ACER TECHNICAL MEMORANDUM NO. 2
ASSISTANT COMMISSIONER - ENGINEERING AND RESEARCH
DENVER, COLORADO

FREEBOARD CRITERIA AND GUIDELINES FOR COMPUTING FREEBOARD ALLOWANCES FOR STORAGE DAMS

U.S. DEPARTMENT OF THE INTERIOR
Bureau of Reclamation
Revised 1992 (1981)

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PREFACE

This document presents Bureau of Reclamation (Reclamation) policy pertaining to freeboard allowances for storage dams. Freeboard is provided to prevent overtopping of the dam crest by waves. Freeboard also provides an additional measure of safety of the dam to account for the uncertainties in accurately estimating reservoir water surfaces and wave heights, and uncertainties in reservoir operation and structure performance. This memorandum contains information on the many considerations that are required for freeboard analysis as well as methods to account for these considerations and perform the analysis.

This edition (1992) of ACER Technical Memorandum No. 2 is significantly different from the original 1981 edition. This second edition offers a simplified approach to freeboard calculations under certain conditions and suggests a probabilistic approach if the simplified computations are not appropriate. More extensive wind data are also now available to facilitate the use of probability. The wind effects on water have also been revised according to the latest Corps of Engineer's studies. The crest elevations of both new and existing dams may be evaluated on the basis of a calculated risk.

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FOR

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I. INTRODUCTION

A. Purpose

Specification of freeboard is critical in protecting downstream areas against possible hazards resulting from overtopping of a dam. The objective of freeboard is to provide defense against overtopping due to high reservoir inflows, wind setup and wave runup, landslides and seismic activity, unanticipated settlement of the embankment, malfunction of water release structures, uncertainties in the operation and maintenance of the dam and appurtenant structures, and hydrologic uncertainties.

This basic objective of freeboard does not necessarily require total prevention of splash over the dam 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. However, the objectives of freeboard allowance for 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, or cause other adverse effects not necessarily associated with the general safety of the structure.

B. Freeboard Policy for Bureau of Reclamation Dams

A policy for freeboard is important so that the risk of failure due to overtopping achieves minimum standards. The amount of freeboard required to reduce the risk of failure varies depending on the type, condition, and setting of the dam. Therefore, Bureau of Reclamation freeboard policy has been developed for three categories of dam types relative to their age and erodibility, as follows:

1. New concrete dams. - Dams made with conventional concrete or roller compacted concrete, and any other types of dams that can resist the erosive action of temporary overtopping flow should be designed so that the top of the nonoverflow section of a dam is coincident with the maximum water surface (MWS) elevation. The standard 3.5-foot (1.1-m)[1] high solid parapet entirely above the elevation of the nonoverflow section provides for minimum freeboard in the event of the probable maximum flood (PMF). Due to the ability of concrete dams to resist erosion, this is ordinarily the only type of freeboard necessary to consider. Exceptional cases may point to a need for more freeboard, depending on the anticipated wave height or other factors that will be discussed later.

2. New embankment dams. - Freeboard should be determined for new embankment dams from various water surface elevations in order to select a design crest elevation that adequately protects the embankment from the full range of wind and flood loading conditions. The design crest elevation should be the highest that would result from calculating all freeboard requirements.

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[1]Metric equivalents are nominal conversions, not exact.

a. Freeboard requirements. - Although the following freeboard requirements can be defined for both concrete and embankment dams, criteria for the computation of freeboard are only presented within the embankment dams freeboard policy discussion because embankment dams are more sensitive to overtopping. Unlike concrete dams, embankments are erodible and subject to failure without adequate allowance for freeboard.

(1) Freeboard criteria at MWS. - When the reservoir is at MWS, the minimum freeboard should be the greater of: (a) 3 feet (0.9 m), or (b) the sum of the setup and runup that would be generated by the average winds that would be expected to occur during large floods, (i.e., floods greater than the 1,000-year event). Such a wind velocity should be obtained from local authorities or meteorologists who can associate the winds that would occur during flood events. Due to uncertainty that cannot be avoided for such an estimate, a good freeboard analysis should consider a variety of winds that may occur. If the reservoir or watershed is very large in comparison to the size of the storm, the wind events that occur when the water surface is near maximum may be statistically independent of the storm that created the flood. In this case, a typical wind of not less than 10 percent exceedance probability[2] should be used to compute a runup for minimum freeboard.

(2) Normal water surface freeboard criteria. - When the reservoir is at the normal water surface, 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 that could reasonably occur (e.g., 60-100 mi/h; 95-160 km/h).

