Design Standards No. 13

Embankment Dams

Chapter 6: Freeboard
Phase 4 Final

U.S. Department of the Interior

June 2021

Mission Statements

The U.S. Department of the Interior protects and manages the Nation's natural resources and cultural heritage; provides scientific and other information about those resources; and honors its trust responsibilities or special commitments to American Indians, Alaska Natives, and affiliated Island Communities.

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

Design Standards Signature Sheet

Design Standards No. 13

Embankment Dams

DS-13(6)-2.1: Phase 4 Final

June 2021
Chapter 6: Freeboard

Revision Number DS-13(6)-2.1

Summary of revisions:

This standard was revised in June 2021 to address the following:

- Figures 6.2.2-1 and B-1 were updated to address inconsistencies identified with the original figures.
- Additional equations, information, and example computations are provided in appendices B and C for calculating exceedance values of wave runup other than the 2-percent exceedance value.
- A reference to section 7.3.3 of DS-13(7)-2.1 has been added in section B.3.2 in appendix B to direct the reader to the section in Reclamation's Riprap Slope Protection design standard that outlines how various exceedance values of wind speeds are developed from weather station data.
- Minor editorial and grammatical changes have been made.

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Foreword

Purpose

The Bureau of Reclamation (Reclamation) design standards present technical requirements and processes to enable design professionals to prepare design documents and reports necessary to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public. Compliance with these design standards assists in the development and improvement of Reclamation facilities in a way that protects the public's health, safety, and welfare; recognizes needs of all stakeholders; and achieves lasting value and functionality necessary for Reclamation facilities. Responsible designers accomplish this goal through compliance with these design standards and all other applicable technical codes, as well as incorporation of the stakeholders' vision and values, that are then reflected in the constructed facilities.

Application of Design Standards

Reclamation design activities, whether performed by Reclamation or by a non-Reclamation entity, must be performed in accordance with established Reclamation design criteria and standards, and approved national design standards, if applicable. Exceptions to this requirement shall be in accordance with provisions of *Reclamation Manual Policy*, Performing Design and Construction Activities, FAC P03.

In addition to these design standards, designers shall integrate sound engineering judgment, applicable national codes and design standards, site-specific technical considerations, and project-specific considerations to ensure suitable designs are produced that protect the public's investment and safety. Designers shall use the most current edition of national codes and design standards consistent with Reclamation design standards. Reclamation design standards may include exceptions to requirements of national codes and design standards.

Proposed Revisions

Reclamation designers should inform the Technical Service Center (TSC), via Reclamation's Design Standards Website notification procedure, of any recommended updates or changes to Reclamation design standards to meet current and/or improved design practices.

Chapter Signature Sheet
Bureau of Reclamation
Technical Service Center

Design Standards No. 13

Embankment Dams

Chapter 6: Freeboard

DS-13(6)-2:[1] Phase 4 Final
June 2013

Chapter 6 – Freeboard Criteria and Guidelines is a chapter within Design Standards No. 13 and was developed to provide:

- A consistent approach to the derivation of an appropriate amount of normal and minimum freeboard to protect an embankment dam primarily from overtopping due to wind-generated waves and reservoir setup

- Relationships in agreement with current research and other Federal agencies for the computations of wave characteristics, runup, and setup

Other factors that could affect freeboard for an embankment dam are presented for consideration, but these are discussed in less detail, and additional sources of information are referenced.

- This chapter supersedes the Assistant Commissioner – Engineering and Research (ACER) Technical Memorandum (TM) No. 2 [15] for freeboard analysis, as it pertains to embankment dams. The update centers around the most current research by the U.S. Army Corps of Engineers in the 2008 and 2011 publications of the Coastal Engineering Manual, EM-1110-2-1100 [16a and 16b]. This update also changes the preferred method of analysis to one that is simpler than the complicated analysis using the conditional probabilities of multiple reservoir levels and wind velocities that was included in the last version of this design standard and ACER TM 2.

[1] DS-13(6)-2.1 refers to Design Standards No. 13, chapter 6, revision 2.

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Freeboard

6.1 Introduction

6.1.1 Purpose

The purpose of this design standard chapter is to convey consistent approaches to the analysis and design of freeboard to protect an embankment dam from overtopping primarily as a result of wind-generated wave runup and reservoir setup. Freeboard for embankment dams should include prevention of any overtopping of the dam by either frequent or infrequent high waves that might interfere with efficient operation of the project, create conditions hazardous to personnel, cause other adverse effects not necessarily associated with the general safety of the structure, or cause a dam breach and failure. This, however, does not necessarily require total prevention of splashover or spray by occasional waves under full surcharge and extreme conditions, but does require that such occurrences will be of such magnitude and duration as to not threaten the safety of the dam. Also, increased freeboard can be used to help mitigate security concerns at a dam, but this application is not covered in detail in this chapter.

6.1.2 Deviations from Standard

Analysis and design of freeboard for embankment dams within the Bureau of Reclamation should adhere to concepts and methodologies presented in this design standard. Rationale for deviation from the standard should be presented in technical documentation for the dam and should be approved by appropriate line supervisors and managers.

6.1.3 Revisions of Standard

This standard will be revised periodically as its use and the state of practice suggests. Comments and/or suggested revisions should be sent to the Bureau of Reclamation, Technical Service Center, Attn: 86-68300, Denver, CO 80225.

6.1.4 Scope

This chapter is primarily concerned with the establishment of appropriate freeboard to minimize the potential for dam overtopping and failure from wind-generated wave action. Reservoir fetch[1] and wind velocity analyses are presented. Methods for determining the minimum, intermediate, and normal freeboard are given. Table 6.1.4-1 shows the calculations that are usually performed to derive the minimum and normal freeboard for new dams or check the adequacy of the minimum and normal freeboard for existing dams.

Wave runup and reservoir setup caused by wind shear over the reservoir water surface are the predominant factors for embankment dam freeboard discussed in this chapter. Other factors, besides wind loading, are presented in this chapter for additional freeboard considerations, but are not covered in detail. Other freeboard considerations, including flood loading and related appurtenant works operations, are provided in chapter 2 of Design Standard No. 14 [19]. Risk analysis related to flood loadings, overtopping, and freeboard is covered in the *Dam Safety Risk Analysis Best Practices Training Manual* [22]. Security concerns should also be considered whenever freeboard is being determined for a dam. Reclamation's Security, Safety, and Law Enforcement (SSLE) office can offer guidance in this type of analysis.

Table 6.1.4-1 Freeboard Calculations

Type of Freeboard[1]	Approach to Freeboard Analysis	
Minimum[2] (section 6.2.1)	Select a design crest elevation the higher of:	
	MRWS + 3 feet	MRWS + runup and setup from a wind velocity exceeded 10% of the time
Normal[3] (section 6.2.2)	NRWS + runup and setup from a 100-mile-per-hour wind velocity	
Checks[3] (section 6.3.1(2))	Design crest elevation is adequate for runup and setup during the IDF when reservoir is within 2 feet of the MRWS	
	Design crest elevation is adequate for runup and setup during the IDF when reservoir is within 4 feet of the MRWS	

[1] For new dams, perform all types of freeboard calculations and checks listed including, if necessary, calculations for intermediate freeboard (which is not shown in the table).
[2] Since security concerns are associated with elevated reservoir levels, freeboard for security may be incorporated into the minimum freeboard amount.
[3] For existing dams, the adequacy of normal freeboard is checked according to the approach described in section 6.2.2, and the adequacy of minimum freeboard is checked according to the approach described in section 6.3.1(2).
Note: MRWS = maximum reservoir water surface, NRWS = normal reservoir water surface, IDF = inflow design flood.

6.1.5 Applicability

The guidance and procedures in this chapter are applicable to the analysis of freeboard for new and existing embankment dams and dikes.

[1] All terms are defined in appendix A.

6.1.6 General

Freeboard is most commonly thought of as the distance from a reservoir water surface to the top (design crest elevation) of the dam.[2] Even though the top of the impervious core of some embankment dams may be a small distance short of the design crest elevation, the fill above the core is likely stable enough to retain a high reservoir for a short time and is usually considered part of the freeboard. However, this may not be true if a flood raised the reservoir water surface over the core for a long time. Risk analysis for existing dams can account for this situation by developing a “fragility curve” correlating the probability of failure to the amount of freeboard (negative if the dam is overtopped). An example of such a curve is given in section 6.3.2. Details on the development of fragility curves can be found in the *Dam Safety Risk Analysis Best Practices Training Manual* [22]. Camber built into a dam to accommodate settlement is not part of freeboard. Parapet walls are sometimes used to provide freeboard for wave runup and setup, but generally not for flood storage. Additional freeboard can also be included above high reservoir levels for security to thwart adversarial attacks to the dam crest, but this special type of analysis is not covered in this chapter.

Experience has shown that embankment dams with large reservoirs and long fetches can be subject to the buildup of very large waves that could run high up on the upstream slope. Dam crests can be damaged and embankment dams can possibly fail by wave action even before they would be subject to flood overtopping. Some small dams have been damaged as well. Splash and spray over a dam crest is not uncommon when high velocity winds occur over high reservoir water surfaces. Hurricanes can produce this damaging combination because high velocity winds can persist through the peak reservoir levels. Wind and wave action have long been considered significant factors in the determination of the design crest elevation (not including camber) of an embankment dam and the analysis of all types of freeboard.

Wind-generated wave heights and wave runup are probably the most thoroughly studied and understood factors that influence freeboard. Much of the study has been carried out and reported by the U.S. Army Corps of Engineers [1, 9, 11, 13, 16]. Wave generation is influenced by wind characteristics such as velocity, duration, and orientation with respect to the reservoir; by topographic configuration of the reservoir, including depth and shoaling effects; and by fetch. Fetch accounts for the effects of the length of the open-water approach of the waves. Wave runup is governed by the height and steepness of the waves; by the slope, roughness, and porosity of the dam face; by changes in the slope of the dam face; and by the presence of berms on the dam face. Setup is caused by the shearing effect of the wind that tends to tilt the reservoir higher in the direction of the wind. Appendix B shows the derivation of fetch and contains equations for the significant wave height, wind setup, and wave runup.

[2] Appendix A defines key terms associated with freeboard.

- The wind data that the Bureau of Reclamation uses in freeboard and riprap analyses was compiled by the Battelle National Laboratories for wind energy purposes from wind data of the 1950s, 1960s, and 1970s divided into various regions covering all of the lower 48 States [14]. Wind velocity persistence data from this source are processed into hourly probability relationships in a Reclamation computer program called PFARA [17]. The program generates a site-specific curve of hourly probability of wind (P_{W_H}) versus wind velocity (VMPH) for use in analysis. Appendix B discusses this in more depth.

Freeboard analysis is usually different for new dams than for existing dams. Unless otherwise stated, the criteria given in this standard are for new embankment dams. There is a section specifically devoted to existing dams later in this standard (starting at paragraph 6.3.2).

Selection of the inflow design flood (IDF) is not within the scope of this design standard. The standard addressing the selection of the IDF is Design Standards No. 14, chapter 2 [19]. Hydrologic loads for a risk analysis are discussed in chapter 3 of the *Dam Safety Risk Analysis Best Practices Training Manual* [22], while chapter 11 covers risk analysis for flood overtopping failure of dams. These topics are touched upon, but not covered in detail in this chapter since they are addressed in full detail in these other references. Freeboard for embankment dams may consider risk as well as a variety of other factors. This chapter defines many of these other factors and provides guidance on how they should be incorporated into freeboard. Wind-generated wave runup and setup are two primary factors in a freeboard analysis that are described in this chapter. These factors should also be included in a hydrologic risk analysis.

Another function of "freeboard" is to ensure that there is a sufficient amount of embankment above the reservoir level to resist an overtopping failure should slope failures and crest loss occur due to seismic loading. For some new and existing dams, the seismic issues may dictate the selection of freeboard more than wave action. Consideration of freeboard related to seismic deformations is not addressed in this standard; rather it is covered in Design Standard No. 13, Chapter 13, "Seismic Design and Analysis."

6.2 Freeboard Criteria for New Dams

6.2.1 Freeboard Criteria at Maximum Reservoir Water Surface Elevation – "Minimum Freeboard"

The maximum reservoir water surface (MRWS) is the expected reservoir water level when the IDF is routed through the reservoir and dam (as intended by design) in accordance with the Standing Operating Procedures. At Reclamation, the IDF can be the probable maximum flood (PMF) or a flood corresponding to a long return period. The PMF is usually used for new dams. More discussion on the hydrology of dams, the selection of the IDF, and some other factors that could affect freeboard besides wind loads can be found in Design Standards No. 14, Chapter 2, "Hydrologic Considerations" [19].

