such obstructions to flow exist.

Most of the methods used for estimating dam break hydrographs, including the widely used HEC-RAS model, require selecting the size, shape, and time of formation of the dam breach as input parameters for the computations. Therefore, **sensitivity analyses are considered necessary**. Sensitivity analyses, based on varying flood inflow conditions and breach parameters, should be performed only to the extent necessary to make a decision.

2-3.1.5 Incremental Hazard Evaluation for Inflow Design Flood Determination

The IDF is determined through an iterative process known as an incremental hazard evaluation. In other words, to evaluate the incremental increase in consequences due to dam failure, the evaluation begins with the normal full reservoir level with normal stream flow conditions prevailing. That condition should be routed through the dam and downstream areas, with the assumption that the dam remains in place. The same flow should then be routed through the dam with the assumption that the dam fails.

The incremental increase in downstream water surface elevation between the with-failure and without-failure conditions should then be determined (in other words, how much higher would the water downstream be if the dam failed than if the dam did not fail?). The amount of damage that could result should then be identified. If the incremental rise in flood water downstream indicates an additional threat to downstream life and/or property, assess the need for remedial action.

If the study under normal flow conditions indicates no adverse consequences, the same analyses should be done for several larger flood flows to determine the greatest unacceptable threat to downstream life and/or property. Under each incrementally larger inflow condition, identify the consequences of failure. For each larger assumed flood inflow condition (which can be percentages of the PMF):

- assume the dam remains in place during the non-failure conditions, and
- assume the dam fails when the peak reservoir elevation is attained for the assumed inflow condition.

It is not appropriate to assume that a dam fails on the rising limb of the inflow hydrograph. For example, current methods available cannot accurately determine the extent of overtopping that an earth dam can withstand or how rapidly the dam will erode and ultimately breach from overtopping. Therefore, until such methodologies are available and proven, a conservative approach should be followed that assumes that failure occurs at the peak of the flood hydrograph. The assumption should also be made that the dam has been theoretically modified to contain or safely pass all lower inflow floods. This is an appropriate assumption since this procedure requires that the dam break analyses start at the normal operating condition, with incremental increases in the flood inflow condition for each subsequent failure scenario up to the point where a failure no longer constitutes a threat to downstream life and property. In summary, before one selects larger floods for analysis, you should determine that failure at a lower flood constituted a threat to downstream life and property.

The above procedure should be repeated until the flood inflow condition is identified such that a failure at that flow or larger flows (up to the PMF) will no longer result in an additional hazard to downstream life and/or property. The resultant flood flow is the IDF for the project. The maximum IDF is always the PMF, but in many cases the IDF will be substantially less than the PMF.

It is important to investigate the full range of flood flow conditions to verify that a failure under flood flows larger than the selected IDF up through the PMF will not result in any additional hazard. In addition, once the design for remedial repairs is selected, the IDF should be verified for that design.

Appendix II-C provides specific guidance and procedures, including a comprehensive flowchart, for conducting an incremental hazard evaluation to select the appropriate IDF for a dam and determine the need for remedial measures.

2-3.1.6 Criteria for Selecting the Inflow Design Flood

The selection of the appropriate IDF for a dam is related to the hazard potential classification and is the result of the incremental hazard evaluation.

There is not a separate IDF for each different section of a dam. A dam is assigned only <u>one</u> IDF, and it is determined based on the consequences of failure of the section of the dam that creates the worst hazard potential downstream. This should not, however, be confused with the design criteria for different sections of a dam that may be based on the effect of their failure on downstream areas.

The criteria for selecting an IDF for the design of a dam requires consideration of the consequences of dam failure under both normal and flood flow conditions.

The PMF should be adopted as the IDF in those situations where consequences attributable to dam failure for flood conditions less than the PMF are unacceptable. The determination of unacceptability clearly exists when the area affected is evaluated and indicates there is a potential for loss of human life and/or extensive property damage.

A flood less than the PMF may be adopted as the IDF in those situations where the consequences of dam failure at flood flows larger than the selected IDF are acceptable. In other words, where detailed studies conclude that the risk is only to the dam owners' facilities and no increased damage to downstream areas is created by failure, a risk-based approach is acceptable. Generally, acceptable consequences exist when evaluation of the area affected indicates:

- There are no permanent human habitations, or known national security installations, commercial or industrial development, nor are such habitations, or commercial or industrial developments projected to occur within the potential hazard area in the foreseeable future.

- There are permanent human habitations within the potential hazard area that would be affected by failure of the dam, but **there would be no significant incremental increase in the hazard to life and/or property resulting from the occurrence of a failure during floods larger than the proposed IDF.** For example, if an impoundment has a small storage volume and failure would not add appreciably to the volume of the outflow flood hydrograph, it is likely that downstream inundation would be essentially the same with or without failure of the dam.

The consequences of dam failure may not be acceptable if the hazard potential to these habitations is increased appreciably by the failure flood wave or level of inundation. **When a dam break analysis shows downstream incremental effects of approximately two feet or more in an inhabited area, engineering judgment and further analysis may be necessary to evaluate the need for modification to the dam. In general, the consequences of failure are not expected to cause a probable loss of**

life when the incremental effects on downstream structures are approximately two feet or less. However, the two-foot increment is not an absolute decision-making point. Sensitivity analyses of the inflow conditions and breach parameters, and engineering judgment are the tools used in making final decisions. For example, if it is determined that a mobile home sitting on blocks can be moved and displaced by as little as six inches of water, then the acceptable incremental impact would be much less than two feet. As a second example, if a sensitivity analysis demonstrates that the largest breach width recommended by this chapter is the only condition that results in an incremental rise of two feet, then engineering judgment becomes necessary to determine whether a smaller breach having acceptable consequences of failure is more realistic for the given conditions (e.g. flow conditions, characteristics of dam, velocity in vicinity of structures, location and type of structures).

In addition, selection of the appropriate magnitude of the IDF may include consideration of whether a dam provides vital community services such as municipal water supply or energy. Therefore, a higher degree of protection may be required against failure to ensure those services are continued during and following extreme flood conditions when alternate services are unavailable. If the economic risk of losing such services is acceptable, the IDF can be less conservative. However, loss of water supply for domestic purposes may not be an acceptable public health risk.

2-3.2 Probable Maximum Floods for Dam Safety

The PMF is the upper limit of floods to be considered when selecting the appropriate IDF for a dam.

2-3.2.1 General

A deterministic approach should be used to determine the PMF. In the deterministic approach, a flood hydrograph is generated by modeling the physical atmospheric and drainage basin hydrologic and hydraulic processes. The approach attempts to represent the most severe combination of meteorologic and hydrologic conditions considered reasonably possible for a given drainage basin. The PMF represents an estimate of the upper limit of run-off that is capable of being produced on the watershed. **Chapter VIII of these Guidelines provides criteria for determining the PMF.**

2-3.2.2 Probable Maximum Precipitation (PMP)

The concept that the PMP represents an upper limit to the level of precipitation the atmosphere can produce has been stated in many hydrometeorological documents. The commonly used approach in deterministic PMP development for non-orographic regions is to determine the limiting surface dew point temperature (used to obtain the moisture maximization factor) and collect a "sufficient" sample of extreme storms. The latter is done through a method known as storm transposition, i.e., the adjustment of moisture observed in a storm at its actual site of occurrence to the corresponding moisture level at the site from which the PMP is to be determined. Storm transposition is based on the concept that all storms within a meteorologically homogeneous region could occur at any other location within that region with appropriate adjustments for effects of elevation and moisture supply. The maximized transposed storm values are then enveloped both depth-durationally and depth-areally to obtain PMP estimates for a specific basin. Several durations of PMP should be considered to ensure the most appropriate duration is selected.

In orographic regions, where local influences affect the delineation of meteorological homogeneity, transposition is generally not permitted. Alternative procedures are offered for these regions that are less reliant on the adequacy of the storm sample. Most of these procedures involve development of both non-orographic and orographic components (sometimes an orographic intensification factor is used) of PMP. Orographic and non-orographic PMP's are then combined to obtain total PMP estimates for an orographic basin.

To date, no single orographic procedure has been developed that offers universal applicability. These techniques have been discussed at length in various National Weather Service (NWS) reports and in the Manual for Estimation of PMP (WMO, 1986). Currently, PMP estimates are available for the entire conterminous United States, as well as Alaska, Hawaii, and Puerto Rico.

As our understanding and the availability of data increases, the "particular" PMP estimates that appear in NWS Hydrometeorological Reports may require adjustment in order to better define the conceptual PMP for a specific site. Therefore, **it is appropriate to refine PMP estimates with site specific or regional studies performed by a qualified hydrometeorologist with experience in determining PMP.** The results of available research such as that developed by the Electric Power Research Institute for the Wisconsin and Michigan areas should be considered in performing site specific studies. Since these studies can become very time consuming and costly, the benefit of a site specific study must be carefully considered.

See Appendix IIB for guidelines adopted by FERC staff on the use of Hydrometeorological Report (HMR) Nos. 51 and 52 vs. HMR No. 33.

2-3.3 Low Hazard Dams

Dams identified as having a low hazard potential should be designed to at least meet a minimum standard to protect against the risk of loss of benefits during the life of the project. Flood frequency and risk base analyses may be used for this analysis. The IDF for low hazard potential dams is typically the One Percent (100-year) flood.