(3) Intermediate water surface freeboard criteria. - When the reservoir is at an intermediate elevation, that is, an elevation between the MWS and the normal water surface or top of joint-use or active conservation capacities, an intermediate freeboard requirement should be determined that has a remote probability (a 10,000-year return period may be used as an initial estimate) of being exceeded by any combination of wind-generated waves and water surfaces occurring simultaneously.

b. Scope of freeboard analysis. - The recommended approach to perform an embankment dam freeboard analysis is to start by choosing 3 feet (0.9 m) of minimum freeboard above the maximum water surface as protection against wind-generated waves, then check to see if such a crest elevation would satisfy normal and intermediate water surface freeboard requirements. Allowances for camber and additional factors are added after the 3 feet of minimum freeboard is deemed satisfactory. Figure 1 shows the steps that are recommended for the analysis.

For some dams, even large, high-hazard structures, a freeboard analysis may be quite simple using the following checks.

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[2]Derivation of site-specific wind events and their probability are explained in appendix II.

Two relatively conservative checks can be performed initially to determine if 3 feet of minimum freeboard is sufficient to prevent overtopping. For new Reclamation embankment dams, these two checks are the first steps of the analysis. If it is found that the dam design fails either of these two checks, then the probabilistic method should be used to evaluate the adequacy of 3 feet or more minimum freeboard. The probabilistic method described in appendix II is a more rigorous analysis that evaluates the probability of overtopping various target crest elevations from all possible water surfaces below the maximum. Finally, whether or not the probabilistic method is used, the evaluator should 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 may be required to quantify these wind velocities. These wind events can be specified in the probabilistic method as minima to associate with the PMF or an additional amount of freeboard could be added to the minimum freeboard to account for such extreme possibilities.

The simple checks to evaluate 3 feet (0.9 m) of minimum freeboard are as follows:

(1) The first check is used to see if the amount of flood storage plus 3 feet of minimum freeboard is enough to protect the dam from overtopping should waves build up from a 100-mi/h (160-km/h) wind while the reservoir is operating normally, below flood storage. Given the fetch of the reservoir at the top of active conservation capacity (to compute fetch, see fig. 2 or appendix II.a.) and the slope of the upstream face of the dam, use figure 3 to derive the amount of runup that can be expected from a 100-mi/h wind event. If the distance between the top of the active conservation capacity and the top of the dam (this distance includes 3 feet (0.9 m) of minimum freeboard) is greater than the runup determined from figure 3, then the dam design passes this check and the amount of flood storage plus 3 feet of minimum freeboard is adequate 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 3 feet of minimum freeboard is enough to protect the dam from overtopping in the event of the PMF and winds that would typically occur during a time period equal to the duration that the reservoir water surface is near maximum. Information that is needed for this includes:

- Reservoir Elevation versus Time graph derived from routing the PMF.
- Hourly Probability of the Wind (P_{W_u}) versus Wind Velocity derived from the analysis of wind data part of the probabilistic method (see app. 2.).

Horizontal lines are drawn across the Reservoir Elevation versus Time graph 2 and 4 feet (0.6 and 1.2 m) below the MWS. The duration that the reservoir is within 2 feet (0.6 m) of the MWS is equal to the length of the upper horizontal line below the flood curve. The inverse of this duration is equal to the hourly probability of the largest wind event that may typically occur while the reservoir is within 2 feet of the MWS. A wind velocity is taken from the P_{W_n} versus Wind Velocity curve. This wind velocity and the reservoir's fetch (compute fetch at a reservoir elevation equal to the MWS) are plotted on figure 5. If the point plots below the curve representing the upstream slope of the dam, then less than 5 feet (1.5 m) of runup will be generated by this wind velocity and 3 feet (0.9 m) of minimum freeboard is adequate to protect the dam from this wind event when the reservoir is 5 feet below the crest. The duration represented by the horizontal line drawn 4 feet (1.2 m) below the MWS on the Reservoir Elevation versus Time graph is used to find the P_{W_n} and maximum wind velocity that may typically occur while the reservoir is 4 feet from the MWS. Similarly, the wind velocity associated with the inverse of this duration and the reservoir's fetch are plotted on figure 6. If this point also plots below the line representing the upstream slope of the dam, then less than 7 feet (2.1 m) of runup will be generated by this wind velocity and 3 feet (0.9 m) of minimum freeboard is adequate to protect the dam from this wind event when the reservoir is 7 feet below the crest. Points plotting below the lines representing the upstream slope of the dam on figures 5 and 6 indicate that the dam design passes this second check and typical winds would not generate waves to overtop the dam during the PMF with 3 feet of minimum freeboard.

If the dam design passes both of the above checks, then no other method needs to be used to calculate freeboard. If there is no reason to believe that exceptionally high wind velocities (those not necessarily typical) would be blowing while the water surface is within 4 feet (1.2 m) of the maximum water surface during the largest flood events, then these two checks are sufficient for the freeboard analysis. A minimum freeboard equal to 3 feet (0.9 m) above the MWS is sufficiently safe protection against overtopping due to wind generated waves. On the other hand, if the dam design fails either of the above two checks, then the probabilistic method should be used according to appendix II. It may still show that a crest elevation 3 feet above the MWS would provide for an acceptable design probability. The above two simple checks have been developed conservatively and the probabilistic method is a more precise design tool. In summary, the scope of a freeboard analysis for new Reclamation embankment dams can be described by the flow chart given on figure 1.

c. Top of impervious zone. - To preclude development of seepage caused by the maximum reservoir water surface, the top of the impervious zone must be designed so that after settlement it is at the elevation of the MWS plus wind setup (but not 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 be provided to control leakage through cracks or frost lense separations, or the reservoir water surface must be kept below the depth of such effects. The most economical solution should be used.