When the reservoir is at the MRWS, the minimum freeboard to prevent embankment dam overtopping due to wind loads should be the greater of: (1) 3 feet or (2) the sum of the setup and runup that would be generated by the typical winds that would be expected to occur during large floods (i.e., the IDF, the PMF, or floods greater than the 1,000-year event, whichever is associated with the design MRWS).

The first criterion of 3 feet is Reclamation's historic standard for embankment dams. To some degree, it accounts for uncertainty in wind loadings as well as possible changes in both wind and hydrologic loadings through the life of the dam. The standard of 3 feet should be increased when either there is significant uncertainty in these loadings or other freeboard factors, or when the dam poses an unusually high risk of failure due to overtopping. The first criterion adds robustness to the freeboard component of a dam design as a way to address the uncertainty.

For the second criterion, a typical wind is appropriate for most cases in which the reservoir or watershed is very large in comparison to the size of the storm, there is significant lag time between the passing of the storm and the maximum rise of the reservoir, and the wind events that occur when the water surface is near maximum are not related to the storm that created the flood. Thus, part of the requirements for minimum freeboard include the consideration that at least some sort of typical wind may be blowing while the reservoir water surface is at maximum during the IDF. Ordinarily, the lowest typical wind velocity is that which has an hourly exceedance probability equal to 10 percent (P_{W_H} = 0.1). The smallest minimum freeboard is computed by adding the runup and setup caused by this wind event.

If winds that occur during the IDF when the reservoir is at the MRWS are not independent of the hydrologic event (for example, with a hurricane-related storm event or when a flood occurs over a small watershed), then local authorities or meteorologists should be consulted to estimate the wind velocities that would occur at this time. Due to uncertainty that cannot be avoided for such an estimate, a freeboard sensitivity analysis can be performed, incorporating a variety of winds that may occur.

Since security concerns relative to attacks on a dam crest may be associated with high reservoir levels, freeboard for security may be incorporated into the minimum freeboard. Freeboard for security would not likely be added to the very highest anticipated reservoir level (i.e., the MRWS, as is the case for minimum freeboard). For security, a more common "elevated" reservoir water surface (i.e., "intermediate;" see section 6.2.3) may be more reasonable from which to determine an adequate amount of freeboard. It is probably not necessary to add a freeboard amount for security concerns on top of the minimum freeboard required for wind and other factors; rather, the freeboard required for security can be included as all or part of the freeboard required for wind and other factors. For guidance on freeboard analysis and design to address security concerns, consult Reclamation's SSLE office.

6.2.2 Normal Reservoir Water Surface Freeboard Criteria – "Normal Freeboard"

When the reservoir is at the normal reservoir water surface (NRWS) elevation, top of joint-use capacity, or top of active conservation capacity, a normal freeboard should be specified that protects the dam against wind-generated waves that would occur due to the highest sustained velocity winds at the site. The elevation on the dam considered to be the "normal" reservoir water surface is the highest within the typical range of annual operations (not including flood operations). It is Reclamation's standard for embankment dams to apply a wind velocity of 100 miles per hour (mi/h) over the NRWS to derive the required normal freeboard. Figure 6.2.2-1 can be used for determining the wind setup and wave runup resulting from a 100-mi/h sustained wind velocity. This figure is based on the equations of appendix B derived by the U.S. Army Corps of Engineers [16b]. The 2 percent associated with runup is a probability of exceedance that is to be used for design to minimize the number of overtopping waves.

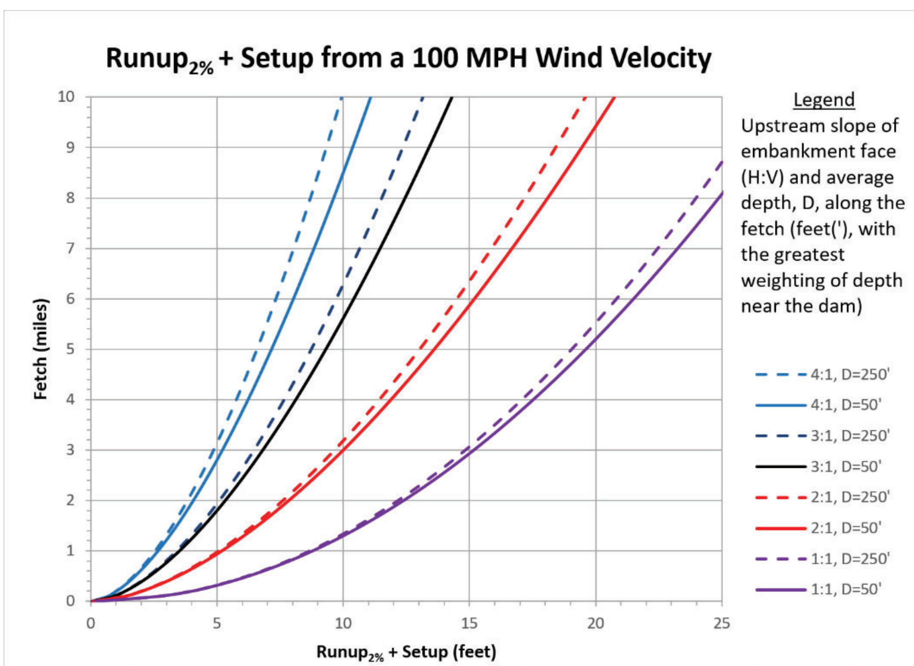

Figure 6.2.2-1. Runup$_{2\%}$ + setup from a 100-mi/h sustained wind velocity on a surface protected by riprap. (Reproduce in color.)

6.2.3 Intermediate Water Surface Freeboard Criteria

For some dams, operational situations, or reservoir allocations, when the reservoir is at an intermediate elevation (i.e., an elevation between the MRWS and the NRWS or top of joint-use or active conservation capacity), an intermediate freeboard requirement may need to be determined to provide an acceptably remote probability of being exceeded by any combination of wind-generated waves and water surfaces occurring simultaneously. This is a particularly important consideration if reservoir water levels are expected to frequently exceed the NRWS or are often near the MRWS and maintain the high levels for an extended period of time. Dams that are operated to utilize flood storage frequently may need an intermediate freeboard analysis. In other words, for dams where reservoir water surfaces are frequently above the NRWS and near the MRWS level for long periods of time, an intermediate freeboard analysis should be considered.

To compute an amount of intermediate freeboard required above a particular water surface elevation between the NRWS and MRWS, a wind velocity first needs to be selected from the site-specific wind probability curve (see section 6.2.2 and appendix B). The wind velocity is taken from the wind probability curve at a wind probability equal to an acceptable overall design probability divided by the probability of occurrence of the intermediate water surface elevation being analyzed. The wind velocity is used to compute wind setup and wave runup with the equations from appendix B of this chapter. The intermediate freeboard for the particular water surface elevation is the sum of the wind setup and wave runup. This analysis should be iterative, as it may be done for numerous intermediate reservoir levels that are expected to occur frequently for long periods of time, especially if such levels are near the MRWS.

A conservative approach to derive an intermediate freeboard for a frequently high reservoir level above the NRWS is to select the intermediate freeboard to be greater than the wave runup and wind setup generated by a 100-mi/hr wind velocity over the considered reservoir elevation. Figure 6.2.2-1 can be used for this determination. This may result in the application of the freeboard requirement for the NRWS to reservoir elevations near the MRWS, thus necessitating more freeboard than would otherwise be required under just the normal or minimum freeboard computations as described in the previous two sections of this chapter.

6.2.4 Deviations from the Above Criteria

While at least 3 feet of freeboard above the MRWS is the minimum amount of freeboard for new embankment dams recommended in this design standard (see section 6.2.1, above), deviations on this have been accepted, depending on particular hydrologic, operational, or dam circumstances. For instance, offstream storage dams without spillways may require more than 3 feet of minimum freeboard to incorporate some added safety or conservatism in the event of misoperation of the outlet works or feeder canal inflows [18]. Some dams could be designed so that they are quite resistant to overtopping flows and are expected to perform well with less freeboard. Freeboard evaluation for existing dams, as discussed below, would likely be performed in a risk context and may not be so closely tied to the PMF. Further discussion of special considerations is given below. As with any deviations from a design standard, the considerations justifying the deviations need to be well documented and approved.

6.3 Scope of Freeboard Analysis

6.3.1 New Embankment Dams

The recommended approach to perform a freeboard analysis for a new embankment dam is to start by choosing a design crest elevation (without camber) and then check to see if it would satisfy the minimum and normal freeboard requirements. The design crest elevation is selected as the higher of either: (1) the MRWS elevation plus 3 feet or (2) the MRWS plus the runup and setup that would be generated by a wind with a 10-percent probability of exceedance. Allowances for camber and additional factors, such as security, are added (or incorporated into the freeboard) after the design crest elevation is deemed satisfactory.

Two relatively conservative checks are performed to determine if the design crest elevation is sufficient to prevent wind-generated wave overtopping:

(1) The first check is used to see if the design crest elevation is high enough to protect the dam from overtopping should waves build up from a 100-mi/h wind while the reservoir is operating normally below flood storage. Given the fetch of the reservoir at the NRWS (to compute fetch, see appendix B, figure B-1), the slope of the upstream face of the dam, and the average depth along the fetch (giving greater weight to depths near the dam), use figure 6.2.2-1 or the equations in appendix B to derive the amount of runup and setup that can be expected from a sustained 100-mi/h wind event. If the vertical distance between the normal reservoir water surface and the design crest elevation of the dam (this distance includes the minimum freeboard) is greater than the vertical distance of runup and setup determined from figure 6.2.2-1, then the dam design passes this check, and the design crest elevation is adequately high to protect the dam from waves generated by 100-mi/h winds when the reservoir is not in flood stage.

(2) The second check is used to see if the design crest elevation is high enough to protect the dam from overtopping in the event of the IDF and winds that could be expected during the IDF event (i.e., during time periods equal to durations that the reservoir water surface is near maximum). Necessary information for this includes:

- Reservoir elevation versus time graph derived from routing the IDF, as shown in figure 6.3.1-1

- Hourly probability of the wind (P_{W_H}) versus wind velocity (VMPH) derived from the analysis of wind data (from the computer program PFARA [17]; see appendix B)

Horizontal lines are drawn across the reservoir elevation versus time graph, at 2 and 4 feet below the MRWS as shown in figure 6.3.1-1. The duration that the reservoir is within 2 feet of the MRWS is equal to the length of the upper horizontal line below the flood curve. The inverse of this duration can be assumed to be the hourly exceedance probability of the largest wind event that could be expected while the reservoir is within 2 feet of the MRWS. A wind velocity is taken from the P_{W_H} versus wind velocity curve. With equations from appendix B, compute the runup and setup that would be generated by this wind velocity. If this is less than 5 feet, then 3 feet of minimum freeboard is adequate to protect the dam from this wind event when the reservoir is 5 feet below the crest. Similarly, the duration represented by the horizontal line drawn 4 feet below the MRWS on the reservoir elevation versus time graph is used to find the P_{W_H} and maximum wind velocity that could be expected while the reservoir is 4 feet from the MRWS. The inverse of this duration can be assumed to be the hourly probability of the largest wind event that could be expected while the reservoir is within 7 feet of the dam crest. Again, a wind velocity is taken from the P_{W_H} versus wind velocity curve, and runup and setup are computed with this wind velocity using equations in appendix B. If the runup and setup generated by this wind velocity are less than 7 feet, then 3 feet of minimum freeboard is adequate to protect the dam from this wind event when the reservoir is 7 feet below the crest. The second check is satisfied if the runup and setup are computed to be less than 5 and 7 feet as described above.

If the dam design passes both of the above checks, then no other method needs to be used to calculate freeboard requirements due to wind-generated waves. If there is no reason to believe that exceptionally high wind velocities (those not necessarily expected) would be blowing while the water surface is within 4 feet of the MRWS during the largest flood events, these two checks are sufficient for the freeboard analysis pertaining to the factor of wind.