2-4 Accommodating Inflow Design Floods

2-4.1 Flood Routing Guidelines

2-4.1.1 General

Site-specific considerations should be used to establish flood routing criteria for each dam and reservoir. The criteria for routing the IDF should be consistent with the reservoir regulation procedure that is to be followed in actual operation. General guidelines to be used in establishing criteria follow.

2-4.1.2 Guidelines for Initial Elevations

Specific guidance for establishing the initial reservoir elevation during the PMF is provided in Chapter VIII of these Guidelines. These criteria should also be applied to routing the IDF when the IDF is less than the PMF. In general, if there is no allocated or planned flood control storage (e.g. run-of-river), the flood routing usually begins with the reservoir at the normal maximum pool elevation. If regulation studies show that pool levels would be lower than the normal maximum pool elevation during the critical IDF season, then the results of those specific regulation studies would be analyzed to determine the appropriate initial pool level for routing the IDF.

2-4.1.3 Reservoir Constraints

Flood routing criteria should recognize constraints that may exist on the maximum desirable water surface elevation. A limit or maximum water surface reached during a routing of the IDF can be achieved by providing spillways and outlet works with adequate discharge capacity. Backwater effects of flood flow into the reservoir must specifically be considered when constraints on water surface elevation are evaluated. Reservoir constraints may include the following:

- Topographic limitations on reservoir stage which exceed the economic limits of dike construction.

- Public works around the reservoir rim which are not to be relocated, such as water supply facilities and sewage treatment plants.

- Dwellings, factories, and other developments around the reservoir rim which are not to be relocated.

- If there is a loss of storage capacity caused by sediment accumulation in portions of the reservoir, then this factor should be accounted for in routing the IDF. Sediment deposits in reservoir headwater areas may build up a delta which can increase flooding in that area, as well as reduce flood storage capacity, thereby having an effect on routings.

- Geologic features that may become unstable when inundated, and result in landslides which would threaten the safety of the dam, domestic and/or other developments, or displace needed storage capacity.

- Flood plain management plans and objectives established under Federal or State regulations and/or authorities.

2-4.1.4 Reservoir Regulation Requirements

Considerations to be evaluated when establishing flood routing criteria for a project include:

- regulation requirements to meet project purposes;
- the need to impose a maximum regulated release rate to prevent flooding or erosion of downstream areas and control rate of drawdown;
- the need to provide a minimum regulated release capacity to recover flood control storage for use in regulating subsequent floods;
- and the practicability of evacuating the reservoir for emergencies and for performing inspection, maintenance, and repair.

Spillways, outlet works, and penstocks for powerplants are sized to satisfy project requirements and must be operated in accordance with specific instructions if these project works are relied upon to make flood releases, subject to the following limitations:

- Only those release facilities which can be expected to operate reliably under the assumed flood condition should be assumed to be operational for flood routing. Reliability depends upon structural competence and availability for use. Availability and reliability of generating units for flood release during major floods should be justified. Availability of a source of auxiliary power for gate operation, effects of reservoir debris on operability and discharge capacity of gates and other facilities, accessibility of controls, design limits on operating head, reliability of access roads, and availability of operating personnel at the site during flood events are other factors to be considered in determining whether to assume release facilities are operational.

- A positive way of making releases to the natural watercourse by use of a bypass or wasteway must be available if canal outlets are to be considered available for making flood releases.

- Bypass outlets for generating units may be used if they are or can be isolated from the turbines by gates or valves.

- In flood routing, assumed releases are generally limited to maximum values determined from project uses, by availability of outlet works, tailwater conditions including effects of downstream tributary inflows and wind tides, and downstream non-damaging discharge capacities until allocated storage elevations are exceeded.

When a reservoir's capacity in regulating flows is exceeded, then other factors, particularly dam safety, will govern releases.

- During normal flood routing, the rate of outflow from the reservoir should not exceed the rate of inflow until the outflow begins to exceed the maximum project flood discharge capacity at normal pool elevation, nor should the maximum rate of increase of outflow exceed the maximum rate of increase of inflow. This is to prevent outflow conditions from being more severe than pre-dam conditions. An exception to the preceding would be the case where streamflow forecasts are available and pre-flood releases could reduce reservoir levels to provide storage for flood flows.

2-4.1.5 Evaluation of Domino-like Failure

If one or more dams are located downstream of the site under review, the failure wave should be routed downstream to determine if any of the downstream dams would breach in a domino-like action. The flood routing of flows entering the most upstream of a series of such dams may be either dynamic or level pool. The routing through all subsequent downstream reservoirs should be dynamic. Tailwater elevations should consider the effect of backwater from downstream constrictions.

2-4.2 Spillway and Flood Outlet Selection and Design

2-4.2.1 General

Spillways and flood outlets should be designed to safely convey major floods to the watercourse downstream from the dam and to prevent overtopping of the dam. They are selected for a specific dam and reservoir on the basis of release requirements, topography, geology, dam safety, and project economics.

2-4.2.2 Gated or Ungated Spillways

An ungated spillway releases water whenever the reservoir elevation exceeds the spillway crest level. A gated spillway can regulate releases over a broad range of water levels.

Ungated spillways are more reliable than gated spillways. Gated spillways provide greater operational flexibility and large discharge capacity per unit length. Operation of gated spillways and/or their regulating procedures should generally ensure that the peak flood outflow does not exceed the natural downstream flow that would occur without the dam.

The selection of a gated or ungated type of spillway for a specific dam depends upon site conditions, project purposes, economic factors, costs of operation and maintenance, and other considerations.

The following paragraphs focus on considerations that influence the choice between gated and ungated spillways:

(1) Discharge capacity - For a given spillway crest length and maximum allowable water surface elevation, a gated spillway can be designed to release higher discharges than an ungated spillway because the crest elevation may be lower than the normal reservoir storage level. This is a consideration when there are limitations on spillway crest length or maximum water surface elevation.

(2) Project objectives and flexibility - Gated spillways permit a wide range of releases and have capability for pre-flood drawdown.

(3) Operation and maintenance - Gated spillways may experience more operational problems and are more expensive to construct and maintain than ungated spillways. Constant attendance or several inspections per day by an operator during high water levels is highly desirable for reservoirs with gated spillways, even when automatic or remote controls are provided. During periods of major flood inflows where automatic or remote controls are not provided, the spillway should be constantly manned. Gated spillways are more subject to clogging from debris and jamming from ice, whereas, properly designed ungated spillways are basically free from these problems. Gated spillways require regular maintenance, and, as a minimum, an annual operation test for safety purposes. However, ungated spillways can have flashboards, trip gates, stop log sections, etc. which can have operational problems during floods and may require constant attendance or several inspections per day during high water levels.

(4) Reliability - The nature of ungated spillways reduces dam failure potential associated with improper operation and maintenance. Where forecasting capability is unreliable, or where time from the beginning of runoff to peak inflow is only a few hours, ungated spillways are more reliable, particularly for high hazard structures. Consequences of failure of operation equipment or errors in operation are more severe for gated spillways.

(5) Data and control requirements - Gated spillways require reliable real time hydrologic and meteorological data to make proper regulation possible.

(6) Emergency evacuation - Unless ungated spillways have removable sections such as flashboards, trip gates, or stop log sections, they cannot be used to evacuate a reservoir during emergencies. The capability of gated spillways to draw down pools from the top of the gates to the spillway crest can be an advantage when emergency evacuation to reduce head on the dam is a concern.

(7) Economics and selection - Designs to be evaluated should be technically adequate alternatives. Economic considerations often indicate whether gated or ungated spillways are selected. The possibility of selecting a combination of more than one type of spillway is also a consideration. Final selection of the type of crest

control should be based on a comprehensive analysis of all pertinent factors, including advantages, disadvantages, limitations, and feasibility of options.

2-4.2.3 Design Considerations

Dams and their appurtenant structures should be designed to give satisfactory performance and to practically eliminate the probability of failure. These guidelines identify three specific classifications of spillways (service, auxiliary, and emergency) and outlet works that are used to pass floodwaters, each serving a particular function. The following paragraphs discuss functional requirements.

<u>Service spillways</u> should be designed for frequent use and should safely convey releases from a reservoir to the natural watercourse downstream from the dam. Considerations must be given to waterway freeboard, length of stilling basins, if needed, and amount of turbulence and other performance characteristics. It is acceptable for the crest structure, discharge channel (e.g., chute, conduit, tunnel), and energy dissipator to exhibit marginally safe performance characteristics for the IDF. However, they should exhibit excellent performance characteristics for frequent and sustained flows such as up to the 1 percent chance flood event. Other physical limitations may also exist which have an effect on spillway sizing.

<u>Auxiliary spillways</u> are usually designed for infrequent use and it is acceptable to sustain limited damage during passage of the IDF. The design of auxiliary spillways should be based on economic considerations and be subject to the following requirements:

- The auxiliary spillway should discharge into a watercourse sufficiently separated from the abutment to preclude abutment damage and should discharge into the main stream a sufficient distance downstream from the toe of the dam so that flows will not endanger the dam's structural integrity or usefulness of the service spillway.