3. Existing concrete and embankment dams. - A freeboard analysis for an existing dam attempts to identify hydrologic or hydraulic deficiencies that might lead to failure of the dam. In the case of an embankment dam, if the MWS of the reservoir is close enough to the dam crest such that wind generated runup and setup would wash over or if the MWS 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. - Normally, 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 (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 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 is also 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.

- Vegetation and surface roughness. - In general, vegetation will act to prevent erosion unless trees and other obstructions are intermittent enough to cause turbulence or a flow concentration. Surface roughness will retard the flow of water or limit the washover of waves across the top of the dam.

- Permeability of surface materials. - In general, a permeable, unsaturated surface will impede wave runup and flow velocity more readily than an impermeable, saturated surface. During flood events which would cause overtopping, it can be assumed that the surface of the embankment dam is saturated to the point where water would rather run off than infiltrate the dam. If water flows over a very permeable embankment crest, seepage may accelerate destruction.

- 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 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. 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 roughnesses. The time at which failure is initiated is the time during the PMF when the critical depth of overtopping is reached.

- Wind. - The two checks that are given in section I.B.2.b. can be used to verify if there is enough freeboard to prevent overtopping caused by wind generated waves. The first check uses figure 3 with the fetch of the reservoir at the top of active conservation capacity (to compute fetch, see fig. 2 or app. 2.a.) and the slope of the upstream face of the dam to derive the amount of runup that can be expected from a 100-mi/h (160-km/h) wind event. If the distance between the top of active conservation capacity and the top of the dam is greater than the runup that is determined from figure 3, 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 PMF Reservoir Elevation vs. Time curve at elevations 3, 5, and 7 feet (0.9, 1.5, and 2.1 m) below the crest elevation to derive the maximum durations that the reservoir is at or above those elevations. The inverse of the duration is the probability of the wind event that correlates to the wind velocity on the P_{W_a} vs. Wind Velocity curve. Figures 4, 5, and 6 are used, respectively, to determine if wind-generated waves may wash over the dam crest during extreme flood events. If it is determined that waves may wash over the crest even though the maximum 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.

Freeboard for an existing concrete dam is not as critical as it is for an embankment because concrete is not likely to wash away if the dam is overtopped. In the case of a concrete dam, failure will almost always depend on the ability of the abutments and foundation to survive the force of the water flowing over the concrete dam. Although some existing concrete structures may be in such a poor condition that overtopping may cause extensive damage, a failure that would threaten the safety of life and property downstream is not likely to be the result of this damage. However, the geology of the damsite should be carefully examined by engineers and geologists to make a judgment on the potential for erosion or plucking of the materials. Fault zones and other types of discontinuities, weathered rock, friable or weakly cemented material, and soft intact rock are a few geologic types that may not survive well during overtopping. If erosion of the abutments or foundation leads to undercutting of the concrete structures, failure may result.

A Safety of Dams evaluation may be performed when it is decided that insufficient freeboard would lead to dam failure.

The freeboard requirements for an existing dam may be different than the requirements for a new dam. The incremental costs for raising a new dam a few feet during design would be minimal compared to the costs for doing the same to an existing dam. Expenses would 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 changing the spillway design to accommodate larger floods is extremely costly for an existing dam while it may have little impact on the cost for a new dam under design. Not only are all of these costs higher for an existing dam, but the benefits from raising an existing dam a few feet are not as great as the benefits from building a dam under design higher by an equal amount. The cumulative benefits from the raising (in terms of reduced risk cost) are lower for an existing dam because of its reduced remaining service life. Thus, a risk cost analysis could provide a basis for selecting an amount of freeboard different for an existing dam from the amount of freeboard that would be acceptable for new dams. Such a study may be needed before a decision to raise an existing dam would be warranted. The decision to modify an existing dam to provide more freeboard should consider the above factors in addition to the policies described in subsection I.B.2., which outline initial freeboard requirements for new dams.

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 spillway and outlet works should be better known than at the time of original design because of 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 embankment dam cases.

4. <u>Parapet walls</u>. - A standard 3.5-foot (1.1-m) high parapet wall provides all of the freeboard that is required for concrete dams. This wall is intended to keep waves from washing over the dam during high reservoir water levels.

Use of parapet walls to provide freeboard allowances for embankment dams may be considered on a case-by-case basis. The parapet wall ordinarily only replaces the portion of the freeboard needed to prevent overtopping from wave runup but, in some cases, can be used to retain the uppermost flood storage of the extreme flood events for a very short time. When used, the following safeguards must be met:

- The parapet wall should be adequately tied into the impervious zone and proper zoning provided to prevent piping.