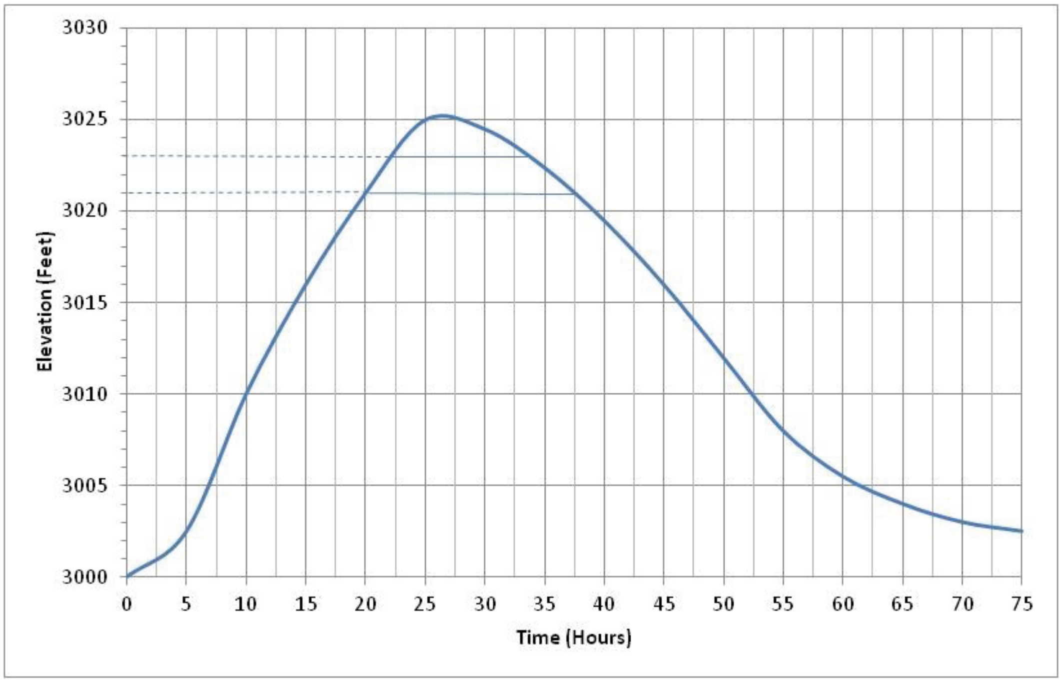

Figure 6.3.1-1. Reservoir versus time for IDF routing.

If the dam design fails either of the above two checks, the design crest elevation should be raised iteratively until the two checks are both satisfied.

The evaluator may need to consider the possibility of winds and floods occurring that are even more severe than those used in the analysis. If the dam is located in an area where extremely high winds may occur when the reservoir reaches its maximum during the largest flood events, then this association should be further developed and included in the freeboard design. A qualified meteorologist should be consulted to quantify these wind velocities, or an additional amount of freeboard could be added to the minimum freeboard to account for such extreme possibilities.

6.3.1.1 Top of Impervious Zone

When a reservoir is expected to be sustained high on a dam for a long time due to a long-duration flood or some other reason, the top of the core or impervious zone should be high enough to prevent seepage or internal erosion from passing uncontrolled or unfiltered through the dam. In such a case, the top of the impervious zone should be designed so that, after settlement, it is at or above the elevation of the MRWS plus wind setup (but not necessarily runup) from winds associated with the largest flood events or typical winds of not less than 10-percent exceedance probability, whichever is greater. Similarly, if the top of the impervious zone could be subjected to frost action or desiccation cracking, zoning of new dams must include filters to control leakage through cracks or frost lens separations, or the reservoir water surface must be kept below the depth of such effects. The risk of these conditions is balanced with the cost of the core materials and placement in selecting the top of the impervious zone.

6.3.2 Existing Embankment Dams

6.3.2.1 Evaluation of Existing Freeboard

A freeboard analysis for an existing dam attempts to identify hydrologic or hydraulic deficiencies that might lead to failure of the dam. If the MRWS of the reservoir is close enough to the dam crest such that wind-generated runup and setup would wash over, or if the MRWS is higher than the existing crest, then the following factors should be considered to evaluate the potential of this high-water condition to cause failure of the embankment:

- **Crest Elevations, Width, and Slope**.—When overtopping is a potential concern, low spots concentrate flow and thus are far more likely to lead to erosion as compared to a dam that has a uniform elevation and sheet flow during overtopping. Usually, the two lowest crest areas on an embankment dam are at the two ends where the camber is least. A crest survey should be performed to determine actual crest elevations and the existence of low spots. A wide crest or a crest that slopes toward the reservoir also tends to reduce the erosion potential during overtopping. Also, it is important to be aware of the possibility of dam failure due to sustained reservoir water levels below the crest elevation if the upper portion of the dam is highly permeable or may be cracked (i.e., the top of the impermeable zone is below the crest).

- **Crest and Downstream Slope Face Materials**.—In general, well-graded, dense, impermeable cohesive soil without a significant amount of coarse-grained material is the type of material at the top (of the core or crest) of an embankment dam that is most resistant to erosion in the event of overtopping. A paved road surface is quite beneficial in terms of reducing the potential for failure during overtopping except that the velocity of the overflow may increase across a paved crest such that more erosion takes place off of the downstream edge of the pavement. An unpaved gravel road can also be beneficial because traffic and road base binder will densify the crest material to the point where, even though it is coarse and permeable, it still may be quite resistant to erosion. Erosion from embankment overtopping will most likely begin on the downstream slope, particularly at the downstream embankment toe where the overtopping flow is quite turbulent. An assessment of an embankment's ability to withstand overtopping should begin near the toe of the downstream slope and along the downstream groins of the dam or at locations where the downstream slope of the dam changes.

- **Vegetation**.—In general, vegetation will act to inhibit erosion unless trees and other obstructions cause turbulence or a flow concentration.

- **Permeability of Surface Materials**.—While a permeable, unsaturated surface would impede wave runup, more seepage would flow through the dam and could initiate erosion of embankment materials when the reservoir is high. When combined with overtopping, the embankment materials can become buoyant, and erosion is accelerated by both seepage forces from the flow through and tractive forces from the flow over the dam.

- **Overall Condition of the Structure**.—The historical record of a dam's ability to resist erosion due to prior overtopping, heavy rainfall, or extremely severe wave action may provide some insight into the expected performance of the dam during the PMF. Dams that are in the same area or built out of the same material as those that have overtopped can also be informative. Natural or manmade exposures of the dam material may provide evidence of the erodibility of the material. A structure that has existed for a long time without any apparent erosion of the crest, downstream face, or toe may endure overtopping better than one that has degraded under natural weathering.

- **Depth, Velocity, and Duration of Overtopping**.—The deeper, faster, and longer that water flows over a dam, the more likely the possibility of failure. Empirical relationships are available to estimate threshold failure conditions where erosion would initiate during overtopping for an embankment dam. The critical depth of overtopping is derived by comparing the tractive shear stress and velocity of the overflow to permissible values. The permissible values are dependent on the roughness, slope angle, and type of material on the downstream face [23]. The duration of overtopping is accounted for by comparing both the depth and the volume of overflow water for selected duration intervals to permissible values for various surface roughness. The time at which failure is initiated is the time during the flood when the critical depth of overtopping is reached.

- **Wind**.—The two checks that are given in section 6.3.1 can be used to verify if there is enough freeboard to prevent overtopping caused by wind-generated waves. The first check uses figure 6.2.2-1 with the fetch of the reservoir at the normal reservoir water surface (to compute fetch, see appendix B) and the slope of the upstream face of the dam to derive the amount of runup and setup that can be expected from a 100-mi/h wind event. If the vertical distance between the normal reservoir water surface and the top of the dam is greater than the vertical component of runup and setup that is determined from figure 6.2.2-1, then wind-generated waves are not expected to wash over the dam crest during normal reservoir operation. For the second check, horizontal lines are drawn across the IDF reservoir elevation versus time curve at elevations 2 and 4 feet below the MRWS elevation, as shown in figure 6.3.1-1, to derive the maximum durations that the reservoir is at or above those elevations. The inverse of the duration can be considered to be the hourly probability of the wind event that correlates to the wind velocity on the P_{W_H} versus wind velocity curve. If it is determined that waves may wash over the crest even though the maximum reservoir water surface is below the dam crest, then factors that are included in this section should be considered to determine if these waves would reach the downstream edge of the crest and if these waves would possibly cause erosion. Waves generated by winds over a reservoir water surface above the dam crest will act to increase the depth of overtopping equal to one-half the wave height. Relationships based on soil material erosion and transportation can be used to study the potential for the development of dam breaching in both cases. Such a discussion is beyond the scope of this chapter, but the reader is referred to references 23, 24, 25, and 26 for more information.

- **Security**.—Attacks to the crest of a dam by adversarial entities intending to purposefully fail the dam by breaching the crest may need to be considered if the dam is vulnerable, the reservoir is often high, or the dam is an attractive target. Security issues may need to be considered when evaluating existing freeboard or determining new freeboard. Reclamation's SSLE office should be consulted in this type of analyses.

A Safety of Dams evaluation may be performed when it is decided that insufficient freeboard could lead to dam failure during a flood event. A risk analysis is usually included in the evaluation to estimate the probability of dam failure and consequences of failure by overtopping. Runup and setup should be factored into estimating the probability of overtopping. While it is common to assume that an embankment dam would fail as a result of any amount of overtopping, the intermittent attack from wave action and the likelihood that any of the above factors increases or decreases the probability of failure could be considered in a risk context. Fragility curves are usually developed in risk analyses to estimate the probability of failure given overtopping or significant loss of freeboard, to account for the likelihood of the pertinent factors bulleted above, and the intermittent nature of wave action (runup and setup). An example fragility curve that has been used in risk analyses (this curve is dam/site specific and not necessarily applicable for a risk analysis of any dam) is shown in figure 6.3.2.1-1.

The evaluation of existing dams also needs to take into account conditions that may have changed since the initial freeboard design determination. For example, the risk of malfunction of the spillway and outlet works should be better known than at the time of original design due to maintenance and operating experience.

When assessing the risk of malfunction, known limitations to gate operation should be considered, as well as improvements in mechanical and electrical features or added provisions for skilled attendance during periods of operation. Because foundation and embankment settlement are likely to have occurred, the addition of a parapet wall may be a feasible method of providing freeboard in some existing embankment dam cases.

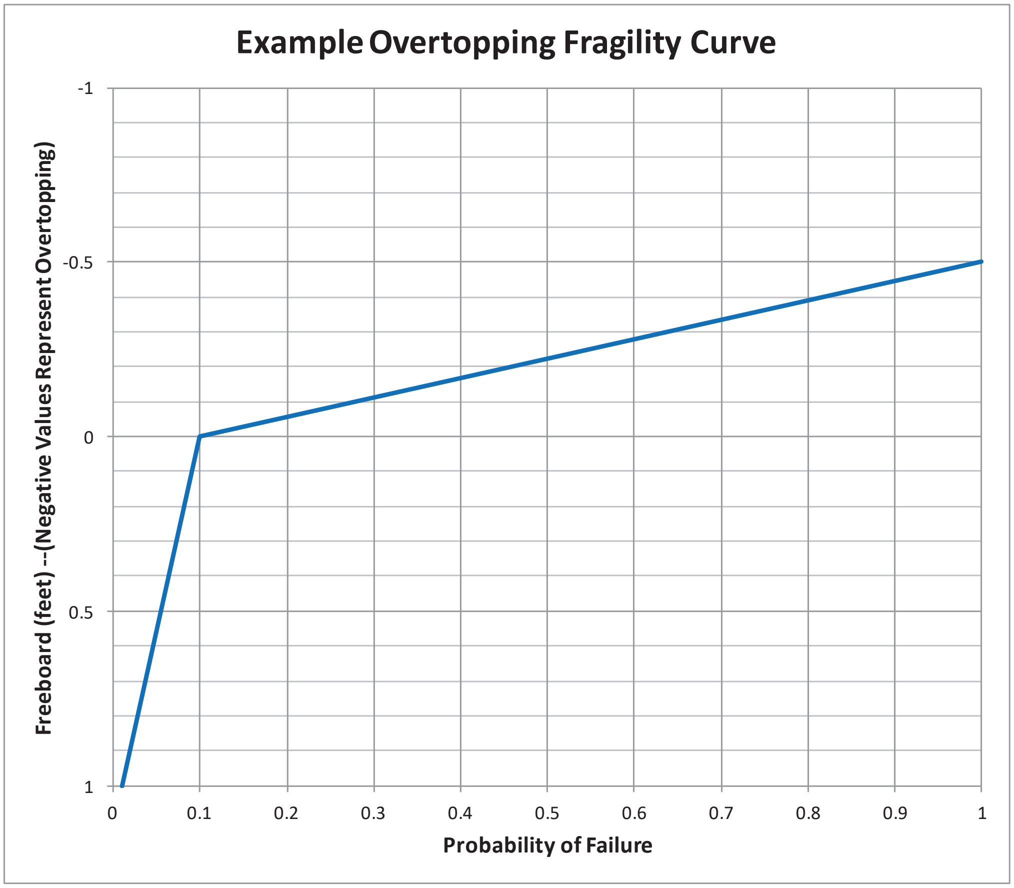

Figure 6.3.2.1-1. Example of an embankment dam fragility curve for hydrologic risk analyses.