- The auxiliary spillway channel should either be founded in competent rock or an adequate length of protective surfacing should be provided to prevent the spillway crest control from degrading to the extent that it results in an unacceptable loss of conservation storage or a large uncontrolled discharge which exceeds peak inflow.

<u>Emergency spillways</u> may be used to obtain a high degree of hydrologic safety with minimal additional cost. Because of their infrequent use it is acceptable for them to sustain significant damage when used and they may be designed with lower structural standards than those used for auxiliary spillways.

An emergency spillway may be advisable to accommodate flows resulting from mis-operation or malfunction of other spillways and outlet works. Generally, they are sized to accommodate a flood smaller than the IDF. The crest of an emergency spillway should be set above the normal maximum water surface (attained when accommodating the IDF) so it will not overflow as a result of reservoir setup and wave action. The design of an emergency spillway should be subject to the following limitations:

- The structural integrity of the dam should not be jeopardized by spillway operation.

- Large conservation storage volumes should not be lost as a result of degradation of the crest during operation.

- The effects of a downstream flood resulting from uncontrolled release of reservoir storage should not be greater than the flood caused by the IDF without the dam.

<u>Outlet works</u> used in passing floods and evacuating reservoir storage space should be designed for frequent use and should be highly reliable. Reliability is dependent on foundation conditions which influence settlement and displacement of waterways, on structural competence, on susceptibility of the intake and conduit to plugging, on hydraulic effects of spillway discharge, and on operating reliability.

2-4.3 Freeboard Allowances

2-4.3.1 General

Freeboard provides a margin of safety against overtopping failure of dams. It is generally not necessary to prevent splashing or occasional overtopping of a dam by waves under extreme conditions. However, the number and duration of such occurrences should not threaten the structural integrity of the dam, interfere with project operation, or create hazards to personnel. Freeboard provided for concrete dams can be less conservative than for embankment dams because of their resistance to wave damage or erosion. If studies demonstrate that concrete dams can withstand the PMF while overtopped without significant erosion of foundation or abutment material, then no freeboard should be required for the PMF condition. Special consideration may be required in cases where a powerplant is located near the toe of the dam. The U.S. Bureau of Reclamation has developed guidelines (Ref. 12) that provide criteria for freeboard computations.

Normal freeboard is defined as the difference in elevation between the top of the dam and the normal maximum pool elevation. Minimum freeboard is defined as the difference in pool elevation between the top of the dam and the maximum reservoir water surface that would result from routing the IDF through the reservoir. Intermediate freeboard is defined as the difference between intermediate storage level and the top of the dam. Intermediate freeboard may be applicable when there is exclusive flood control storage.

2-4.3.2 Freeboard Guidelines

Following are guidelines for determining appropriate freeboard allowances:

- Freeboard allowances should be based on site-specific conditions and the type of dam (concrete or embankment).

- Both normal and minimum freeboard requirements should be evaluated in determining the elevation of the top of the dam. The resulting higher top of dam elevation should be adopted for design.

- Freeboard allowances for wind-wave action should be based upon the most reliable wind data available that are applicable to the site. The significant wave should be the minimum used in determining wave runup; and the sum of wind setup and wave runup should be used for determining requirements for this component of freeboard.

- Computations of wind-generated wave height, setup, and runup should incorporate selection of a reasonable combined occurrence of pool level, wind velocity, wind direction, and wind durations based on site-specific studies.

- It is highly unlikely that maximum winds will occur when the reservoir water surface is at its maximum elevation resulting from routing the IDF, because the maximum level generally persists only for a relatively short period of time (a few hours). Consequently, winds selected for computing wave heights should be appropriate for the short period the pool would reside at or near maximum levels.

- Normal pool levels persist for long periods of time. Consequently, maximum winds should be used to compute wave heights.

- Freeboard allowance for settlement should be applied to account for consolidation of foundation and embankment materials when uncertainties exists in computational methods or data used yield unreliable values for camber design. Freeboard allowance for settlement should not be applied where an accurate determination of settlement can be made and is included in the camber.

- Freeboard allowance for embankment dams for estimated earthquake-generated movement, resulting seiches, and permanent embankment displacements or deformations should be considered if a dam is located in an area with potential for intense seismic activity.

- Reduction of freeboard allowances on embankment dams may be appropriate for small fetches, obstructions that impede wave generation, special slope and crest protection, and other factors.

- Freeboard allowance for wave and volume displacement due to potential landslides which cannot be economically removed or stabilized should be considered if a reservoir is located in a topographic setting where the wave or higher water resulting from displacement may be destructive to the dam or may cause serious downstream damage.

- Total freeboard allowances should include only those components of freeboard which can reasonably occur simultaneously for a particular water surface elevation. Components of freeboard and combinations of those components which have a reasonable probability of simultaneous occurrence are listed in the following paragraphs for estimating minimum, normal, and intermediate freeboards. The top of the dam should be established to accommodate the most critical combination of water surface and freeboard components from the following combinations.

For minimum freeboard combinations the following components, when they can reasonably occur simultaneously, should be added to determine the total minimum freeboard requirement:

(1) Wind-generated wave runup and setup for a wind appropriate for maximum reservoir stage for the IDF.

(2) Effects of possible malfunction of spillway and/or outlet works during routing of the IDF.

(3) Settlement of embankment and foundation not included in crest camber.

(4) Landslide-generated waves and/or displacement of reservoir volume (only cases where landslides are triggered by the occurrence of higher water elevations and intense precipitation associated with the occurrence of the IDF).

For normal freeboard combinations, the more critical of the following two combinations of components should be used for determining normal freeboard requirements:

(1) Combination 1

(a) Wind-generated wave runup and setup for maximum wind, and

(b) Settlement of embankment and foundation not included in camber.

(2) Combination 2

(a) Landslide-generated waves and/or displacement of reservoir volume;

(b) Settlement of embankment and foundation not included in camber; and

(c) Settlement of embankment and foundation or seiches as a result of the occurrence of the maximum credible earthquake.

For intermediate freeboard combinations, in special cases, a combination of intermediate winds and water surface between normal and maximum levels should be evaluated to determine whether this condition is critical. This may apply where there are exclusive flood control storage allocations.

2-5. References

1. Federal Emergency Management Agency (2004). Federal Guidelines for Dam Safety: Hazard Potential Classification System for Dams.

2. Committee on Safety Criteria for Dams (1985). Safety of Dams - Flood and Earthquake Criteria. Prepared under the Auspices of Water Science and Technology Board, Commission on Engineering and Technical Systems, National Research Council, Washington, D.C.: National Academy Press, 374 pp.

3. Federal Emergency Management Agency (2004). Federal Guidelines for Selecting and Accommodating Inflow Design Floods for Dams.

4. Federal Energy Regulatory Commission (1993). Operating Manual for Inspection of Projects and Supervision of Licenses for Water Power Projects.

5. Hydrology Subcommittee, (1981). Estimating Peak Flow Frequencies for Natural Ungaged Watersheds: A Proposed Nationwide Test. U.S. Water Resources Council, 346 pp.

6. Hydrology Subcommittee, (1982). Guidelines for Determining Flood Flow Frequency. Hydrology Subcommittee Bulletin 17B, with editorial corrections. Interagency Advisory Committee on Water Data, U.S. Geological Survey, 28 pp.

7. Interagency Committee on Water Data (1985 Draft). Feasibility of Assigning a Probability to the Probable Maximum Flood. Prepared by Working Group of Hydrology Committee.

8. International Symposium on Flood Frequency and Risk Analyses at Louisiana State University, Baton Rouge, La. (May 14-17, 1986). Proceedings to be published late 1986.

9. Lane, W. L., (1985). Rare Flood Frequency Estimation - A Case Study of the Pecos River, (Abstract). EOS Transactions of AGU, 66(18), 267.

10. Myers, V.A., (1969). The Estimation of Extreme Precipitation as the Basis for Design Floods - Resume of Practice in the United States. Proceedings of the Leningrad Symposium on Floods and Their Computation, August 1967, Vol. 1, International Association of Scientific Hydrology, Gentbrugge, Belgium, 84-101.

11. Naef, F., (1981). Can We Model the Rainfall-Runoff Process Today? Hydrological Sciences Bulletin, 26.

12. U.S. Bureau of Reclamation, (1981). Freeboard Criteria and Guidelines for Computing Freeboard Allowances for Storage Dams. ACER Technical Memorandum No. 2.

13. U.S. Bureau of Reclamation (1988). Downstream Hazard Classification Guidelines. Acer Technical Memorandum No. 11

2.6 APPENDICES

APPENDIX II-A

Dam Break Studies

The evaluation of the downstream consequences in the event of a dam failure is a main element in determining hazard potential and formulating emergency action plans for hydroelectric projects. The solution requires knowledge of the lateral and longitudinal geometry of the stream, its frictional resistance, a discharge-elevation relationship at one boundary, and the time-varying flow or elevation at the opposite boundary.

The current state-of-the-art is to use transient flow or hydraulic methods to predict dam break wave formation and downstream progression. The transient flow methods solve and therefore account for the essential momentum forces involved in the rapidly changing flow caused by a dam break. Another technique, referred to as storage routing or the hydrologic method, solves one-dimensional equations of steady flow ignoring the pressure and acceleration contributions to the total momentum force. For the same outflow hydrograph, the storage routing procedures will always yield lower water surface elevations than hydraulic or transient flow routing.