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

- Consideration must be given to hydrostatic and hydrodynamic (wave) loads, drainage off of the crest around or through the wall, adjoining and sealing the wall units together with each other and each end of the dam, maintenance and esthetics.

II. FACTORS WHICH INFLUENCE FREEBOARD

A. Floods

Flood characteristics such as the shape of the inflow design flood (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.

Ordinarily, the IDF is taken to be the PMF. It can arbitrarily be assigned a mean return period equal to 10,000 years. This is the basis to which other probability relationships having to do with flood levels are established. In other words, 10,000 years can be considered a reference point to which other flood or combined flood and wind probabilities can be compared in order to know if the occurrence of some event would be more or less likely than an occurrence of the PMF. Therefore, once every 10,000 years should be taken as a relative, rather than an absolute, probability.

In addition to the PMF, lesser floods are evaluated for the full probabilistic calculations. The 95-percent confidence limit of the annual flood frequency curve is generally used to derive the magnitudes of the peak discharge (inflow) of floods lesser than the PMF.

Hydrologic uncertainty involves the condition of being unsure about the value of some of the parameters used in hydrologic computations. The value of some parameters must be inferred from a random sample which might not, and probably does not, represent all of the future possibilities accurately. Consequently, estimates of parameters contain some degree of uncertainty, and resulting errors do not necessarily compensate each other. The impact of errors in one direction due to uncertainty can be quite different from the impact of errors in the other direction. The confidence level in computing the IDF in terms of reliability of data for developing the design storm and snowmelt runoff and other hydrologic parameters are factors that could impact on freeboard determinations. This becomes evident when the adequacy of existing IDF's is reevaluated on the basis of all currently available data. This usually results in increased flood magnitudes. Problems related to inadequate hydrologic data should be resolved to the extent feasible during derivation of the IDF. If conditions exist that justify including a freeboard allowance for hydrologic uncertainty, the value should be based on the judgment of those responsible for developing the IDF.

B. Wind Setup and Wave Runup

Wind setup and wave runup are often the predominant factors in the determination of all types of freeboard. Wind-generated wave heights and wave runup are probably the most studied and understood factors of those which influence freeboard. Much of the study has been carried out and reported by the U.S. Army Corps of Engineers [1].[3]

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.

A probabilistic method to account for wind setup and wave runup which would occur during flood events and control freeboard is presented in appendix II. The target crest elevations mentioned in the appendix pertain to those that would be adequate to prevent overtopping by runup and setup generated by winds over a water surface having a specified probability. The method considers all possible combinations of wind and flood events. It provides the basis upon which freeboard for wind-generated waves and setup is computed should either of the two conservative checks for freeboard show that 3 feet above the MWS is not adequate (see fig. 1).

C. 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 may be a factor to consider when performing flood routings.

D. 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.

1. Ungated spillways. - Ungated spillways are less affected by and, in most cases, are free from improper maintenance and operation problems. Freeboard allowance for malfunction is not required for most dams with ungated spillways except for those reservoirs which depend on the outlet works to discharge a large portion of the floodflows. When shaft

[3]Numbers in brackets refer to references, section III.

spillways are used, particular attention should be given to potential loss of discharge capacity as a result of plugging the inlet by debris. 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.

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.

3. Gated spillways. - Even with the 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. In determining freeboard allowances for malfunction of gated spillways, the following site-specific conditions should be considered:

- Reliability of gate operations from actual experience.
- Sensitivity of gate operation to IDF characteristics and flood-storage capability.
- Training, experience, and physical condition of the dam tender.
- Distance between the dam and dam tender's residence.
- Availability of a substitute dam tender.
- Road condition and accessibility of the gates and control center during floods.
- Size and complexity of the gate structure and its operation.
- Number of gates. - Chances of mechanical-type failure adversely affecting outflow are usually reduced as the number of gates increases, especially in going from one to two or from two to three gates.
- Reliability of commercial and auxiliary power supplies.
- Availability of emergency materials and equipment.
- Availability of warning and communications systems.
- Remoteness of the damsite.

The designer should make an assessment of the foregoing site-specific conditions, making quantitative evaluations where possible. For example, determine the change in MWS resulting from failure of one of three gates to open. For some reservoirs with large surface areas, the change in MWS 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.

In most cases, a minimum additional allowance of 1 foot of freeboard is considered necessary to account for the malfunction of gated spillways; however, 3 or 4 feet may be required in some cases where a valid combination of adverse conditions could reasonably be expected to occur.

E. 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. The amount and type of fault displacement and energy and frequency spectrum are factors influencing wave generation by these two events, respectively. The magnitude of waves resulting from landslides is affected by the volume, speed, and geometry of the slide mass. Wave runup is calculated using the same method as for wind-generated waves.