Human intervention to avert dam failure can also be incorporated into a risk analysis, but this should probably be done conservatively since it is difficult to predict the occurrence of such things during an extreme hydrologic event. Refer to the *Dam Safety Risk Analysis Best Practices Training Manual*, Chapter 11, “Flood Overtopping Failure of Dams” [22], for a more detailed discussion on hydrologic risk analysis.

6.3.2.2 Freeboard Design Considerations for Existing Dams

Previous design standards included the criterion that if an existing dam is to be modified, and freeboard is part of the modification, then the freeboard should be designed according to standards established for a new dam (previous sections). This is often inappropriate because considerations for determining freeboard requirements for an existing dam that is being modified can be different than the requirements for a new dam. The incremental costs for raising a new dam a few feet during design might be minimal compared to the costs for doing the same to an existing dam. Expenses may be much greater for a modification to an existing dam in the areas of design data acquisition, design time, contracting, construction mobilization, and unit price of materials. The option of increasing spillway capacity to provide more freeboard could also be very costly for an existing spillway, while it may have relatively little impact on the cost for a new dam under design.

Dam safety decisions made within a risk context may affect the selection of freeboard for the modification of an existing dam. For an existing dam that is being modified, a robustness study should be conducted to aid in the selection of freeboard. Robustness studies are discussed in Design Standards No. 14, *Appurtenant Structures for Dams (Spillway and Outlet Works)*, Chapter 2, "Hydrologic Considerations," and identify uncertainties related to the maximum reservoir water surface during the IDF [19]. Such studies are used to identify and, if possible, quantify uncertainties in analyses or design. These uncertainties may be related to hydrologic uncertainties, misoperation of spillway gates for controlled spillways, debris blockage, and wind generated waves. Flood routings of the IDF are conducted that assume that adverse conditions occur (i.e., all gates are not functional; debris blocks the spillway entrance), and the maximum reservoir water surface under these various conditions are determined. The probability of many of these factors occurring cannot be easily quantified. A robustness study is a way to examine the range of events or factors and a resulting range of maximum reservoir water surface elevations (either sustained or intermittent) that would be achieved under less than ideal conditions. Sensitivity may also be studied to determine how important some uncertainties are to the final design or to identify which uncertainties could be critical. The results of the robustness and sensitivity studies are then used as part of the justification for recommended freeboard.

Costs for providing various levels of freeboard should also be estimated.

Security concerns may be important, depending on the dam, dam site, hydrology, etc., and may need to be accounted for in freeboard analysis and design.

A design team should make a recommendation for the freeboard to be provided, backed up by a rationale that incorporates the robustness study results, incremental costs of various freeboard levels, sensitivity, and other relevant factors.

6.4 Parapet Walls

Use of parapet walls to provide freeboard allowances for embankment dams may be considered on a case-by-case basis.

Parapet walls are generally vertical walls (usually of reinforced concrete) that are placed into the top of an embankment dam and tied into the abutments and usually the impervious zone of the dam. The top elevation of parapet wall is higher than the top elevation of the embankment dam, and this difference is the increase in freeboard afforded by the wall. The shape of parapet walls is usually tapered, narrowing toward the top. Ordinarily, they are continuous structures, cast in place across the length of the dam crest (i.e., not segmented or comprised of many short walls joined together as is done with highway barriers). Reinforced earth or block retaining walls could also be considered parapet walls.

As a barrier to the forces of water, parapet walls may be weaker than an embankment dam proper. Examples of poorly designed walls became evident in New Orleans during hurricane Katrina of 2005, when many walls that were being used to increase the freeboard on levees failed [31]. The use of parapet walls to increase freeboard for embankment dams should be limited, thoroughly evaluated, and carefully designed. Parapet walls should not be used to store floodwaters or provide freeboard for wind setup for a new dam. Parapet walls have been proposed to enhance security at dams by strategically placing them in locations that direct attackers away from the most sensitive areas of the dam crest.

For modifications of existing dams, parapet walls should only be used to provide freeboard for wave runup, not for wind setup or flood storage. This is because wave runup is an intermittent type of loading, while setup and flood storage are constant loadings which could initiate seepage and erosion problems. In some rare cases, a parapet wall has been designed to retain the setup, or the uppermost flood storage of the extreme flood events. If a parapet wall is to be used within flood storage or the wind setup distance, deviations from this standard are to be documented as described at the beginning of this design standard. Such documentation should include seepage and stability (possibly static, hydraulic, and hydrodynamic) analyses, a risk analysis, a cost analysis, and design information demonstrating a positive connection between the wall and the dam and its abutments.

When parapet walls are used, the following safeguards must be met:

- The ends of the wall must be adequately tied into the impervious zone and the abutments of the embankment dam to avoid excessive seepage or scour beneath the wall. It is not always necessary to embed a wall deep into the dam’s core if the wall can be tied into other embankment materials adjacent to the core that are impervious enough to provide a good seepage barrier, are resistant to erosion, or if the exposure to floodwaters is very short.

- Provide proper zonation around and beneath the parapet, including an adequate tie into the impervious zone, if necessary, to prevent undercutting and erosion.

- Future foundation and embankment settlement that would adversely affect the structural integrity of the parapet wall must be accounted for in construction sequencing or the design of the parapet wall.

- The parapet wall must be designed to withstand hydrostatic and hydrodynamic (wave) loads.

- Drainage off the crest around or through the wall must be provided.

- Joining and sealing the wall units together with each other and each end of the dam shall be accomplished.

- Safety and security must be ensured.

- Maintenance, snow and ice removal, sight lines, and aesthetics issues should be addressed.

If evaluating existing parapet walls, note how the walls are founded and tie into the abutments and the impervious zone of the dam. Some existing parapet walls only extend to the ends of the dam, leaving an opening for floodwaters or wave action to concentrate around the ends of the wall where the camber may be the least, and erode the embankment dam along the groins. This should be corrected.

6.5 Other Factors that Influence Freeboard

Some of the factors that influence the ability of an embankment dam to resist overtopping have been given in section 6.3.2. The emphasis above is on wave runup and reservoir setup caused by wind, but there are other factors that may need to be considered in determining an adequate amount of freeboard that are listed in this section. In some cases, these factors may override the freeboard required for wind loadings. This section is intended to present some of these other factors for consideration, but does not provide a full discussion of each. There are other sources of information for more complete coverage of these factors as referenced below and elsewhere in this chapter. In particular, floods, hydrologic, hydraulic, all of the factors listed below and other factors, especially related to spillways and outlet works, are covered in Design Standards No. 14, chapter 2 [19].

6.5.1 Floods

Flood characteristics such as the shape of the IDF hydrograph, peak inflow, and volume will influence the methods used and computations performed to determine freeboard. The inflow hydrograph, reservoir storage capacity, spillway and outlet works characteristics, and reservoir operations are used to route the floods through the reservoir. Climate and climate changes may need to be considered. Design Standards No. 14, chapter 2 [19] addresses the selection of the IDF for dam design.

6.5.2 Reservoir Operation

Depending on water use, the reservoir level can have a large seasonal fluctuation. This seasonal fluctuation should be taken into account along with the seasonal wind variation if data are available. In addition, this will be a factor to consider when performing flood routings.

Remoteness of the dam site and downstream considerations (e.g., safe downstream channel capacity) will likely have an effect on reservoir operations and may need to be taken into account for freeboard considerations. More thorough discussion of these and other factors pertaining to reservoir operations and flood routings can be found in Design Standards No. 14, chapter 2 [19].

6.5.3 Malfunction of the Spillway and Outlet Works

Operation and maintenance factors should be given careful consideration in the determination of freeboard requirements. Malfunction of the spillway and/or outlet works, either due to operation error, mechanical and electrical failure, or as a result of plugging with debris, could cause the reservoir to rise above levels considered in the design. The type and intended operations of the spillway and outlet works can be significant factors in determining an adequate amount of freeboard. These and other factors related to the appurtenant works are discussed in detail in Design Standards No. 14, chapter 2 [19].

6.5.3.1 Ungated Spillways

Ungated spillways are less affected by improper maintenance and operation problems. An exception to this would be an ogee crest spillway with piers (e.g., supporting a bridge, etc.) and openings less than 40 to 50 feet that could become plugged with debris. Freeboard allowance for malfunction is usually not required for most dams with ungated spillways except for those reservoirs that depend on the outlet works to discharge a large portion of the floodflows. When shaft spillways are used, particular attention should be given to potential loss of discharge capacity as a result of debris plugging the inlet. The effect of debris would depend upon the location of flow control in the shaft spillway system. Some freeboard allowance to account for potential loss of discharge capacity as a result of debris may be warranted.

6.5.3.2 Gated Flood Outlet

Where a large, gated flood outlet is used in place of a spillway or results in a smaller overflow spillway, the gated spillway freeboard allowance given in the next paragraph should be used.

6.5.3.3 Gated Spillways

Even with regular maintenance of equipment and adequate attendance by an operator, the possibility of malfunctions of gated spillways and outlet works due to mechanical and electrical power failure or operational error should be recognized.

The designer should conduct an assessment of site-specific conditions, making quantitative evaluations where possible. For example, determine the change in MRWS resulting from failure of one of three gates to open. For some reservoirs with large surface areas, the change in MRWS might be small, while for reservoirs with small surface areas, the result of losing outflow capacity from one of three gates might result in overtopping the dam. The characteristics of the flood hydrograph would also be a factor that influences the severity of the outcome of a malfunction. Such evaluations are dam, reservoir, and site specific and necessitate analyses to determine the sensitivity of the required amount of freeboard to these and other critical factors. Additional discussion on this and possible site-specific factors can be found in Design Standards No. 14, chapter 2 [19].

6.5.4 Cracking Through the Top of the Embankment

Transverse (upstream to downstream) cracks that are deep enough to be in contact with the reservoir can occur through the top of an embankment dam. These cracks could result from differential settlement, desiccation, arching, internal stress concentrations or areas of low internal stresses, seismic activity or other dynamic loadings, adverse foundation geometry or anomalies, conduits or penetrations, drilling, grouting, instrumentation or the presence of other structures, etc. Generally, the stronger the embankment materials, the deeper a crack may extend into an embankment. The depth of a crack is a function of the unconfined compressive strength of the embankment materials [30]. Evidence of cracks may or may not exist on the crest or surface of an existing embankment dam due to maintenance activities or other disturbances. Although cracks usually close with depth, the wider a crack appears at the surface, the deeper it may extend. Embankment materials that were placed dry of their optimum moisture content are more subject to cracking than materials placed with more moisture.

Minimum freeboard is not usually increased to account for cracks. Chimney filter zones downstream of the core are the primary defenses against cracking of an embankment dam. In addition, the return period of flood loadings that would raise the reservoir high enough to intercept a crack which penetrates entirely through the embankment section is long and provides another degree of conservatism.

Normal freeboard, on the other hand, may need to be increased above the level determined by methods described earlier in this standard if cracks can be expected to extend deep enough to intercept the reservoir during normal operations and a downstream filter is absent, or the performance of such a filter is suspect.

Cracks usually narrow with depth and could be discontinuous. Deep cracks may only be able to be caused by earthquake movements. Still, usually filters, not freeboard, are included in the design of modern embankment dams to handle both static and seismic cracks. Within Design Standards No. 13, *Embankment Dams:* Chapter 13 [27] describes this potential seismic failure mode and designs to accommodate it, Chapter 9 [8] covers static deformations, and Chapter 5 [28] describes the design of embankment filters to mitigate all types of cracking.

6.5.5 Earthquake- and Landslide-Generated Waves

Waves can result from earthquakes, either from a fault displacement near or within the reservoir, or from shaking of the reservoir basin (valley floor and valley walls). The amount and type of fault displacement and energy and frequency spectrum are factors which, individually or jointly, influence the severity of waves in reservoirs. The magnitude of waves resulting from landslides is affected by the volume, speed, and geometry of the slide mass. Both types of the earthquake-generated and the landslide-generated waves can be termed "seiche waves." Some seiche waves can overtop a dam multiple times before the reservoir comes to equilibrium a long time after the initiating event. Wave runup is calculated for all seiche waves the same as it is for wind-generated waves (i.e., by using equation 8 or 9 in appendix B).