When routing a dam break flood through the downstream reaches, appropriate local inflows should be included in the routing which are consistent with the assumed storm centering.

The mode and degree of dam failure involves considerable uncertainty and cannot be predicted with acceptable engineering accuracy; therefore, conservative failure postulations are necessary. Uncertainties can be circumvented in situations where it can be shown that the complete and sudden removal of a dam (or dams) will not endanger human life or cause significant property damage.

The following provides references on dam break analyses and criteria which may prove useful as indicators of reasonableness of the breach parameters, peak discharge, depth of flow, and travel time determined by the licensee.

I. REFERENCES

Suggested acceptable references regarding dam failure studies include the following:

A. Fread, D. L. "DAMBRK - The NWS Dam-Break Flood Forecasting Model," National Weather Service, Silver Spring, Maryland, 1988 Version.

B. Fread, D. L. "NWS FLDWAV Model: The Replacement of DAMBRK for Dam-Break Flood Prediction", Proceedings, Association of State Dam Safety Officials, 10th Annual Conference, Kansas City, Missouri, September 26-29, 1993.

C. Westmore, Jonathan N. and Fread, Danny L., "The NWS Simplified Dam-Break Flood Forecasting Model," National Weather Service, Silver Spring, Maryland, 1981. (Copy previously furnished to each Regional Office with a detailed example).

D. Fread, D. L., 1977: The development and testing of a dam-break flood forecasting model, "Proceedings, Dam-Break Flood Modeling Workshop," U.S. Water Resources Council, Washington, D.C., 1977, pp. 164-197.

E. Hydrologic Engineering Center, "Flood Hydrograph Package (HEC-1) User's Manual for Dam Safety Investigations," September, 1990.

F. Hydrologic Engineering Center, "HEC-RAS River Analysis System Hydraulic Reference Manual," Version 4.1, January 2010.

G. Hydrologic Engineering Center, "Hydrologic Modeling System (HEC-HMS) User's Manual," August 2010.

H. Gandlach, D. L. and Thomas, W. A., "Guidelines for Calculating and Routing a Dam-Break Flood," Research Note No. 5, U.S. Army Corps of Engineers, Hydrologic Engineering Center, 1977.

I. Cecilio, C. B. and Strassburger, A. G., "Downstream Hydrograph from Dam Failure," Engineering Foundation Conference on Evaluation of Dam Safety, 1976.

J. Soil Conservation Service, "Simplified Dam-Breach Routing Procedure," March 1979. (To be used only for flood routing technique, not dam break discharge).

K. Chow, V. T., <u>Open Channel Hydraulics</u>, McGraw-Hill Book Company, Inc., New York, 1959, Chapter 20.

L. Henderson, F. M., <u>Open Channel Flow</u>, McMillan Company, New York, 1966, Chapters 8 and 9.

M. Hydrologic Engineering Center, "Flood Emergency Plans, Guidelines for Corps Dam," June 1980. (Forwarded to all Regional Engineers by memorandum dated February 11, 1981).

N. Hydrologic Engineering Center, "UNET, One-Dimensional Unsteady Flow Through a Full Network of Open Channels", September 1992.

II. CRITERIA

The following criteria may prove useful as an indicator of the reasonableness of a dam break study:

A. If the dam break analysis has been performed by an acceptable method (**Reference F is the preferred methods**), then generally only the breach parameters, peak discharge, and flood wave travel time should be verified as an indicator of the licensee's correct application of the method selected. Downstream routing parameters (i.e., Manning's "n") should be reviewed for acceptability and inundation maps should be reviewed for clarity and completeness of information (i.e., travel times). The following criteria are considered to be adequate and appropriate for verifying the selected breach parameters and peak discharge:

1. Breach Parameters - Most serious dam failures result in a situation resembling weir conditions. Breach width selection is judgmental and should be made based on the channel or valley width with failure occurring at the deepest section. The bottom of the breach should generally be assumed to be at the foundation elevation of the dam. This appendix contains suggested breach parameters and should be used when verifying the selected breach parameters. For worst case scenarios, the breach width should be in the upper range while the time of failure should be in the lower range. However a sensitivity analysis is recommended to determine the reasonableness of the assumptions.

2. Peak Discharge - The peak discharge may be verified by use of equations (11) and (13) of Reference A. Although the equations assume a rectangular-shaped breach, a trapezoidal breach may be analyzed by specifying a rectangular breach width that is equal to the average width of the trapezoidal breach.

Equation 11:

$$C = (23.4\ A_S) / \overline{BR}$$

Where: C=constant

A_s = reservoir surface area, in acres

$\overline{BR}$ = average breach width, in feet

Equation 13:

$$Q_{bmax} = 3.1\ \overline{BR}\ (C / \{t_f + (C / H^{1/2})\})^3$$

Where: Q_{bmax} = maximum breach outflow, in cfs
t_f = time of failure, in hours
H = maximum head over the weir, in feet

This equation for Q_{bmax} has been found to give results within +5% of the Q_{peak} from the full DAMBRK model.

In a rare case where a dam impounding a small storage volume has a large time of failure, the equations above will predict a much higher flow than actually occurs.

At a National Weather Service Dam-Break Model Symposium held in Tulsa, Oklahoma, June 27-30, 1983, Dr. Danny Fread presented an update to his simplified method. Equation 13 has been modified as follows to include additional outflow not attributed to breach outflow:

$$Q_{bmax} = Q_o + 3.1\,\overline{BR}\,(C / \{t_f + (C / H^{1/2})\})^3$$

Where: Q_o = Additional (non-breach) outflow (cfs) at time t_f (i.e., spillway flow and/or crest overflow) (optional data value, may be set to 0).

This equation has also been modified to address instantaneous failure, because in some situations where a dam fails very rapidly, the negative wave that forms in the reservoir may significantly affect the outflow from the dam.

3. Flood Wave Travel Time - Reasonableness of the flood wave travel time may be determined by use of the following "rule-of-thumb" approximation for average wave speed:

(a) Assume an equivalent rectangular channel section for the selected irregular channel section.

(b) Assume a constant average channel slope.

(c) Compute depth of flow from the following adjusted Manning's equation.

$$d = (\{Q\ n\} / \{1.46\ B\ S^{1/2}\})^{0.6}$$

Where: d = depth of flow for assumed rectangular section, ft.
Q = peak discharge, cfs
B = average width (rectangular), ft.

S = average slope, ft/ft
n = Manning's roughness coefficient

(d) Compute average velocity from Manning's Equation:

$$V = (1.49\, S^{0.5}\, d^{0.67}) / n$$

Where: V = average velocity, fps

(e) Compute wave speed, C (Kinematic velocity):

$$C = (5/3)\, V\, (0.68)$$

Where: C = wave speed (mph)

Note: 1 fps = 0.68 mph

(f) Determination travel time, TT

$$TT = X / C$$

Where: TT = travel time, hr.
X = distance from dam, mi.

Note: If the slope is flat, the following "rule-of-thumb" provides a very rough estimate of the wave speed:

$$C = 2\, S^{0.5}$$

Where: C = wave speed, mph
S = average slope, ft/mi

In addition, as a "rule-of-thumb", the dynamic routing (HEC-RAS) method should be used whenever severe backwater conditions at downstream areas occur and/or the slope is less than 20 ft/mi. When these restrictions are not present normal hydrologic routing (HEC-HMS) may provide reasonable results. It is recommended that HEC-RAS be used to determine the resulting water surface elevations when HEC-HMS is used for the dam break study. Other modeling software may be used if pre-agreed to by the Commission.

B. If a dam break analysis has been performed by a method other than one of the suggested acceptable methods, the selected breach parameters, peak discharge, depth of flow and travel time of the flood wave shall be verified by one of the two methods:

1. **Unsteady Flow - Dynamic Routing Method (Recommended)**

The Army Corps of Engineers' HEC-RAS model (Reference F) is the recommended one dimensional method. As the flood wave travels downstream, the peak discharge and wave velocity generally, but not always, decrease. This attenuation in the flood wave is primarily due to energy dissipation when it is near the dam and to valley storage as it progresses in an unsteady flow downstream. It is important that the HEC-RAS model be calibrated to historical floods, if at all possible.

2. **Steady Flow Method (Provides a rough estimate)**

If this method is selected, the breach parameters and peak discharge shall be verified as in part "A" above. The method described below should be utilized only for preliminary assessments and the obtained values may be far from the actually expected results. Sound judgement and extensive numerical experience is necessary when evaluating the results.

For a rough estimate of the travel time and flood wave, it is recommended that one of the following two steady state methods be used for verification of the licensee's values:

a. When steam gage data are available, the depth of flow and travel time can be estimated as follows (This method will indirectly take valley storage into consideration):

(1) Identify existing stream gages located downstream of the dam.

(2) Obtain the stage-discharge curve for each gage.

(3) Assuming Q_{peak} remains constant, extrapolate the curves to the Q_{peak} value of the flood wave and determine the corresponding water surface elevation.

(4) Using the continuity equation to determine the velocity, estimate the travel time between each cross-section.

b. When stream gage data is not available, the depth of flow and travel time can be estimated based on the following steady-state method:

(1) Assume the area downstream of the dam is a channel. This will neglect valley storage.