Seiches or earthquake-generated water waves can develop when the resonance of a reservoir equals the resonance of the seismic shaking. Also, a wave develops in a reservoir when water rushes in to fill a "void" caused by faulting in the form of either vertical displacement or tilting under or adjacent to the reservoir. A large seiche wave (1 ft or more) is considered very unlikely because the duration of shaking is almost always less than what is needed to get a large oscillating wave started. Very often 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 fault displacement wave 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. Methods and theories of seiche analysis can be found in references 2, 3, and 4. For fault displacement waves, applied hydraulics are used to evaluate wave height and propagation.

Rapid reservoir drawdown, earthquakes, rain, and other factors may trigger landslides in a reservoir. 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. Of these three, velocity is the most critical. The height of a reservoir wave from a landslide can vary from a minimum disturbance to a "Vaiont 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 landslide-generated water waves" [6]. Another useful paper, 14th ICOLD Conference in Rio de Janeiro (1982), is "Prediction of Landslide-generated Water Waves" by C. A. Pugh and D. W. Harris [7].

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. 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 which may indicate a need for freeboard or other mitigating measures.

F. 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. This distance above the design crest camber usually ranges from 0 at the abutments to a finite amount in the middle reach of the dam. Requirements and calculation methods for camber are given in the Design Standard No. 13, Embankment Dams, Chapter 9 [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.

G. Additional Factors

Additional factors to consider in freeboard analysis include climate, downstream considerations, the type of spillway, and remoteness of the damsite.

III. REFERENCES

[1] "Wave Runup and Wind Setup on Reservoir Embankments, "Engineering Technical Letter (ETL) No. 1110-2-221, U.S. Army Corps of Engineers, Washington, D.C., 1976.

[2] Murty, T. S., "Seismic Sea Waves Tsunamies," Bulletin No. 198, Fisheries Research Board, 1977.

[3] Richter, C. F., Elementary Seismology, W. H. Freeman and Co., San Francisco, California, 1958.

[4] Wilson, B. W., "Seiches," V. T. Chow (ed.), p. 94, Advances in Hydrosciences, New York Academic Press, 1972.

[5] Kiersch, George A., "Vaiont Reservoir Disaster", ASCE Civil Engineering, vol. 34, No. 3, March 1964.

[6] Slingerland, R. L., and B. Voight, "Occurrences, Properties, and Predictive Models of Generated Water Waves," Rockslides and Avalanches, vol. 2, Elsevier Science Publishing, 1979.

[7] Pugh, C. A., and D. W. Harris, "Prediction of Landslide-generated "Waterwaves" 14th ICOLD Conference (paper), Rio de Janeiro (1982), Bureau of Reclamation, Denver, Colorado, 1981.

[8] Design Standard No. 13, Embankment Dams, Chapter 9, "Embankment and Foundation Consolidation Deformation Analysis," Bureau of Reclamation, Denver, Colorado (unpublished draft document).

[9] "Determining Sheltered Water Wave Characteristics," ETL No. 1110-2-221, U.S. Army Corps of Engineers, Washington, D.C., 1984.

[10] Design of Small Dams, Bureau of Reclamation, Denver, Colorado, third edition, 1987.

[11] "Waves in Inland Reservoirs - Summary Report in CWI Projects CW-164 and CW-165," Technical Memorandum No. 132, U.S. Army Corps of Engineers, Washington, D.C., 1962.

[12] Saville, Thorndike, Jr., Elmo W. McClendon, and Albert L. Cochran, "Freeboard Allowances for Waves in Inland Reservoirs," Proceedings ASCE, May 1962.

[13] Shore Protection Manual, vols. I and II, Coastal Engineering Research Center, Fourth Edition, U.S. Army Corps of Engineers, 1984.

[14] Wind Energy Resource Atlas, Battelle Pacific Northwest Laboratory, vols. 1, 2, 7, 8, and 9, 1980 and 1981.