In terms of mechanisms for the earthquake-generated waves, in an event with fault displacement, reservoir water rushes to fill a void caused by vertical displacement or tilting under or adjacent to the dam. In the event of ground shaking of the reservoir basin, reservoir water follows the ground ripples, which can be of significant proportions. An earthquake quite distant from the reservoir may have more chance of creating a seiche in a reservoir than an earthquake near the reservoir. A wave generated by fault displacement spreads radially from the point of maximum vertical displacement. If the water depth does not exceed the displacement, the wave breaks and dissipates rapidly. However, a displacement wave (bore wave), in which the water piles up behind a vertical front, is not affected by reservoir shape. If the dam is on the downdrop side of the fault, the “lowered” crest height increases the chance of a wave overtopping the dam. Hebgen Lake Dam, Montana, partially failed in 1959 by overtopping of a wave generated when a fault displaced along the right side of the reservoir and tilted the reservoir toward the dam [32]. Methods and theories of seiche analysis can be found in references [2], [3], [4], [20], and [21]. For waves generated by fault displacement, applied hydraulics are used to evaluate wave height and propagation. Attempts have been made to predict runup from seiche waves caused by seismic activity [20]. Jackson Lake Dam and Lake Tahoe Dam have been analyzed for wave loadings generated by fault displacement and modified with overtopping protection to prevent failure in the event of overtopping [29]. Seiche wave modeling combined with probabilistic seismotectonics analysis has been applied at Joes Valley Dam and A.V. Watkins Dam to evaluate the risk of overtopping events for dam safety decisions [21].

Rapid reservoir drawdown, earthquakes, rain, and other factors may trigger landslides in a reservoir. Landslides are site specific. The waves generated by landslides in a reservoir must be analyzed individually as to their potential maximum height and their attenuation characteristics in the reservoir before reaching the dam. The height of landslide-generated waves is dependent on several factors. The mass and velocity of the slide and its orientation to the reservoir probably are the most significant factors for evaluating landslide-generated waves. The height of a reservoir wave from a landslide can vary from a minimum disturbance to a “Vajont size” [5]. Some methods exist for estimating the approximate size of landslide-generated water waves. A starting point for this analysis can be found in a chapter entitled “Occurrences, Properties, and Predictive Models of Generated Water Waves” [6]. Another useful paper, 14th International Commission on Large Dams Conference in Rio de Janeiro (1982), is “Prediction of Landslide-Generated Water Waves” by C.A. Pugh and D.W. Harris [7].

In some cases, a freeboard component for “large” waves may be beyond the economics or realities of any project. When a real danger of wave overtopping exists for a proposed or an existing dam, then an evaluation is required that may indicate a need for additional normal freeboard or other mitigating measures. Normal freeboard would be increased to accommodate the largest of the series of multiple waves. Aside from adding freeboard, landslide waves can be mitigated by stabilizing the landslide. Limited Reclamation experience indicates that it may be more economical to provide overtopping protection for embankments subject to fault displacement waves rather than increase freeboard [29].

6.5.6 Unanticipated Settlement of an Embankment

An embankment is usually constructed to an elevation above the design crest elevation to allow for long-term consolidation of the embankment and its foundation under static conditions. This vertical distance of embankment constructed above the design crest is called “camber.” Camber usually ranges from zero at the abutments to 1 or more feet in the middle reach of the dam. Requirements and calculation methods for camber are given in chapter 9 of Design Standards No. 13, *Embankment Dams* [8]. Camber is not part of the freeboard. However, additional freeboard may be required if the amount of settlement is not easily predictable and could be greater than determined analytically.

There are a number of methods used to estimate permanent vertical deformations of an embankment dam resulting from seismic loadings. Seismic deformation analysis of an embankment dam is discussed in chapter 13 of Design Standards No. 13 [27]. Normal freeboard should be sufficient to accommodate seismic deformations. For existing dams, seismic deformation estimates are compared to the normal freeboard in a risk-based context for dam safety assessments [22].

6.5.7 Security Concerns

The amount of freeboard and the crest width can greatly affect the response of an embankment to a security-related incident, such as an explosive device placed on the crest or an attempt to trench across the crest with mechanical equipment. An increase in freeboard or crest width will generally result in a reduction in security-related vulnerabilities and risk. Staff in the SSLE office and Technical Service Center can provide data on the predicted depth and width of different blast loads, based on the cross-section and materials of a specific embankment. This allows the designer to determine if the freeboard and crest width are sufficient to protect the structure for different design-based threats. The Reclamation SSLE office should be contacted for additional information and consultation.

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Appendix A

List of Terms

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List of Terms

ξ_p	Surf-similarity parameter. This parameter characterizes the form of the waves (i.e., spilling, plunging, collapsing, or surging).
F	The reservoir's fetch, F, is an average horizontal distance over which wind acts to generate waves at a particular point (miles).
g	The acceleration due to gravity (32.2 ft/s^2, 79,036.4 mi/h^2).
H_s	The significant wave height, the average of the highest one-third of the waves of a wave field or spectrum. "Wave height" is the vertical distance between a wave crest and the preceding trough (feet)
IDF	The inflow design flood [19]. Often, the probable maximum flood.
L	The deep water wave length is the horizontal distance between similar points on two successive waves (feet)
MRWS	Maximum reservoir water surface elevation (feet). The highest reservoir elevation resulting from routing the inflow design flood.
PFARA	Probabilistic Freeboard and Riprap Analysis computer program by the Bureau of Reclamation [17] to get a correlation of site-specific wind velocity (VMPH) versus hourly probability (P_{W_H}).
PMF	The probable maximum flood event.
P_{W_H}	The probability of the wind event (velocity) being exceeded any hour.
RUNUP	The movement of water up a structure or beach on the breaking of a wave. The runup is the vertical height that the water reaches above still water level (feet)
SEICHE	A wave, usually very large, caused by seismic deformations or a significant landslide entering into a body of water. The primary type of seismic activity that generates seiche waves and analyzed for freeboard is that associated with fault displacement within or adjacent to a reservoir.

SETUP	The wind setup is the vertical rise in the still water level on the leeward side of a body of water due to wind stresses on the surface of the water (feet). (Also called "wind tide.")
T	The period of the deep water wave. The time for two successive wave crests to pass a fixed point(s).
TAC	Top of active conservation elevation (feet).
t_{min}	The minimum time required to build up (fully develop) the maximum waves for a given wind velocity and reservoir fetch (hours).
V	The wind velocity over land (miles per hour).
VMPH	The wind velocity over water (miles per hour).
α	The angle of the upstream face of the embankment dam with the horizon (degrees).
β	The angle of incidence of the oncoming waves direction with a line perpendicular to the dam's centerline (degrees).

Appendix B

Computations for Embankment Dam Freeboard Analyses for Wind Loadings

Appendix B

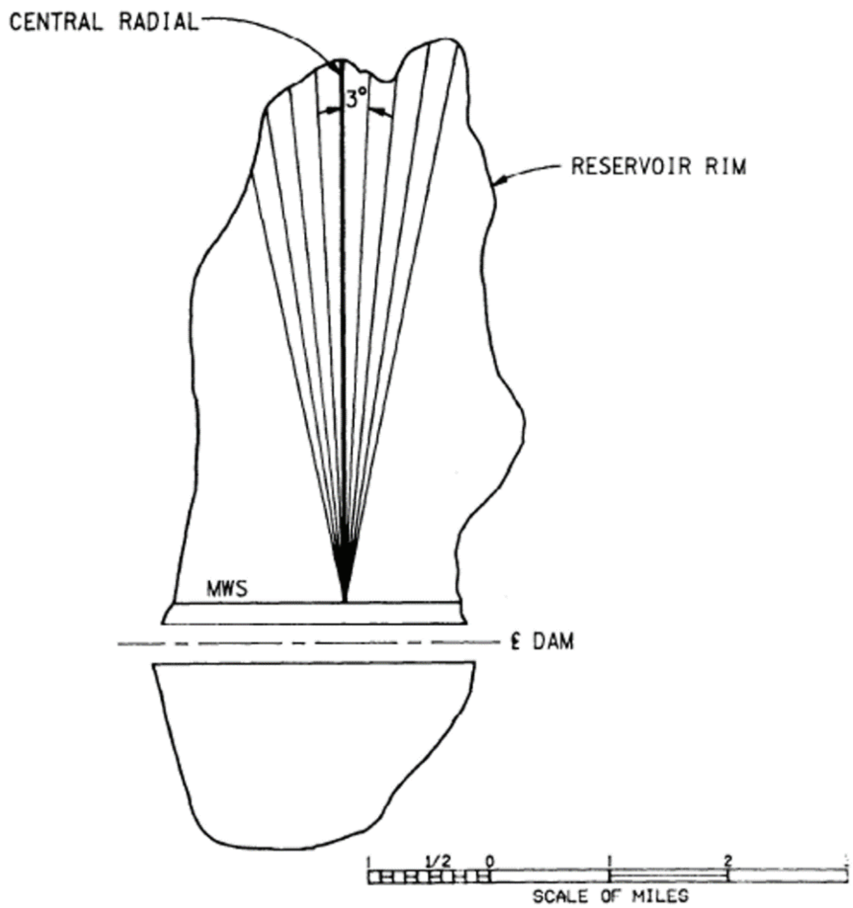
Computations for Embankment Dam Freeboard Analyses for Wind Loadings

B.1 Fetch Calculations

The recommended procedure for estimating the fetch over an inland reservoir having an irregularly shaped shoreline consists of constructing nine radials from the point of interest at 3-degree intervals and extending these radials until they first intersect the shoreline again on the opposite side of the reservoir [9] (see figure B-1). The length of each radial is measured and arithmetically averaged. While 3-degree spacing of the radials is recommended, any other small angular spacing could be used. This calculation should be performed for several directions (of the central radial) approaching the dam, including the direction where the central radial is normal to the dam axis and also the direction where the total spread results in the longest possible set of radials.

For each fetch calculated, the angle of the central radial with respect to a line normal to the dam's axis should be determined. This angle will be used with an appropriate reduction factor to adjust the runup, considering that the wave may approach the dam from a less severe direction.

In earlier wave prediction methodologies, the effect of fetch width (effective fetch) was considered to be important in limiting wave growth. The effective fetch distance was an attempt to account for the effect of the proximity of the shoreline along the fetch length. However, some of the physical arguments on which this is based are no longer considered valid. Hence, no definition of effective fetch has been formulated in this design standard. It is possible that fetch width is important if the reservoir is actually very narrow along the fetch. The effect of fetch width (i.e., the parameter W/L) has not been found to scale with relative length. Tests using the wave growth curves against observations indicate that if the "effective fetch" is used, the waves will be underestimated. Thus, **effective fetch must not be used with the curves of this standard.**



$$\text{FETCH} = \left(\sum_{N=1}^{9} \text{LENGTH OF RADIALS}\right) / \left(\text{NUMBER OF RADIALS}\right)$$

$$= \frac{(3.50 + 4.00 + 4.15 + 4.25 + 4.35 + 4.25 + 4.25 + 4.50 + 4.55)}{9}$$

$$= 37.8/9 = 4.20 \text{ MILES}$$

Figure B-1. Fetch calculation (units in miles to match text).

B.2 Methods of Freeboard Analysis

B.2.1 Design of Small Dams

The 1987 edition of *Design of Small Dams* [10] contains freeboard guidelines for small embankment dams with low hazard classifications. Table B-1 designates the least amount recommended for normal and minimum freeboard. Accompanying the table is a statement that the design of the dam should satisfy the more critical requirement of the two. The values in the table are based on empirical relationships using a wind speed of 100 miles per hour (mi/h) for normal freeboard and 50 mi/h for minimum freeboard. *Design of Small Dams* also states that "an increase in the freeboard shown (in the table) for dams where the fetch is 2.5 miles and less may be required if the dam is located in very cold or very hot climates, particularly if CL and CH soils are used for construction of the cores." Part of the reason for this is the possibility of cracks in the top of the embankment caused by freeze-thaw action or desiccation. It is also recommended that the amount of freeboard shown in the tabulation be increased by 50 percent if a smooth pavement was to be provided on the upstream slope.

Table B-1. Approximation of Minimum and Normal Freeboard [10]

Longest Fetch (miles)	Normal Freeboard (added to the normal water surface) (feet)	Minimum Freeboard (added to the maximum water surface) (feet)
<1	4	3
1	5	4
2.5	6	5
5	8	6
10	10	7

Note: These values were based on a wind velocity of 100 mi/h blowing over the normal reservoir water surface and 50 mi/h blowing over the MRWS. The effect of wind setup is not considered in the values shown. For embankment dams with soil-cement or other smooth upstream faces, depending on the smoothness of the surface, the values shown should be increased by a factor of up to 1.5.

B.2.2 Probabilistic Method

Assistant Commissioner – Engineering and Research (ACER) Technical Memorandum No. 2 [15] encouraged the use of a sophisticated probabilistic analysis for freeboard computations that is no longer used. The method used to compute a probability distribution function for elevations for a dam crest as the result of combining the probabilities of floods to produce reservoir levels below the crest and the probabilities of wind to generate waves that caused runup and setup to reach the crest elevation. A computer program PFARA (which stands for "Probabilistic Freeboard and Riprap Analysis") [17] was created to perform these computations. This complicated procedure was not used in the 20 years of its existence because the simpler method presented in this design standard makes sense, employs data in a more supportable manner, and gives good results. PFARA is still used in freeboard and riprap analysis but primarily only for the analysis of wind data and the derivation of design wind events. This part is covered in this appendix in section B.3. The program uses site-specific data to produce a probability distribution of wind velocity over water. Other data and methods can be used to derive such a relationship.