(2) Identify on topographic maps all abrupt changes in channel width and/or slope. Using this as a basis, select and plot channel cross-sections.

(3) Assume Q_{bmax} remains constant throughout the entire stream length under consideration.

(4) Selecting a fairly rough Manning's n value, determine the depth of flow by applying Manning's equation to each cross-section. Assume the energy slope is equal to the slope of the channel.

(5) Using the continuity equation to determine the velocity, estimate the travel time between each cross-section.

C. The above criteria for breach parameters, peak discharge, depth of flow, and travel time should provide the necessary "ballpark figures" needed for comparison with licensee's estimates. When large discrepancies in compared values exist, or questions arise about assumptions to be made, or it appears that an extensive review will be necessary, the Regional Engineer should contact the Director or Deputy Director, D2SI for guidance. The methodology used by the licensee should be a part of the study and should be requested if not included.

TABLE 1
SUGGESTED BREACH PARAMETERS
(Definition Sketch Shown in Figure 1)

Parameter	Value	Type of Dam
Average width of Breach ($\overline{BR}$) (See Comment No. 1)*	$\overline{BR}$ = Crest Length	Arch
	$\overline{BR}$= Multiple Slabs	Buttress
	$\overline{BR}$ = Width of 1 or more Usually $\overline{BR} \leq 0.5$ W	Masonry, Gravity Monoliths
	HD $\leq \overline{BR} \leq$ 5HD (usually between 2HD & 4HD)	Earthen, Rockfill, Timber Crib
	$\overline{BR} \geq$ 0.8 x Crest Length	Slag, Refuse
Horizontal Component of Side Slope of Breach (Z) (See Comment No. 2)* Buttress	0 $\leq$ Z $\leq$ slope of valley walls	Arch
	Z = O	Masonry, Gravity Timber Crib,
	¼ $\leq$ Z $\leq$ 1	Earthen (Engineered, Compacted)
	1 $\leq$ Z $\leq$ 2	Slag, Refuse (Non-Engineered)
Time to Failure (TFH) (in hours) (See Comment No. 3)*	TFH $\leq$ 0.1	Arch
	0.1 $\leq$ TFH $\leq$ 0.3	Masonry, Gravity, Buttress
	0.1 $\leq$ TFH $\leq$ 1.0	Earthen (Engineered, Compacted) Timber
Crib		
	0.1 $\leq$ TFH $\leq$ 0.5	Earthen (Non
Engineered		
		Poor Construction)
	0.1 $\leq$ TFH $\leq$ 0.3	Slag, Refuse

Definition:

HD	-	Height of Dam
Z	-	Horizontal Component of Side Slope of Breach
$\overline{BR}$	-	Average Width of Breach
TFH	-	Time to Fully Form the Breach
W	-	Crest Length

Note: See Page 2-A-11 for definition Sketch
**Comments: See Page 2-A-9 - 2-A-10*

Comments:

1. $\overline{BR}$ is the <u>average</u> breach width, which is not necessarily the bottom width. $\overline{BR}$ is the bottom width for a rectangle, but $\overline{BR}$ is not the bottom width for a trapezoid.

2. Whether the shape is rectangular, trapezoidal, or triangular is not generally critical if the average breach width for each shape is the same. What is critical is the assumed average width of the breach.

3. Time to failure is a function of height of dam and location of breach. Therefore, the longer the time to failure, the wider the breach should be. Also, the greater the height of the dam and the storage volume, the greater the time to failure and average breach width will probably be. Time to failure is the time from the start of the breach formation until the complete breach is formed. It does not include the time leading up to the start of the breach formation. For example, the time to erode away the downstream slope of an earth dam is not included. In this situation, the time to failure commences after sufficient erosion of the downstream slope has occurred and actual formation of the breach (the lowering of the crest) has begun.

4. The bottom of the breach should be at the foundation elevation.

5. Breach width assumptions should be based on the type of dam, the height of dam, the volume of the reservoir, and the type of failure (e.g. piping, sustained overtopping, etc.). Slab and buttress dams require sensitivity analyses that vary the number of slabs assumed to fail.

6. For a worst-case scenario, the average breach width should be in the upper portion of the recommended range, the time to failure should be in the lower portion of the range, and the Manning's "n" value should be in the upper portion of the recommended range. In order to fully evaluate the impacts of a failure on downstream areas, a sensitivity analysis is required to estimate the confidence and relative differences resulting from varying assumptions.

    a. To compare relative differences in peak elevation based on variations in breach widths, the sensitivity analysis should be based on the following assumptions:

        1. Assume a probable (reasonable) maximum breach width, a probable minimum time to failure, and a probable maximum Manning's "n" value. Manning's "n" values for sections immediately below the dam

and up to several thousand feet or more downstream of the dam should be assumed to be larger than the maximum value suggested by field investigations in order to account for uncertainties of high energy losses, velocities, turbulence, etc., resulting from the initial failure.

2. Assume a probable minimum breach width, a probable maximum time to failure, and a probable minimum Manning's "n" value.

Plot the resulting water surface elevation at selected locations downstream from the dam for each run on the same graph. Compare the differences in elevation with respect to distance downstream from the dam for the two cases.

b. To compare differences in travel time of the flood wave, the sensitivity analysis should be based on the following assumptions:

1. Use criteria in a. 1.

2. Assume a probable maximum breach width, a probable minimum time to failure, and a probable minimum Manning's "n" value.

Plot the results (elevation-distance downstream) of both runs on the same graph to compare the changes in travel time with respect to distance downstream from the dam.

c. To compare differences in elevation between natural flood conditions and natural flood conditions plus dam break, the sensitivity analysis should be based on the following assumptions:

1. Route natural flood without dam break assuming maximum probable Manning's "n" value.

2. Use criteria in a. 1.

Plot the results (elevation-distance downstream) of both runs on the same graph to compare the changes in elevation with respect to distance downstream from the dam.

7. When dams are assumed to fail from overtopping, wider breach widths than those suggested in Table 1 should be considered if overtopping is sustained for a long period of time.

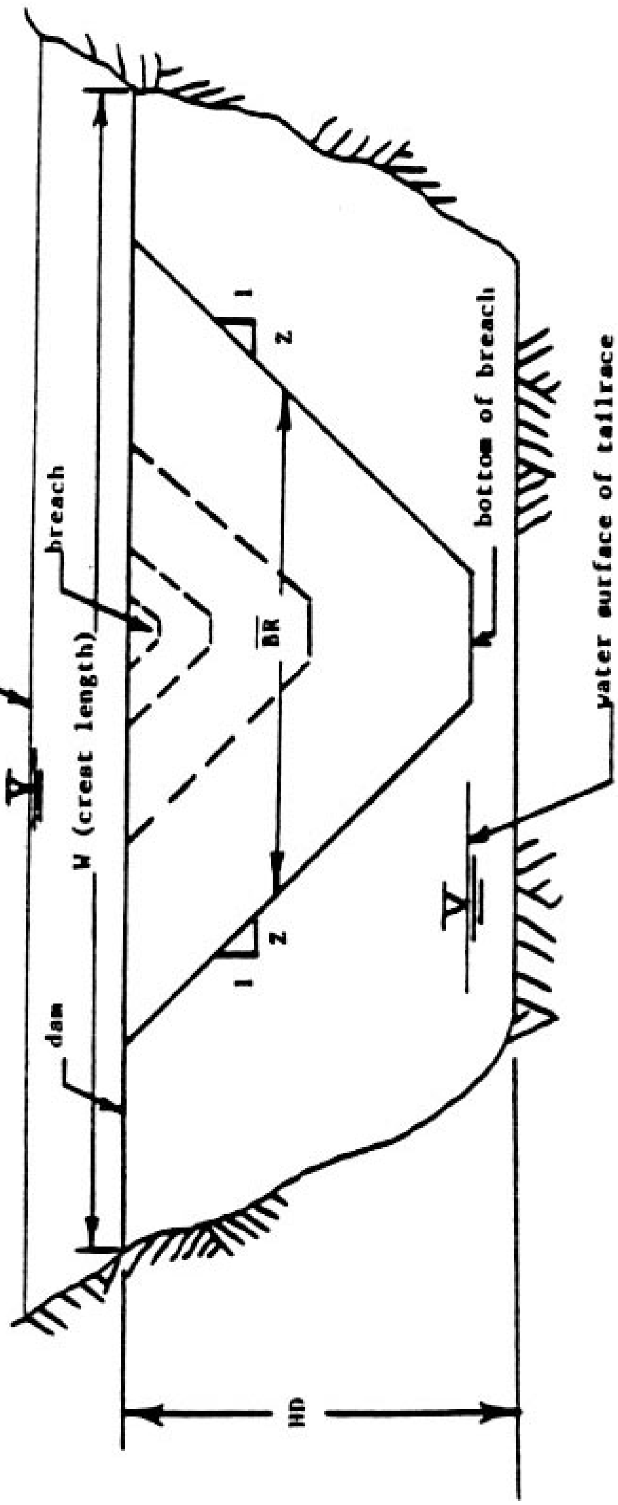


FIGURE 1. DEFINITION SKETCH OF BREACH PARAMETERS

APPENDIX II-B

Hydrometeorological Report (HMR) Nos. 51 and 52 vs HMR No. 33

In accordance with Section 12.35(b)(1) of the Commission's Regulations, if structural failure of project works (water impounding structures) would present a hazard to human life or cause significant property damage, licensed or exempted project works subject to Part 12 of the Commission's Regulations must be analyzed to evaluate their capability to withstand the loading conditions and/or overtopping which may occur from a flood up to the probable maximum flood (PMF) or the capacity of spillways to prevent the reservoir from rising to an elevation that would endanger downstream life and property.