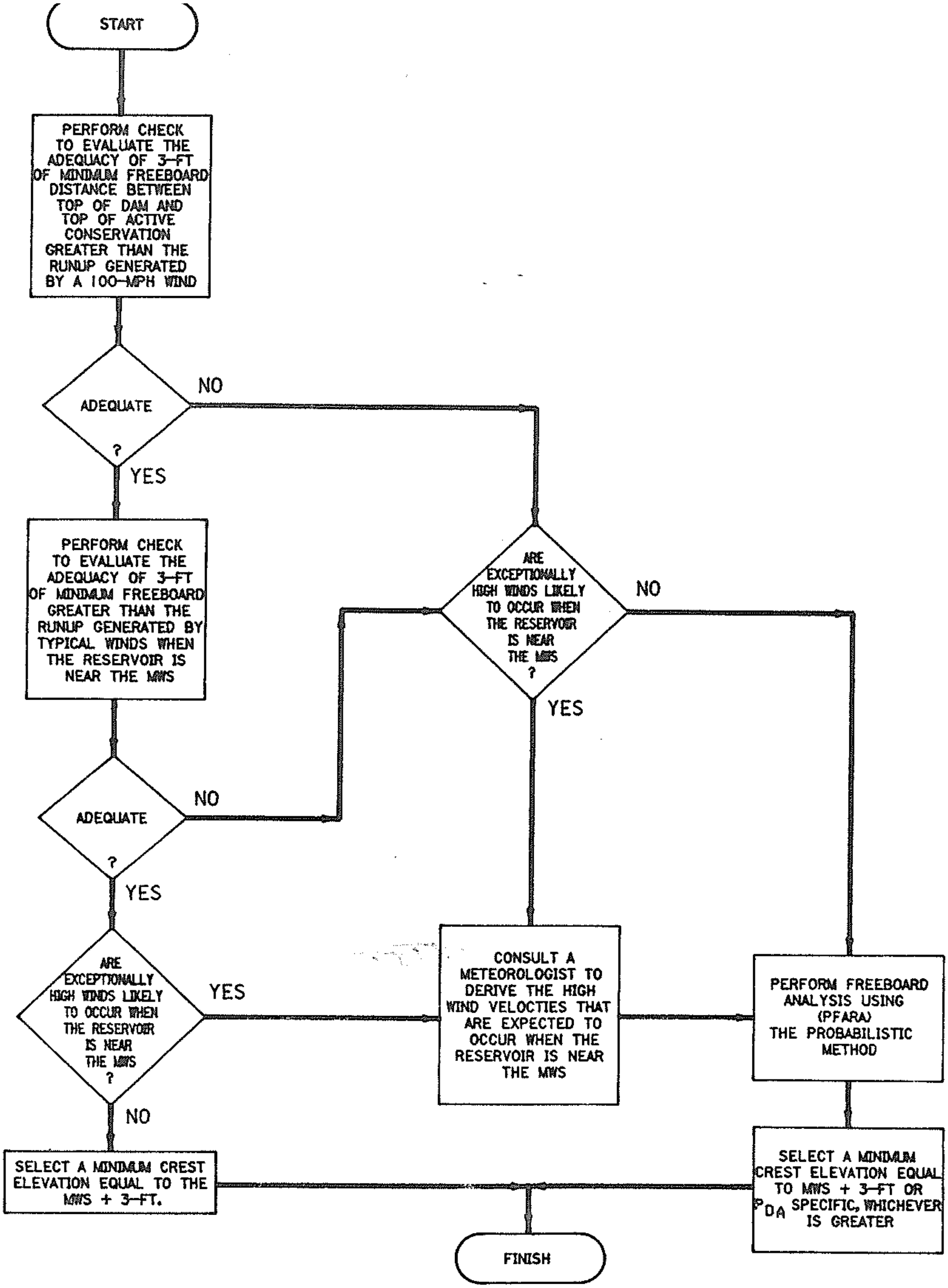
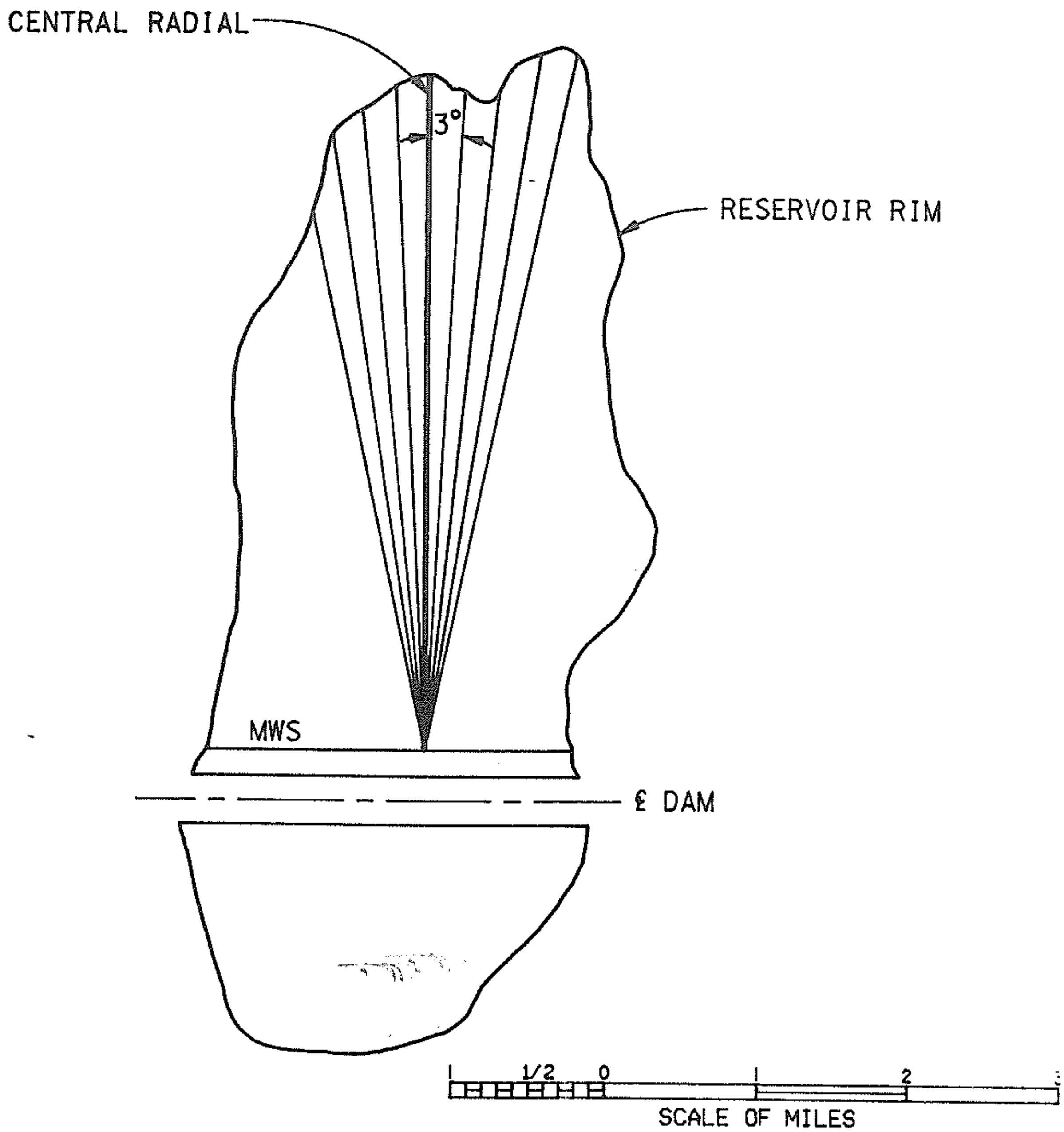