B.2.3 U.S. Army Corps of Engineers

The U.S. Army Corps of Engineers (USACE) has carried out a large amount of research on wave height determination and wave runup on embankments. The results of that research are contained in references [11] and [12]. Additional guidelines were published in Engineering Technical Letter ETL 1110-2-221, dated November 29, 1976 [1], its revision, ETL 1110-2-305, dated February 16, 1984, and the current *Hydraulic Engineering Requirements for Reservoirs*, EM 1110-2-1420, dated September, 24, 2018 [9]. The *Shore Protection Manual*, fourth edition [13], published in 1984, was the basis of previous Bureau of Reclamation references on freeboard; namely, ACER Technical Memorandum No. 2 (1992) [15]. However, the *Shore Protection Manual* has been updated and now is called the *Coastal Engineering Manual* (CEM), numbered EM-1110-2-1100 [16a and 16b]. Part II and Part VI of the CEM are applicable to freeboard computations, particularly chapter 2 of Part II (dated August 1, 2008) for wave characteristics and chapter 5 of Part VI (dated September 28, 2011) for runup and setup calculations, as well as the USACE freeboard analysis and computations for estimating the probability of overtopping. The CEM updates serve as the basis of this version of the Freeboard Design Standard.

B.3 Analysis of Existing Wind Data

B.3.1 Wind Data Stations

Maps showing the status of wind data for National Climatic Center stations in the United States are available in the *Wind Energy Resource Atlases,* published by Battelle Pacific Northwest Laboratory [14]. Those stations for which the wind data have been summarized and digitized are the primary sources of wind persistence data needed for determining design winds for computing freeboard requirements.

Wind data from stations with the highest degree of applicability to the reservoir site should be used for computing wind-generated wave height, wave runup, and wind setup. Applicability includes consideration of proximity, similarity of topography, vegetation and relief, meteorological similarity, and length and content of wind records.

B.3.2 Converting the Wind Data to Probabilities

The tables of wind persistence from Battelle list the number of occurrences that a given wind velocity has been exceeded for a selected number of consecutive hours. By converting the “number of occurrences” to “number of hours,” and dividing by the total number of hours of the period of record, the value P_{W_H}, the probability of the wind exceeding a given velocity for a specific number of hours, is derived. Additional information is provided in the riprap design standard DS-13(7)-2.1 in section 7.3.3 [33].

B.3.3 Transposition of the Probabilities to the Reservoir Site

Probabilities of the wind exceeding a given velocity must be transposed to the reservoir. When data from more than one station are being transposed, weighting factors should be used to account for relative distances to the reservoir; differences between the station and reservoir such as surrounding vegetation, topography, and meteorologic setting; and differences in period of record between stations. Weighting factors are assigned to each station with higher weights going to stations that are closer to the reservoir, those with topography and surrounding vegetation similar to that of the reservoir, those with longer periods of records, and so forth.

To transpose the probability of wind exceeding a given velocity from a given station to the reservoir site, each value of hourly wind probability, $\mathrm{P}_{\mathrm{W}_\mathrm{H}}$, for the station is multiplied by the weighting factor assigned to that station and divided by the sum of weighting factors assigned to all stations. The probability of wind exceeding a given velocity for a given duration at the reservoir is the sum of the transposed probabilities for the respective velocities from each station.

B.3.4 Wind Event Curves

The probability of wind exceeding a given velocity (P_{W_H}) for 1, 2, 3, 4, and 5 consecutive hours at the reservoir is plotted for each wind velocity. The data points are plotted on semilogarithmic paper, and a best fit curve is drawn for each velocity. Each curve represents the probability of the wind exceeding a specific velocity for a selected duration (P_{W_H}) during any wind event.

B.3.5 Overwater Correction

Winds blowing over land change in velocity as they pass over a reservoir. An adjustment must be made, therefore, to convert wind velocities measured over land to overwater velocities. The wind velocities for each wind event curve are measured over land and must be adjusted to overwater velocities for use in calculating wave heights, wave runup, and wind setup. Figure B-2 demonstrates the relationship between winds blowing over land as compared to wind blowing over water 32.8 feet above the ground.

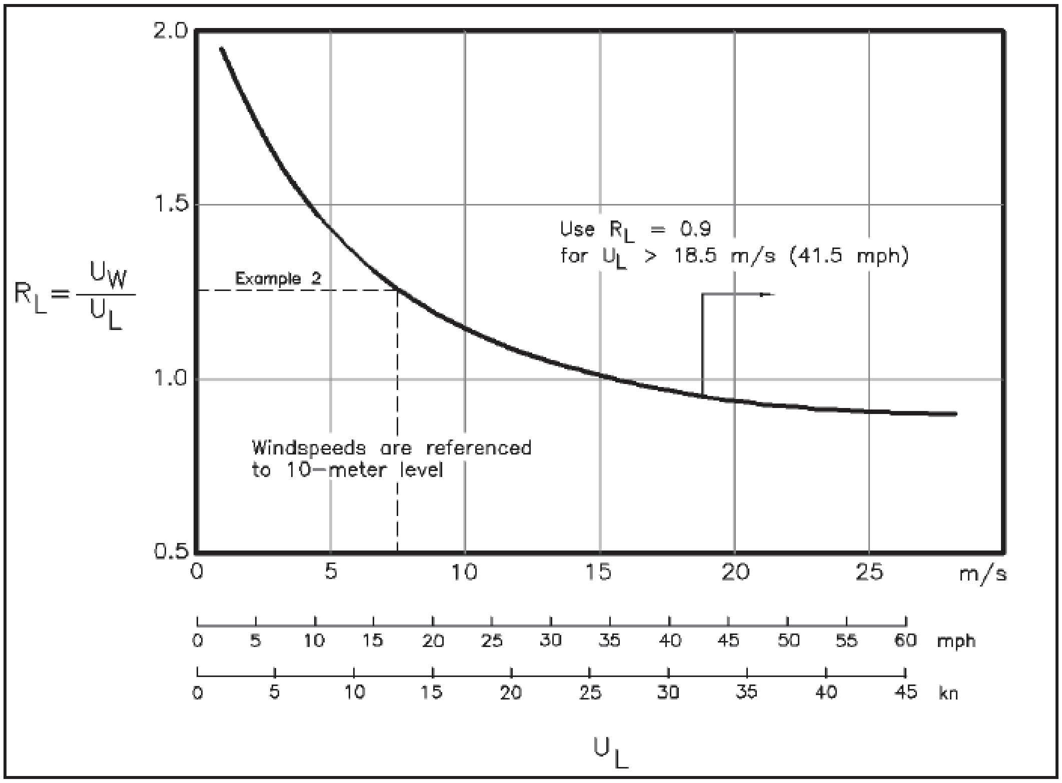


Figure B-2. Ratio of wind speed over water to wind speed over land as a function of wind speed over land [16a].

B.3.6 Minimum Wind Duration to Reach Maximum Wave Heights

Maximum wave heights in a reservoir can be limited by three factors: fetch length (overwater distance the wind blows), wind duration (how long the wind conditions persist), and the wind speed (called fully developed conditions or the maximum wave height for a given wind speed). Typically, the design conditions for Reclamation facilities are limited by fetch length. Additional information on duration-limited and fully developed waves can be found in the Coastal Engineer Manual [16a, 16b].

The duration needed for a given wind speed to generate the highest waves for a given fetch is designated as the minimum duration. The minimum durations can be computed as:

$$t_{min} = 1.87\ (F^{0.67} / VMPH^{0.34}) \qquad \text{[Equation 1]}$$

Where:

t_{min} = The minimum duration required to generate the maximum wave height (hours)

F = Fetch (miles)

VMPH = Wind velocity over water (mi/h)

The minimum durations are then plotted on the respective wind event curves. The probability of each velocity being exceeded for the minimum duration needed to produce a maximum wave height is the ordinate corresponding to the minimum duration plotted on the P_{w_H} wind event curve.

B.3.7 Wind Event Probabilities

A curve joining the minimum wind durations plotted on the wind event curves represents the probability of a selected overwater wind velocity being exceeded for the minimum duration needed to produce its maximum wave. For ease in determining the wind velocity likely to occur for a minimum duration during a given reservoir water surface event, a curve of probability of wind velocity being exceeded (P_{w_H}) versus wind velocity (over water) should be drawn on semilogarithmic paper. Values of P_{w_H} and their respective overland (converted to over water) velocities corresponding to the minimum duration for each velocity should be used.

B.4 Wind Effects on Water

B.4.1 Wave Height

Wind-generated waves are not uniform in height, but they consist of a distribution of waves with various heights [11, 12]. The significant wave height (H_s) is defined as the average of the highest one-third (33.33 percent) of the waves in a wave field. The fetch-limited significant wave height (in feet) is given by:

$$H_s = 0.0245\ F^{1/2}\ VMPH\ (1.1+0.0156\ VMPH)^{1/2} \qquad \text{[Equation 2]}$$

Using a Rayleigh distribution, other statistical wave height measures can be estimated from the significant wave height using the values provided in table B-2. For example, it can be seen from table B-2 that the 2-percent exceedance wave height is approximately 1.4 H_s, and the 0.4-percent exceedance wave height is approximately 1.67 H_s.

Table B-2. Common Wave Height Relationships

Percent of Total No. of Waves in Series Averaged to Compute Specific Wave Height (H)	Ratio of Specific Wave Height, H, to Average Wave Height, H_{ave} (H/H_{ave})	Ratio of Specific Wave Height, H, to Significant Wave Height H_s (H/H_s)	Percent of Waves Exceeding Specific Wave Height (H)
1	2.66	1.67	0.4
5	2.24	1.40	2
10	2.03	1.27	4
20	1.80	1.12	8
25	1.71	1.07	10
30	1.64	1.02	12
33.33 (i.e., H_s)	1.60	1.00	13
40	1.52	0.95	16
50	1.42	0.89	20
75	1.20	0.75	32
100	1.00	0.62	46

Depending on which formulation of A and C parameters is used in section B.4.3, the wave height statistic used to compute wave runup is different. When computing wave runup from table B-4 and equation 8, the input parameter is the significant wave height, H_S, and the output is the 2-percent exceedance wave runup, $R_{2\%}$. When computing wave runup from table B-5 and equation 9, the approximate exceedance value of the statistical wave height, $H_{i\%}$, corresponds to the approximate exceedance of wave runup, $R_{i\%}$ (e.g., $H_{0.4\%}$ would yield $R_{0.4\%}$). The exceedance value used for wave runup is typically $R_{2\%}$ [9]; however, a different exceedance value can be selected based on the ability of the crest and downstream slope to withstand overtopping by wave action. When the crest and downstream slope are adequately protected against erosion, will not slough or soften excessively, and when public traffic will not be interrupted, a 4-percent exceedance wave height (1.27 x height of significant wave) may be justified. A wave height equal to 1.67 x height of the significant wave should be used if overtopping by only an infrequent wave is permissible.

Wave heights for fetches that are not normal to the dam axis should be reduced according to a factor derived from figure B-3. Just as wave heights in a wave field are not uniform, there is also a distribution (spread) in wave directions. The significant wave height is multiplied by the reduction factor to obtain a reduced significant wave height for design.

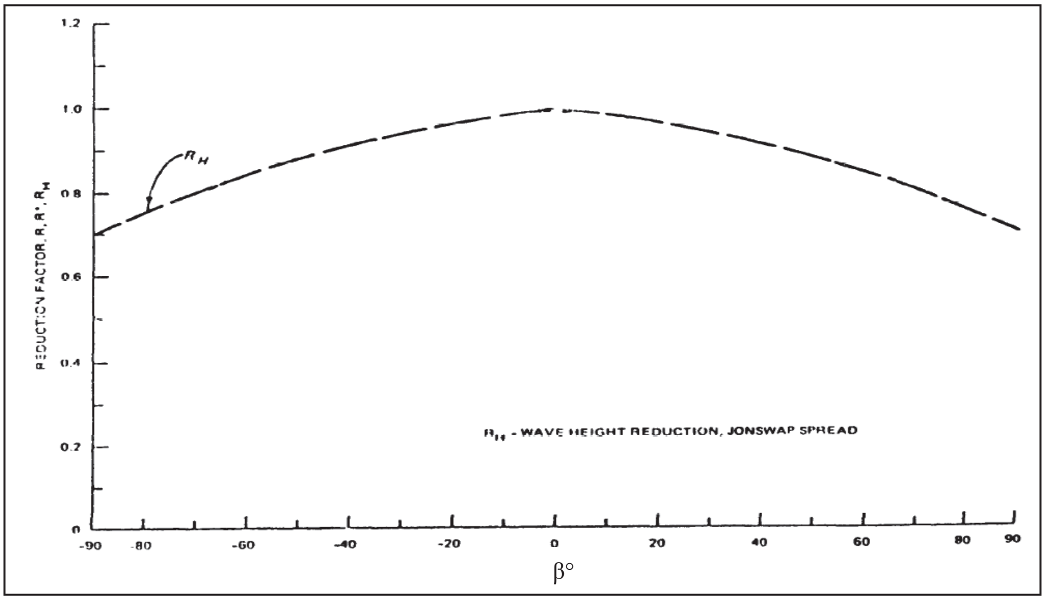


Figure B-3. Wave height reduction due to angular spread [16a].