As a result of the recent publications of Hydrometeorological Reports Nos. 51 and 52 (HMR Nos. 51 and 52), the FERC Staff has adopted the following guidelines for evaluating the spillway adequacy of all licensed and exempted projects located east of the 105th meridian:

(1) For existing structures where a reasonable determination of the Probable Maximum Precipitation (PMP) has not previously been made using suitable methods and data such as contained in HMR No. 33 or derived from specific meteorologic studies, or the PMF has not been properly determined, the ability of the project structures to withstand the loading or overtopping which may occur from the PMF must be re-evaluated using HMR Nos. 51 and 52.

(2) For existing structures where a reasonable determination of the PMP has previously been made, a PMF has been properly determined, and the project structures can withstand the loading or overtopping imposed by that PMF, a re-evaluation of the adequacy of the spillway using HMR Nos. 51 and 52 is not required. Generally, no PMF studies will be repeated solely because of the publication of HMR Nos. 51 and 52. However, there is no objection to using the two reports for necessary PMF studies for any water retaining structure, should you so desire.

(3) For all unconstructed projects and for those projects where any proposed or required modification will significantly affect the stability of water impounding project structures, the adequacy of the project spillway must be evaluated using:

   (a) HMR Nos. 51 and 52, or

   (b) Specific basin studies where the project lies in the stippled areas on Figures 18 through 47 of HMR No. 51.

APPENDIX II-C

FLOWCHARTS FOR SELECTING APPROPRIATE INFLOW DESIGN FLOOD (IDF) AND DETERMINING NEED FOR REMEDIAL ACTION

INTRODUCTION

The purpose of this appendix is to describe the procedures used to select the appropriate inflow design flood (IDF) for a dam, and to determine the need for remedial action. These procedures are presented in two flowcharts. The first flowchart describes the steps needed to determine. . .

- If the probable maximum flood (PMF) was used in the original design of the dam,
- If the PMF or some lesser flood is the appropriate IDF, and,
- Whether remedial action at the dam is needed to enable it to safely accommodate the appropriate PMF and/or IDF.

In order to determine whether the PMF or some lesser flood is the appropriate IDF, it may be necessary to conduct an incremental hazard evaluation. This process is presented in the second flowchart.

Following each flowchart is a breakdown of the procedures. Each block is presented individually, and includes an explanation of the steps taken.

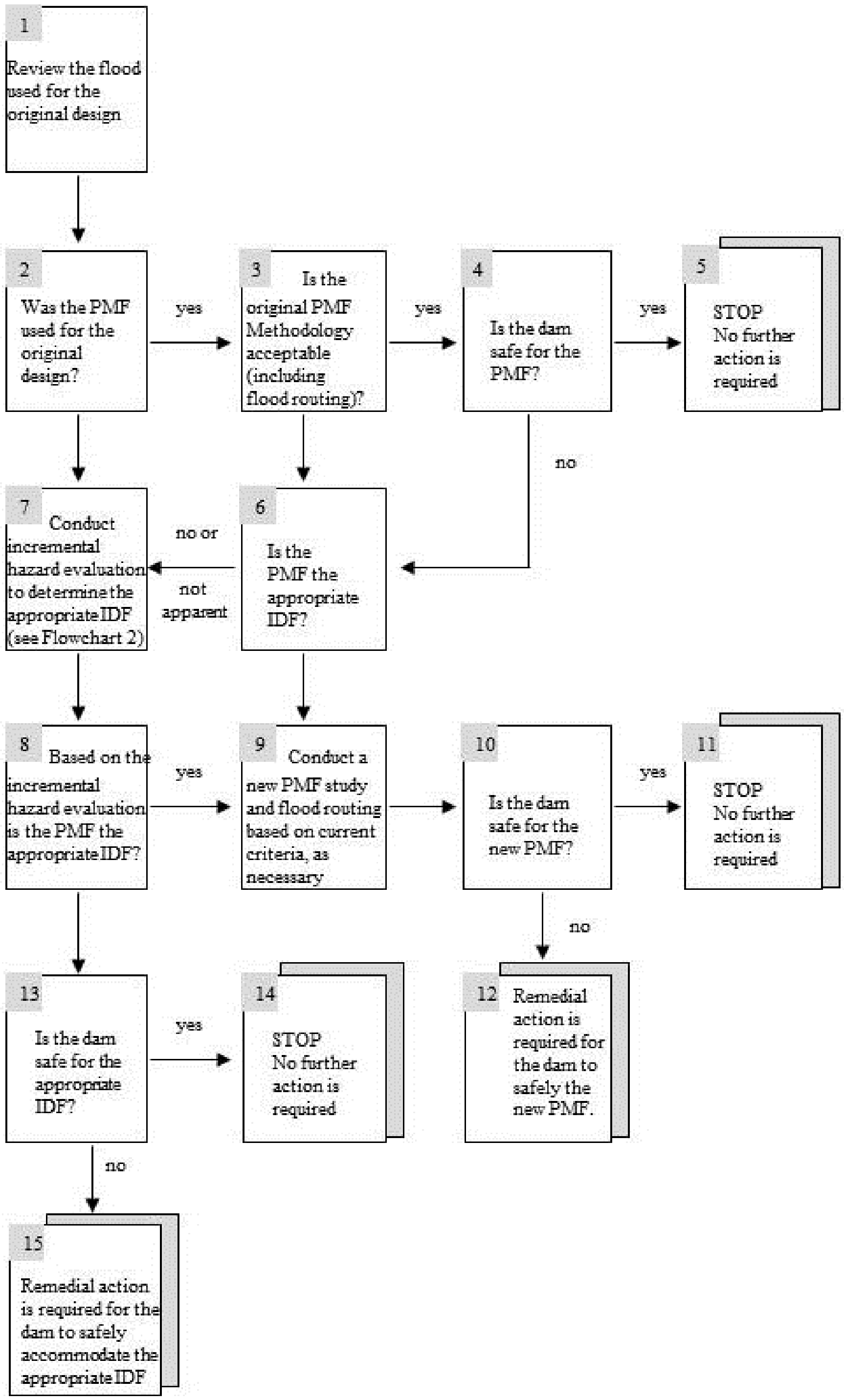
PROCEDURES FOR DETERMINING THE APPROPRIATE IDF
AND THE NEED FOR REMEDIAL ACTION

Flowchart 1 in Figure 1 presents a logical, step-by-step approach for evaluating the hydrologic design of an existing dam, and determining the appropriate IDF for the dam and whether remedial action is needed in order for the dam to safely accommodate the IDF.

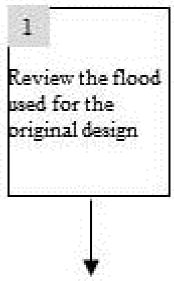
⇨

Flowchart 1 is on the next page

FIGURE 1. FLOWCHART 1 -- PROCEDURES FOR DETERMINING THE APPROPRIATE INFLOW DESIGN FLOOD (IDF) AND THE NEED FOR REMEDIAL ACTION



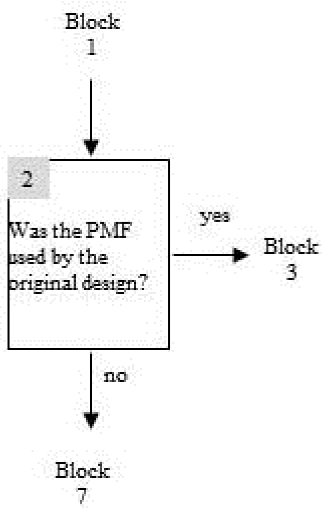
Block 1



The initial step in selecting the appropriate IDF and determining the need for dam safety modification is to review the basis for the original hydrologic design of an existing dam. This information will provide valuable insight regarding whether the flood originally used for design purposes satisfies current criteria or whether detailed investigations and analyses will be required to determine the appropriate IDF for the dam.

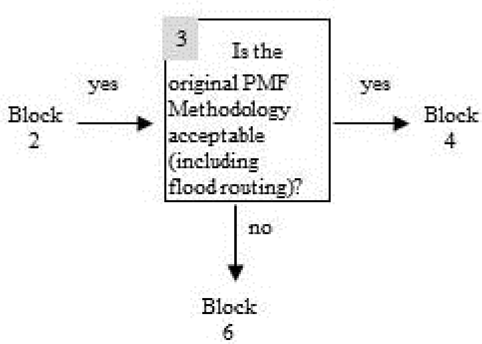
In those situations where the original design information has been lost, detailed investigations and analyses will normally be required.

Block 2



Once you have identified the basis for the original hydrologic design, the next step is to determine if the flood used for the original design is the probable maximum flood (PMF). This question is important, since the upper limit of the IDF is the PMF.
If your answer is **YES**, continue to Block 3.
If your answer is **NO**, go to Block 7. In Block 7 you will perform an incremental hazard evaluation to determine the appropriate IDF.