Where:

β° = The angle between the fetch and the dam axis (degrees). (0° is normal incidence and is commonly used to compute fetch, which is directly perpendicular to the dam axis).

B.4.2 Wave Length and Wave Period

The deep water wave length (in feet) can be computed by the relationship:

$$L = \frac{gT^2}{2p} = 5.12\,T^2 \qquad \text{[Equation 3]}$$

in which T = the wave period in seconds from the equation below. The fetch -limited peak wave period (in sec) is given by:

$$T = 0.464\,F^{1/3}\,VMPH^{1/3}(1.1 + 0.0156VMPH)^{1/6} \qquad \text{[Equation 4]}$$

Wave periods are normally distributed about the peak period for locally generated waves. It may be assumed that the wave period, T, is the same for all waves in the wave field.

Most dams have relatively deep reservoirs compared to the wind-generated wave length, and the wave is unaffected by the reservoir floor. The above equations for wave height, wave period, and minimum duration (equations 4, 6, and 1) are valid when the water is deeper than one-half of the wave length. If reservoir depth becomes a limiting factor, different relationships for shallow water waves can be used; or, in some instances, deep water conditions can be conservatively assumed [9]. Wave height, wave period, and minimum duration for shallow water waves can be obtained from USACE EM 1110-2-1100 (Part VI) [16b] and USACE EM 1110-2-1420 [9].

B.4.3 Wave Runup

If a deep water wave reaches a sloping embankment without major modification in characteristics, the wave will ultimately break on the embankment and run up the slope to a height governed by the angle of the slope, the roughness and permeability of the embankment surface, and the wave characteristics. Wave runup, R, is the vertical difference between the maximum level attained by the rush of water up the slope and the still water elevation.

To compute the runup, a surf similarity factor for peak wave heights, ξ_p is first computed from the following equation:

$$\xi_p = \frac{\tan\alpha}{\sqrt{s_p}} \qquad \text{[Equation 5]}$$

Where:

α = The slope angle of the upstream face of the embankment dam with the horizontal. Commonly, the upstream slope of an embankment dam is 3(H):1(V) such that the tan α = 0.33. Note: these equations used to compute runup should be used only for dam slopes of 5(H):1(V) or steeper.

s_p = The steepness of the peak waves, computed as follows:

$$s_p = \frac{H_{i\%}}{L} = \frac{2p}{g}\,\frac{H_{i\%}}{T^2} \qquad \text{[Equation 6]}$$

$H_{i\%}$= statistical wave height (feet) of the incident waves from equation 2 and table B-2.

L = Wavelength (feet) from equation 3

T = Wave period (seconds) from equation 4

More simply:

$$\xi_p = \frac{2.26\,T\,(tan\,\alpha)}{\sqrt{H_{i\%}}} \qquad \text{[Equation 7]}$$

For typical freeboard evaluations, table B-4 can be used with equation 8 to compute the 2-percent exceedance wave runup, $R_{2\%}$. When this approach is used, H_s is the wave height input parameter and runup is computed from the following equation:

$$R_{2\%} = H_s\left(A\xi_p + C\right)\gamma_r\gamma_b\gamma_h\gamma \qquad \text{[Equation 8]}$$

Where:

$R_{2\%}$ = 2-percent exceedance runup on a relatively impermeable slope (i.e., the upstream slope of an embankment dam) (feet)

H_s = Significant wave height (feet)

ξ_p = Surf similarity factor (from the previous equations)

A, C = Coefficients dependent on x_p (see table B-4 below) and the exceedance probability of the runup (2 percent is used for typical freeboard and riprap calculations)

$\gamma_r\gamma_b\gamma_h\gamma$ = Reduction factors provided in table B-3 and figure B-4

On certain facilities, a deviation from using the 2-percent exceedance value may be desirable. To use a less conservative exceedance value, a design standard deviation is required as outlined in Section YY. This might include instances where only an occasional overtopping wave is permissible, or instances where the crest is armored and more frequent overtopping is permissible. When wave runup heights other than the 2-percent exceedance value are needed, equation 9 can be used with table B-5 to compute various wave runup exceedance values. When using table B-5 to compute wave runup, $R_{i\%}$ is obtained from the following equation:

$$R_{i\%} = H_{i\%}\left(A\xi_p + C\right)\gamma_r\gamma_b\gamma_h\gamma \qquad \text{[Equation 9]}$$

Where:

$R_{i\%}$ = i^{th} percent exceedance runup on a relatively impermeable slope (i.e., the upstream slope of an embankment dam) (feet)

$H_{i\%}$ = Selected statistical wave height (feet)

ξ_p = Surf similarity factor (from the previous equations)

A, C = Coefficients dependent on ξ_p (see table B-5 below) and the exceedance probability of the runup (i^{th} percent)

$\gamma_r\gamma_b\gamma_h\gamma$ = Reduction factors derived as follows provided in table B-3 and figure B-4.

Under this alternate form of the wave runup equation, the exceedance value of wave runup equals the exceedance value of wave height that is used as the input parameter (e.g., $H_{0.4\%}$ yields $R_{0.4\%}$, $H_{4\%}$ yields $R_{4\%}$, etc.).

To account for the roughness of the slope, γ_r is a reduction factor taken from table B-3 for use in the runup equation, above. For riprap, a value of 0.55 is suggested. γ_b is a reduction factor for the influence of a berm ($\gamma_b = 1.0$ for nonbermed profiles). γ_h is a reduction factor for the influence of shallow-water conditions, where the wave height distribution deviates from the Rayleigh distribution (γ_h = 1.0 for Rayleigh distributed waves).

Table B-3. Surface Roughness Reduction Factor (valid for 1 <x_p <3-4) [16b]

Type of Slope Surface	γ_r
Smooth, concrete, asphalt	1.0
Smooth block revetment	1.0
Grass (3 centimeters in length)	0.90 – 1.0
One layer of rock, diameter *D*, (H_s/D = 1.5 – 3.0)	0.55 – 0.6
Two or more layers of rock, (H_s/D = 1.5 – 6.0)	0.50 – 0.55

To account for a reduction in runup, due to the direction of the fetch relative to the dam axis, use figure B-4 to derive γ_β.

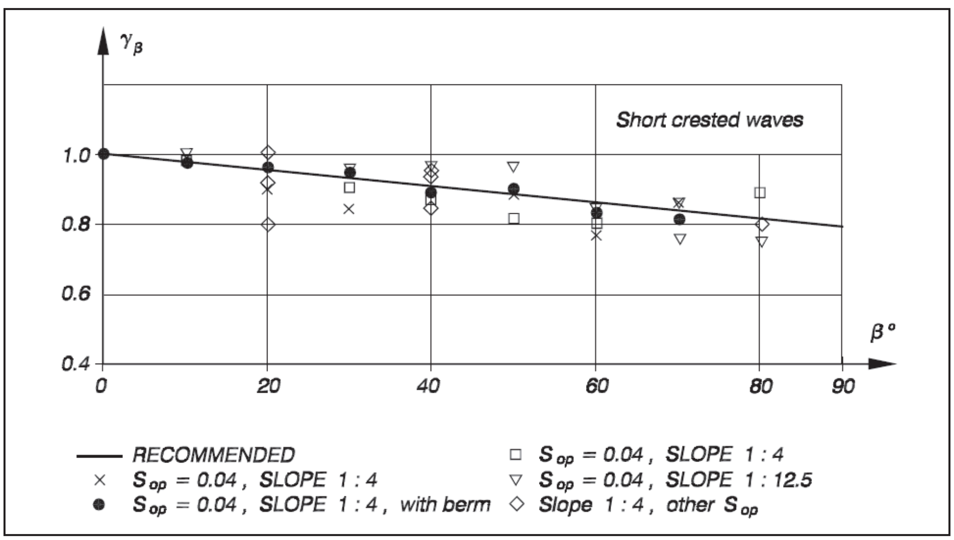


Figure B-4. Influence of angle of incidence, β, and the directional spreading on runup on smooth slopes at Delft Hydraulics [16b].

Given the surf similarity factor for peak waves, ξ_p, table B-4 is used to derive the variables A and C in the above equation for 2-percent runup (average of the highest 2 percent of the runups, which is commonly used in the CEM, as well as for freeboard and riprap analysis).

Table B-4. Values for Variables *A* and *C* for Runup Equation 8 [16b]

ξ-Limits	*A*	*C*
$\xi_p \leq 2.5$	1.6	0
$2.5 < \xi_p < 9$	-0.2	4.5

Table B-5. Values for Variables *A* and *C* for Runup Equation 9 [16b]

ξ-Limits	*A*	*C*
$\xi_p \leq 2.5$	1.35	0
$2.5 < \xi_p < 9$	-0.15	3.0

For runup calculations on most embankment dam freeboard analyses, γ_b and γ_h are set equal to 1.0. If shallow water wave distributions are greatly different than a Rayleigh distribution, or if there is a berm on the upstream slope, USACE EM 1110-2-1100 (Part VI) [16b] and EM 1110-2-1420 [9] should be referenced for these other reduction factors.

For typical embankment slopes, equations 8 and 9 will yield similar results for 2-percent exceedance runup. For example, if H_s is used as an input for equation 8 and table B-4 is used, the computed 2-percent exceedance wave runup will be nearly identical to the wave runup computed by using $H_{2\%}$ as an input with equation 9 and table B-5.

B.4.4 Wind Setup

Wind blowing over a water surface exerts a horizontal shear force on the water, driving it in the direction of the wind. In an enclosed body of water, the wind effect results in a rise in the water level at the downwind end of the fetch. This effect is termed “wind tide” or “wind setup.”

Wind setup in feet, S, is computed as follows:

$$S = \frac{VMPH^2 F}{1400\, D} \qquad \text{[Equation 10]}$$

Where:

VMPH = The design wind velocity over water (mi/h)
F = Wind fetch (miles)
D = Average depth of water (feet)

The value of D should be a reasonable approximation of the average depth along the fetch length, with more emphasis given to depths within a few miles of the location for which the setup is being computed. The direction of fetch is taken as that of the central radial used in computing fetch.

Appendix C

Example Minimum and Normal Freeboard Computations for Wind Loadings

Appendix C

Example Minimum and Normal Freeboard Computations for Wind Loadings

Given:

Inflow design flood (IDF) inflow curve shown in figure 6.3.1-1
Maximum reservoir water surface (MRWS) = 3025.0 feet (ft)
Normal reservoir water surface (NRWS) = 3000.0 ft
Fetch computed as shown in figure B-1
Average depth along the fetch = D = 50 ft
Upstream slope of riprap 3:1 (horizontal:vertical)
tan α = 1/3 = 0.33
Site-specific wind data from the wind probability curve shown in figure C-1.

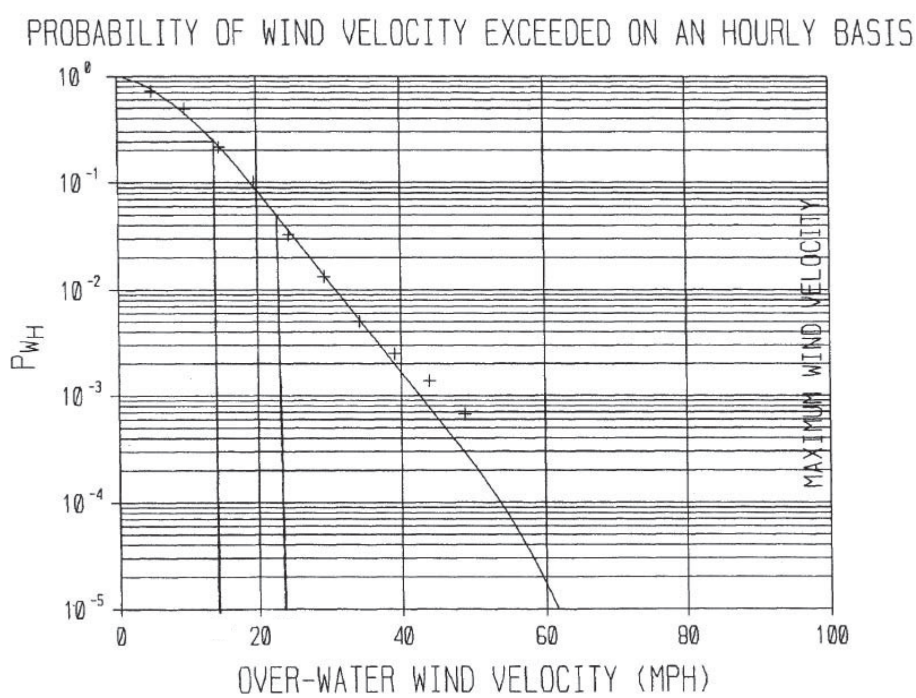


Figure C-1. Wind probability curve.