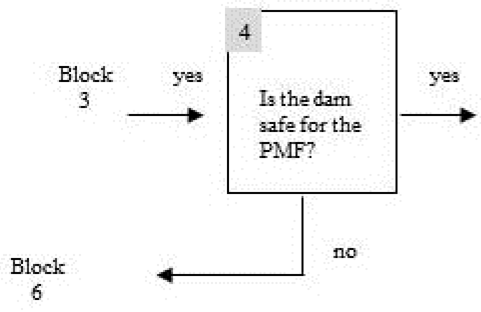
Block 3



To ensure the reliability of the original PMF study or the assumptions made on the various parameters affecting the study, it is necessary to determine if the PMF methodology originally used is still acceptable under current criteria.
If your answer is **YES**, continue to Block 4.
If your answer is **NO**, go to Block 6. In Block 6, you will answer the question: Is the PMF the appropriate IDF?

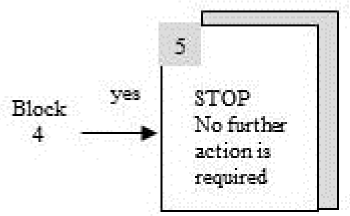
Continued…

Block 4



Determine if the dam is safe for the PMF. Your answer to this question will indicate whether remedial action will be required.
If your answer is **YES**, continue to Block 5.
If your answer is **NO**, go to Block 6, you will answer the questions: Is the PMF the appropriate IDF?

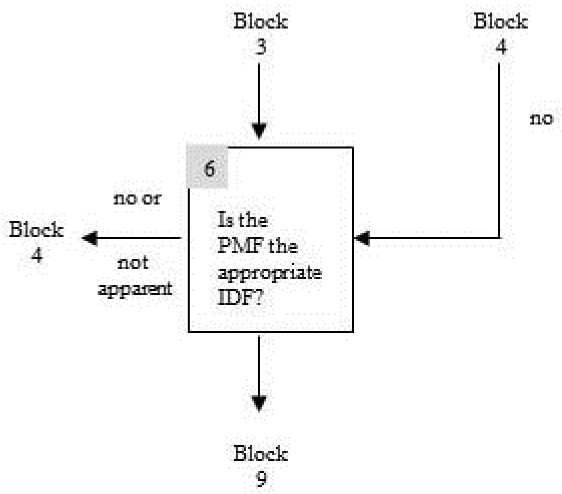
Block 5



If the PMF is considered to be the appropriate IDF for the dam, no further investigations or remedial work for hydrologic conditions will be required.

Continued…

Block 6



IF . . .

In Block 3 you determined that the original PMF methodology is NOT acceptable,

OR . . .

in Block 4 you determined that the dam is **NOT** safe for the PMF,

THEN . . .

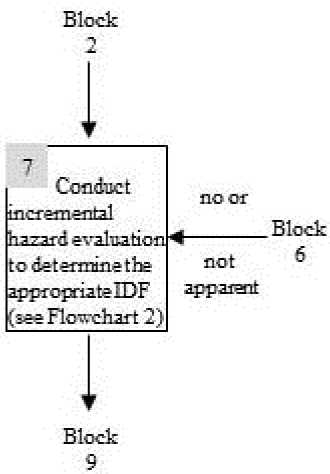
You need to determine if the PMF is the appropriate IDF.

In some cases, such as when the dam is totally submerged during the PMF, it may be obvious that the appropriate IDF is something less than the PMF. In other cases, it will not be apparent whether the IDF should be the PMF or something less. In these two cases, it will be necessary to perform an incremental hazard evaluation to determine the appropriate IDF for the dam. Continue to Block 7.

Sometimes, based on the size and volume of the dam and reservoir, the proximity of the dam to downstream communities, or even because of political decisions, it will be obvious that the IDF should be the PMF. If this is the case, a new PMF study will be required. Go to Block 9.

Continued…

Block 7



IF . . .

In Block 2 you determine that the flood used in the original design is **NOT** the PMF,

OR . . .

In Block 6 you determined that it is **obvious** that the IDF should be less than the PMF or it is **not apparent** if the IDF should be the PMF or something less,

THEN . . .

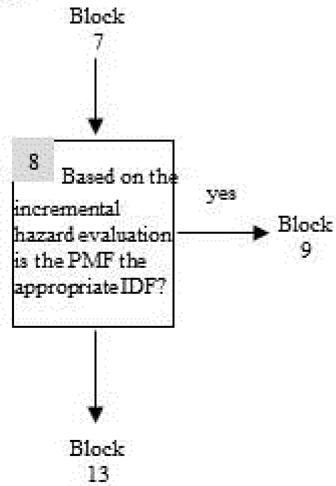
You need to perform an incremental hazard evaluation to determine the appropriate IDF. Performing the incremental hazard evaluation involves:

- Conducting dambreak sensitivity studies,
- Reviewing incremental rises between with-failure and without-failure conditions for a range of flood inflows (see Flowchart 2).
- Selecting the appropriate IDF on the basis of the dambreak studies and incremental impacts on downstream areas.

A procedural flow chart for performing a hazard evaluation appears in Flowchart 2 (Figure 2), followed by an explanation of the procedure.

Continued…

Block 8



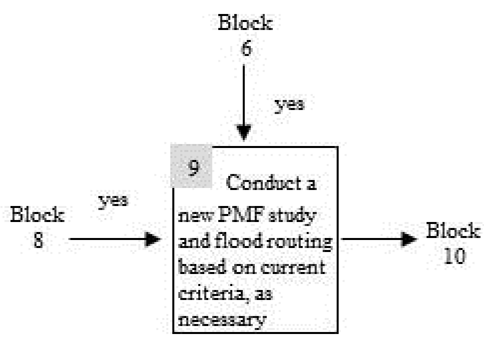
You should use the results of the incremental hazard evaluation and dambreak studies conducted in Block 7 to determine if the PMF is the appropriate IDF.

The IDF should be the PMF when the incremental consequences of failure are unacceptable, regardless of how large the assumed flood inflow becomes.

If your answer is **YES**, continue to Block 9.

If your answer is **NO**, go to Block 13.
In Block 13 you will answer the question: Is the dam safe for the appropriate IDF?

Block 9



IF . . .

In Block 6 you determined that the PMF is obviously the appropriate IDF,

OR . . .

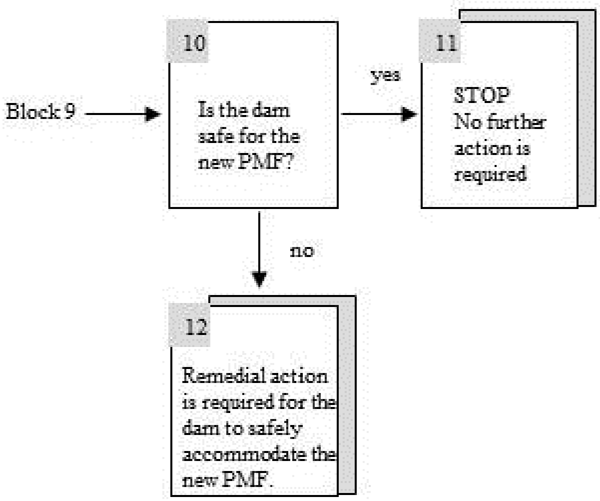
If, based on the incremental hazard evaluation conducted in Block 8, the PMF is the appropriate IDF,

THEN . . .

You should conduct a new PMF study and flood routing based on current criteria, unless it was determined in Block 3 that the original PMF is acceptable under current criteria.

Continued…

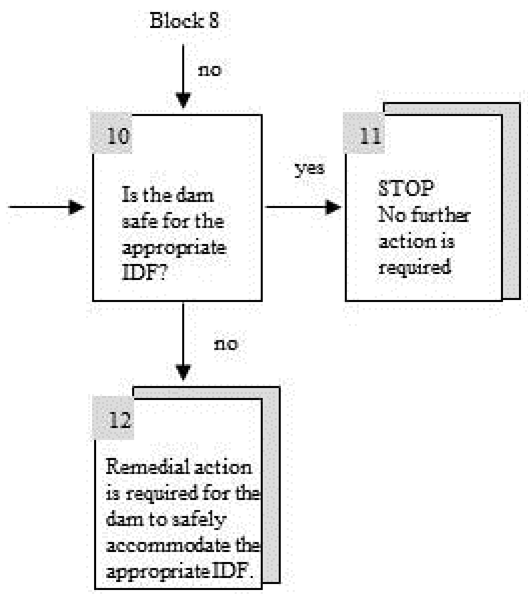
Blocks 10, 11 and 12



Once the new PMF is calculated, you should determine if the dam is safe for the new PMF. If the dam is **SAFE** for the new PMF, no further investigations or remedial actions for hydrologic conditions are required.

If the dam is **NOT SAFE** for the new PMF, remedial action is required for the dam to safely accommodate the PMF.

Blocks 13, 14 and 15



IF . . .

In Block 8 you determined that the PMF is **NOT** the appropriate IDF,

THEN . . .

You need to determine if the dam is safe for the appropriate IDF.

If the dam is **SAFE** for the appropriate IDF, no further investigations or remedial action for the hydrologic conditions are required.

If the dam is **NOT SAFE** for the appropriate IDF, remedial action is required for the dam to safely accommodate the appropriate IDF.

Depending on the type of remedial action considered, it may be necessary to reevaluate the IDF to ensure that the appropriate IDF has been selected for the design of any modification.

Continued…

INTRODUCTION

As stated previously, if the PMF was not used for the original design of a dam, or if the PMF is not the appropriate IDF, an incremental hazard evaluation must be performed to determine the appropriate IDF.

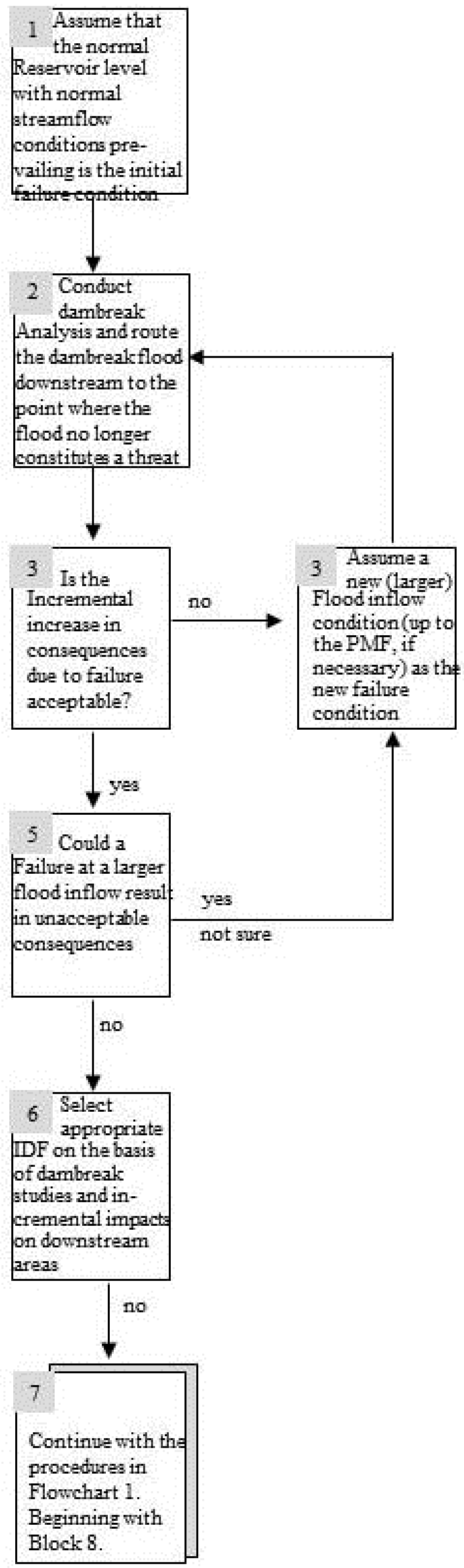
PROCEDURES FOR CONDUCTING AN INCREMENTAL HAZARD EVALUATION

Flowchart 2 in Figure 2 shows the procedures for performing an incremental hazard evaluation. This flowchart is an expansion of Block 7 in Flowchart 1, Figure 1.

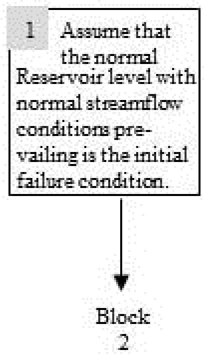
⇨

Flowchart 2 is on the next page.

FIGURE 2. FLOWCHART 2 – PROCEDURES FOR CONDUCTING AN INCREMENTAL HAZARD EVALUATION

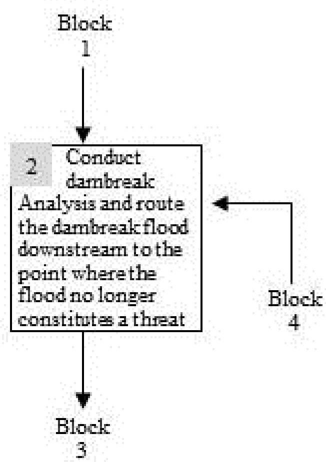


Block 1



Assume that the normal reservoir level with normal streamflow conditions prevailing is the initial failure condition. Starting at this point will ensure that the full range of flood inflow conditions will be investigated and will include the "sunny day" failure condition. It will also assist in verifying the initial hazard rating assigned to the dam. Using the normal maximum water surface level as the initial condition is particularly important if the initial hazard rating was low.

Block 2

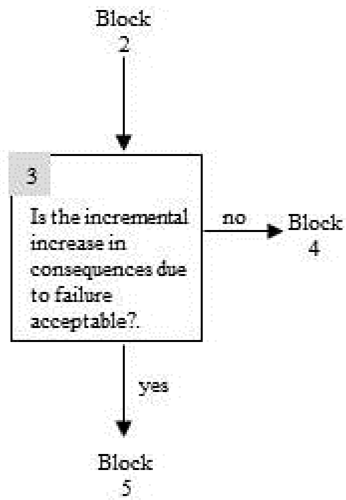


Next, conduct dambreak sensitivity studies (of various breach parameters) and route the dambreak flood to the point downstream where it no longer constitutes a threat to downstream life and property.

It is important to remember that the incremental increases should address the differences between the non-failure condition with the dam remaining in place and the failure condition. Also, the dam should not be assumed to fail until the peak reservoir water surface elevation is attained for the assumed flood inflow condition being analyzed. Dams should be assumed to fail as described in Chapter II of the Engineering Guidelines.

Continued…

Block 3

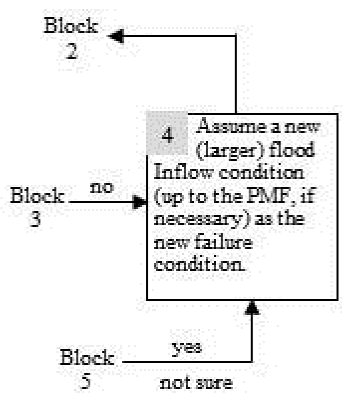


Now, determine if the additional increase in consequences due to failure is acceptable. Answering this question is critical in the incremental hazard evaluation and doing so involves an estimate of loss of life and property with and without dam failure.

If the consequences of failure under the assumed flood flow conditions are **NOT ACCEPTABLE**, go to Block 4.

If the consequences of failure **ARE ACCEPTABLE**, continue to Block 5.

Block 4



IF. . .

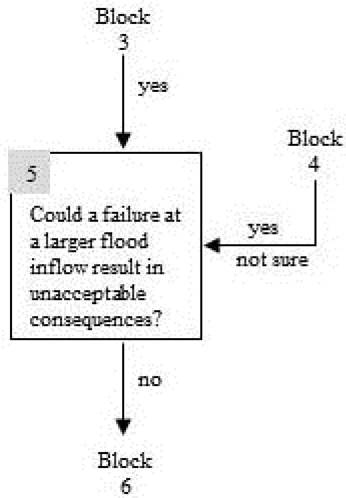
In Block 3 it was determined that the consequences of failure under the assumed flood flow conditions are **NOT ACCEPTABLE,**

THEN . . .

Assume a new (larger flood inflow condition (e.g., some percentage of the PMF) and perform a new dambreak analysis (see Block 2). This procedure should be repeated until an acceptable level of flooding is identified, or the full PMF has been reached.

Continued…

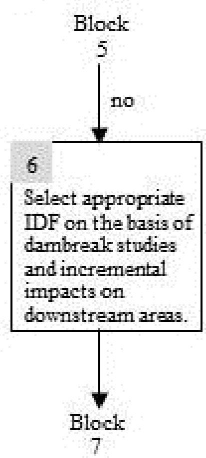
Block 5



IF ...
In Block 3 you determined that the consequences of failure under the assumed flood flow conditions are **ACCEPTABLE**,
i.e., failure of the dam under “sunny day” conditions was insignificant,

THEN ...
Determine if failure at a larger flood inflow condition will result in unacceptable consequences. This question is very important. For example, situations exist where a failure during normal water surface conditions results in the flood wave being contained completely within the banks of a river and obviously would not cause a threat to life and property downstream. However, under some flood flow conditions, the natural river flows may go out-of-bank, and a failure on top of that flood condition will result in an additional threat to downstream life and property.

Block 6



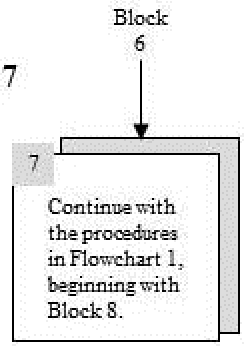
If failure at another flood level will result in **UNACCEPTABLE** consequences, or if you are **NOT SURE**, return to Block 4. Assume larger flood inflow conditions and perform new dam break studies. This procedure should be repeated to determine the acceptable level of flooding.

If failure at another flood level will **NOT** result in unacceptable consequences, continue to Block 6.

You should now select the appropriate IDF based on the results of dambreak studies and incremental impacts on downstream areas.

Continued…

Block 7



Continue this process with the steps in Flowchart 1, Figure 1, starting with Block 8. In Block 8 you will answer the question: Based on the incremental hazard evaluation, is the PMF the appropriate IDF?

Continued…