Minimum Freeboard

Select the highest required dam crest elevation between 3.0 ft above MRWS or the runup and setup from a typical wind when the reservoir is at the MRWS.

To derive the required crest elevation 3.0 ft above the MRWS:

Crest elevation = MRWS + 3.0 ft

Crest elevation = 3025.0 + 3.0 = 3028.0 ft

To derive the required crest elevation from the runup and setup from a typical wind above the MRWS:

Use a wind with a 10% probability of hourly exceedance, $P_{W_H} = 0.1$

$$VMPH = 19\ mph \qquad \text{from figure C-1, above}$$

$$H_s = 0.0245\ F^{1/2}\ VMPH\ (1.1+0.0156\ VMPH)^{1/2} \qquad \text{[Equation 2]}$$

$$H_s = 0.0245\ (4.2)^{1/2}\ (19)(1.1+0.0156\ (19))^{1/2}$$

$$H_s = 1.1\ ft$$

$$T = 0.464\ F^{1/3}\ VMPH^{1/3}(1.1 + 0.0156VMPH)^{1/6} \qquad \text{[Equation 4]}$$

$$T = 0.464\ (4.2)^{1/3}\ (19)^{1/3}(1.1 + 0.0156\ (19))^{1/6}$$

$$T = 2.11\ seconds$$

$$\xi_p = \frac{2.26\ T\ (\tan\alpha)}{\sqrt{H_s}} \qquad \text{[Equation 7]}$$

$$\xi_p = \frac{2.26\ (2.11)\ (0.33)}{\sqrt{1.1}}$$

$$\xi_p = 1.50$$

A = 1.6 from table B-4
C = 0

$$R_{2\%} = H_s\left(A\xi_p + C\right)\gamma_r\gamma_b\gamma_h\gamma \qquad \text{[Equation 8]}$$

$$\gamma_r = 0.55 \qquad \text{for riprap}$$
$$\gamma_b = 1.0$$

$$\gamma_h = 1.0$$

$$\gamma = 1.0$$ for the angle of incidence, β = 0 and figure B-4

$$R_{2\%} = 1.1\big((1.6)(1.50) + 0\big)(0.55)(1.0)(1.0)(1.0)$$

$$R_{2\%} = 1.45 \text{ ft}$$

$$S = \frac{VMPH^2 F}{1400\,D} \qquad \text{[Equation 10]}$$

$$S = \frac{19^2(4.2)}{1400\,(50)}$$

$$S = 0.02 \text{ ft}$$

$$R_{2\%} + S = 1.45 + 0.02 = 1.47 \text{ ft}$$

Crest elevation required from the MRWS elevation 3025 = 3025 + 1.47 = 3026.47 ft

Select the dam crest elevation to be the higher of the two calculations above:

Crest elevation required 3.0 ft above the MRWS of 3025.0 = 3028.0
Crest elevation required above the MRWS due to typical winds = 3026.5

***Therefore**:*

Set the required minimum dam crest elevation = 3028.0 ft
Minimum freeboard = 3.0 ft

Check to see if the crest elevation of 3028.0 is adequate to prevent wave runup and wind setup over the crest elevation by winds that may occur when the IDF is within 2 and 4 feet of the MRWS

These checks are used for existing dams, but can also be used for new dams.

To derive the required crest elevation from 2.0 ft below the MRWS:

2 ft below the MRWS = 3025.0 - 2.0 = 3023.0 ft

Duration at El. 3023.0 = 33.5 hr - 22.5 hr = 11.0 hr from figure 6.3.1-2

$$P_{W_H} = \frac{1}{Duration} = \frac{1}{11.0\ hr} = 0.091$$

$$VMPH = 20\ mph$$ from figure C-1 above

$$H_s = 0.0245\ F^{1/2}\ VMPH\ (1.1+0.0156\ VMPH)^{1/2}$$ [Equation 2]

$$H_s = 0.0245\ (4.2)^{1/2}\ (20)(1.1+0.0156\ (20))^{1/2}$$

$$H_s = 1.2\ ft$$

$$T = 0.464\ F^{1/3}\ VMPH^{1/3}(1.1 + 0.0156VMPH)^{1/6}$$ [Equation 4]

$$T = 0.464\ (4.2)^{1/3}\ (20)^{1/3}(1.1 + 0.0156\ (20))^{1/6}$$

$$T = 2.15\ seconds$$

$$\xi_p = \frac{2.26\ T\ (tan\ \alpha)}{\sqrt{H_s}}$$ [Equation 7]

$$\xi_p = \frac{2.26\ (2.15)\ (0.33)}{\sqrt{1.2}}$$

$$\xi_p = 1.46$$

A = 1.6 from table B-4
C = 0

$$R_{2\%} = H_s\left(A\xi_p + C\right)\gamma_r\gamma_b\gamma_h\gamma$$ [Equation 8]

$\gamma_r = 0.55$ for riprap
$\gamma_b = 1.0$
$\gamma_h = 1.0$
$\gamma = 1.0$ for the angle of incidence, $\beta = 0$

$$R_{2\%} = 1.2\big((1.6)(1.46) + 0\big)(0.55)(1.0)(1.0)(1.0)$$

$$R_{2\%} = 1.54 \text{ ft}$$

$$S = \frac{VMPH^2 F}{1400\ D}$$ [Equation 10]

$$S = \frac{20^2(4.2)}{1400\ (50)}$$

$$S = 0.03 \text{ ft}$$

$$R_{2\%} + S = 1.54 + 0.03 = 1.57 \text{ ft}$$

Crest elevation required from El. 3023 = 3023 + 1.57 = 3024.6 ft OK

To derive the required crest elevation from 4 ft below the MRWS:

4 ft below the MRWS = 3025.0 - 4.0 = 3021.0 ft

Duration at El. 3023.0 = 37.5 hr - 20.0 hr = 17.5 hr from figure 6.3.1-2

$$P_{W_H} = \frac{1}{Duration} = \frac{1}{17.5\ hr} = 0.057$$

$$VMPH = 22\ mph$$ from figure C-1, above

$$H_s = 0.0245\ F^{1/2}\ VMPH\ (1.1+0.0156\ VMPH)^{1/2}$$ [Equation 2]

$$H_s = 0.0245\ (4.2)^{1/2}\ (22)(1.1+0.0156\ (22))^{1/2}$$

$$H_s = 1.3\ ft$$

$$T = 0.464\ F^{1/3}\ VMPH^{1/3}(1.1 + 0.0156VMPH)^{1/6}$$ [Equation 4]

$$T = 0.464\ (4.2)^{1/3}\ (22)^{1/3}(1.1 + 0.0156\ (22))^{1/6}$$

$$T = 2.22\ seconds$$

$$\xi_p = \frac{2.26\ T\ (tan\ \alpha)}{\sqrt{H_s}}$$ [Equation 7]

$$\xi_p = \frac{2.26\ (2.22)\ (0.33)}{\sqrt{1.3}}$$

$$\xi_p = 1.45$$

A = 1.6 from table B-4
C = 0

$$R_{2\%} = H_s\left(A\xi_p + C\right)\gamma_r\gamma_b\gamma_h\gamma$$ [Equation 8]

$\gamma_r = 0.55$ for riprap
$\gamma_b = 1.0$
$\gamma_h = 1.0$
$\gamma = 1.0$ for the angle of incidence, β = 0 and figure B-4

$$R_{2\%} = 1.3\big((1.6)(1.46) + 0\big)(0.55)(1.0)(1.0)(1.0)$$

$$R_{2\%} = 1.67 \text{ ft}$$

$$S = \frac{VMPH^2 F}{1400\ D} \qquad \text{[Equation 10]}$$

$$S = \frac{22^2(4.2)}{1400\ (50)}$$

$$S = 0.03 \text{ ft}$$

$$R_{2\%} + S = 1.67 + 0.03 = 1.70 \text{ ft}$$

Crest elevation required from El. 3021 = 3021 + 1.70 = 3022.7 ft   OK

Normal Freeboard with 2 Percent Exceedance Wave Runup

Check that crest elevation 3028.0 feet satisfies normal freeboard. Runup + setup from a 100 mph wind

$$H_s = 0.0245\ F^{1/2}\ VMPH\ (1.1+0.0156\ VMPH)^{1/2} \qquad \text{[Equation 2]}$$

$$H_s = 0.0245\ (4.2)^{1/2}\ (100)(1.1+0.0156\ (100))^{1/2}$$
$$H_s = 8.2\ ft$$

$$T = 0.464\ F^{1/3}\ VMPH^{1/3}(1.1 + 0.0156VMPH)^{1/6} \qquad \text{[Equation 4]}$$

$$T = 0.464\ (4.2)^{1/3}\ (100)^{1/3}(1.1 + 0.0156\ (100))^{1/6}$$

$$T = 4.09\ seconds$$

$$\xi_p = \frac{2.26\ T\ (tan\ \alpha)}{\sqrt{H_s}} \qquad \text{[Equation 7]}$$

$$\xi_p = \frac{2.26\ (4.09)\ (1/3)}{\sqrt{8.2}}$$

$$\xi_p = 1.08$$

A = 1.6   from table B-4
C = 0

$$R_{2\%} = H_s\left(A\xi_p + C\right)\gamma_r\gamma_b\gamma_h\gamma \qquad \text{[Equation 8]}$$

$\gamma_r = 0.55$ for riprap
$\gamma_b = 1.0$
$\gamma_h = 1.0$
$\gamma = 1.0$ for the angle of incidence, β = 0 and figure B-4

$$R_{2\%} = 8.2\big((1.6)(1.08) + 0\big)(0.55)(1.0)(1.0)(1.0)$$

$$R_{2\%} = 7.8 \text{ ft}$$

$$S = \frac{VMPH^2F}{1400\ D} \qquad \text{[Equation 10]}$$

$$S = \frac{100^2(4.2)}{1400\ (50)}$$

$$S = 0.6 \text{ ft}$$

$$R_{2\%} + S = 7.8 + 0.6 = 8.4 \text{ ft}$$

Crest El. required from the NRWS = 3000.0 ft
Crest elevation to satisfy normal freeboard = 3000.0 + 8.4 = 3008.4 ft

3008.4 < 3028 ✓

Therefore,

Crest elevation of 3028.0 satisfies normal freeboard criteria

Select

Dam crest elevation = 3028.0 ft

Normal Freeboard with 0.4 Percent Exceedance Wave Runup

$H_s = 8.2$ *ft,* Same as above

$T = 4.09$ *seconds* Same as above

$H_{0.4\%} = 1.67\ H_S = 1.67\ (8.2\ ft) = 13.7\ ft$ from table B-2

$$\xi_p = \frac{2.26\,T\,(tan\,\alpha)}{\sqrt{H_{i\%}}} \qquad \text{[Equation 7]}$$

$$\xi_p = \frac{2.26\ (4.09)\ (1/3)}{\sqrt{13.7}}$$

$$\xi_p = 0.83$$

A = 1.35 from table B-5
C = 0

$$R_{0.4\%} = H_{0.4\%}\left(A\xi_p + C\right)\gamma_r\gamma_b\gamma_h\gamma \qquad \text{[Equation 9]}$$

$\gamma_r = 0.55$ for riprap
$\gamma_b = 1.0$
$\gamma_h = 1.0$
$\gamma = 1.0$ for the angle of incidence, β = 0 and figure B-4

$$R_{0.4\%} = 13.7\big((1.35)(0.83) + 0\big)(0.55)(1.0)(1.0)(1.0)$$

$$R_{0.4\%} = 8.5 \text{ ft}$$

$S = 0.6$ ft Same as above

$$R_{0.4\%} + S = 8.5 + 0.6 = 9.1 \text{ ft}$$

Crest El. required from the NRWS elevation 3000: 3000 + 9.1 =3009.1 ft

3009.1 < 3028 ✓

Therefore,

Crest elevation of 3028.0 satisfies normal freeboard criteria for remote waves.

Select

Dam crest elevation = 3028.0 ft