Figure 1. - Flowchart for freeboard analysis to protect new embankment dams from overtopping failure due to wind-generated waves.



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

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

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

Figure 2. - Fetch calculation. (Units in miles to match text.)

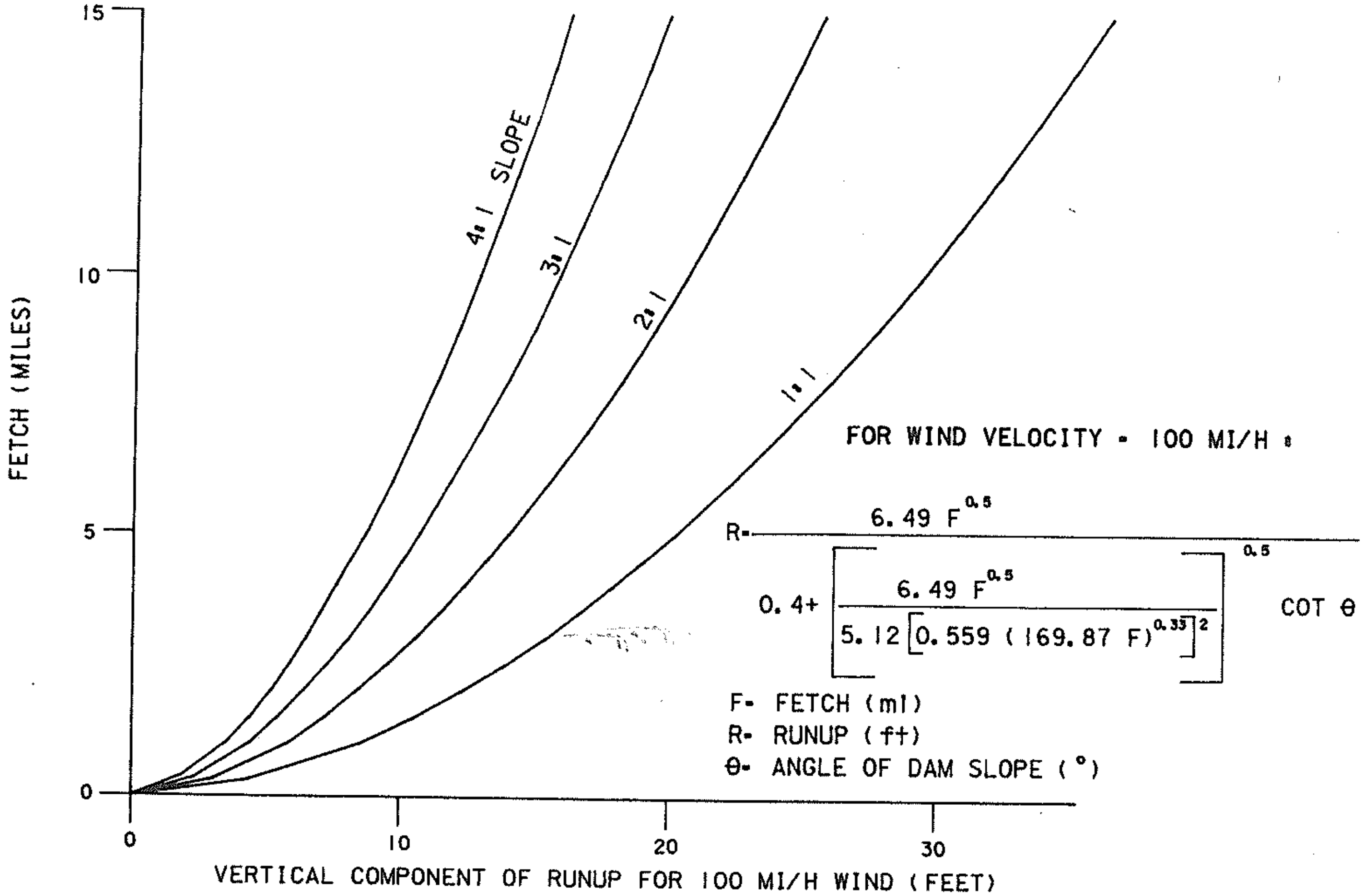


Figure 3. - Vertical distance between the reservoir water surface and the dam crest that is required to prevent overtopping for various slopes of the upsteam dam face and 100-mi/h wind velocity.

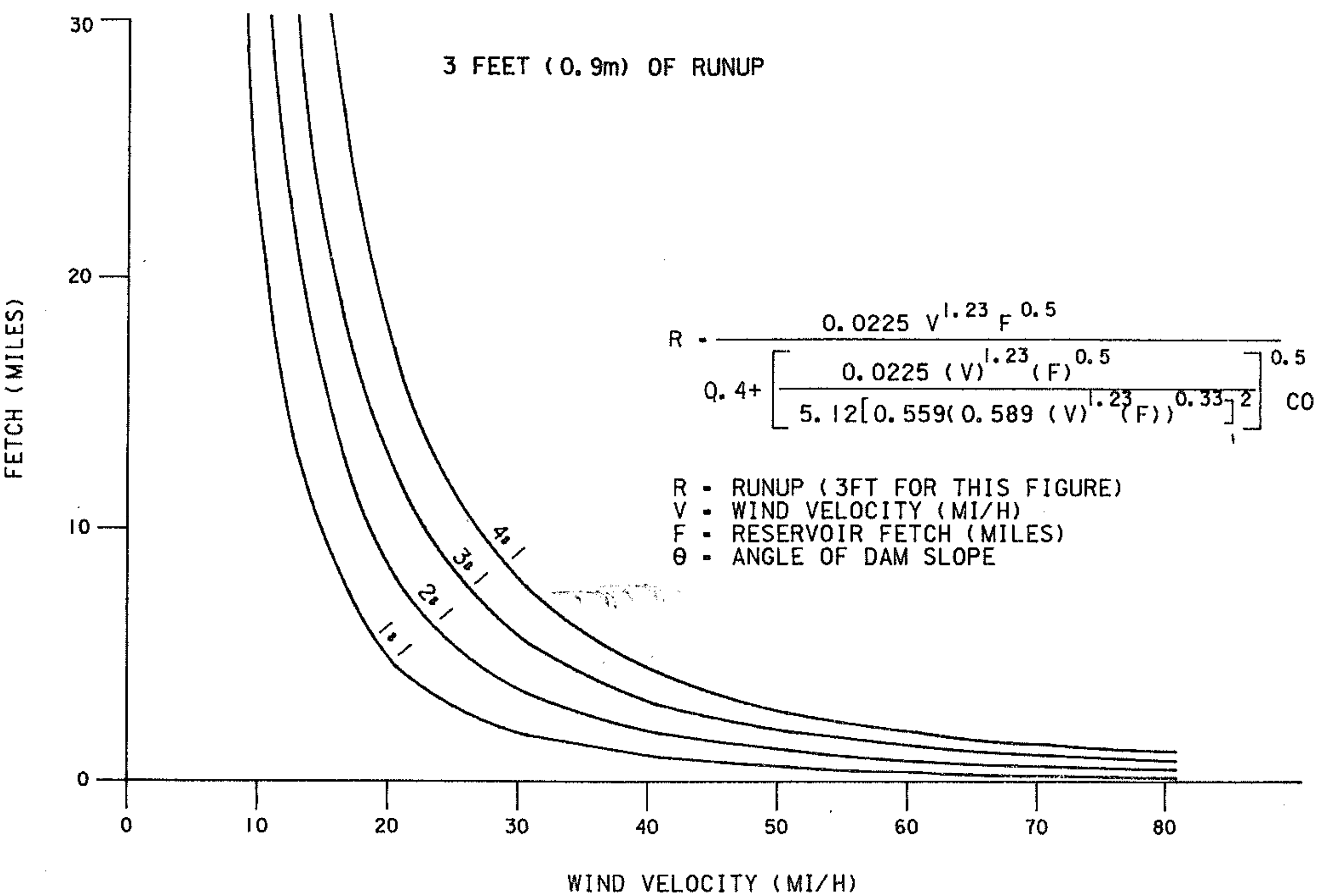


Figure 4. - Wind/fetch relationship to produce 3-ft runup for various slopes of the upsteam dam face.

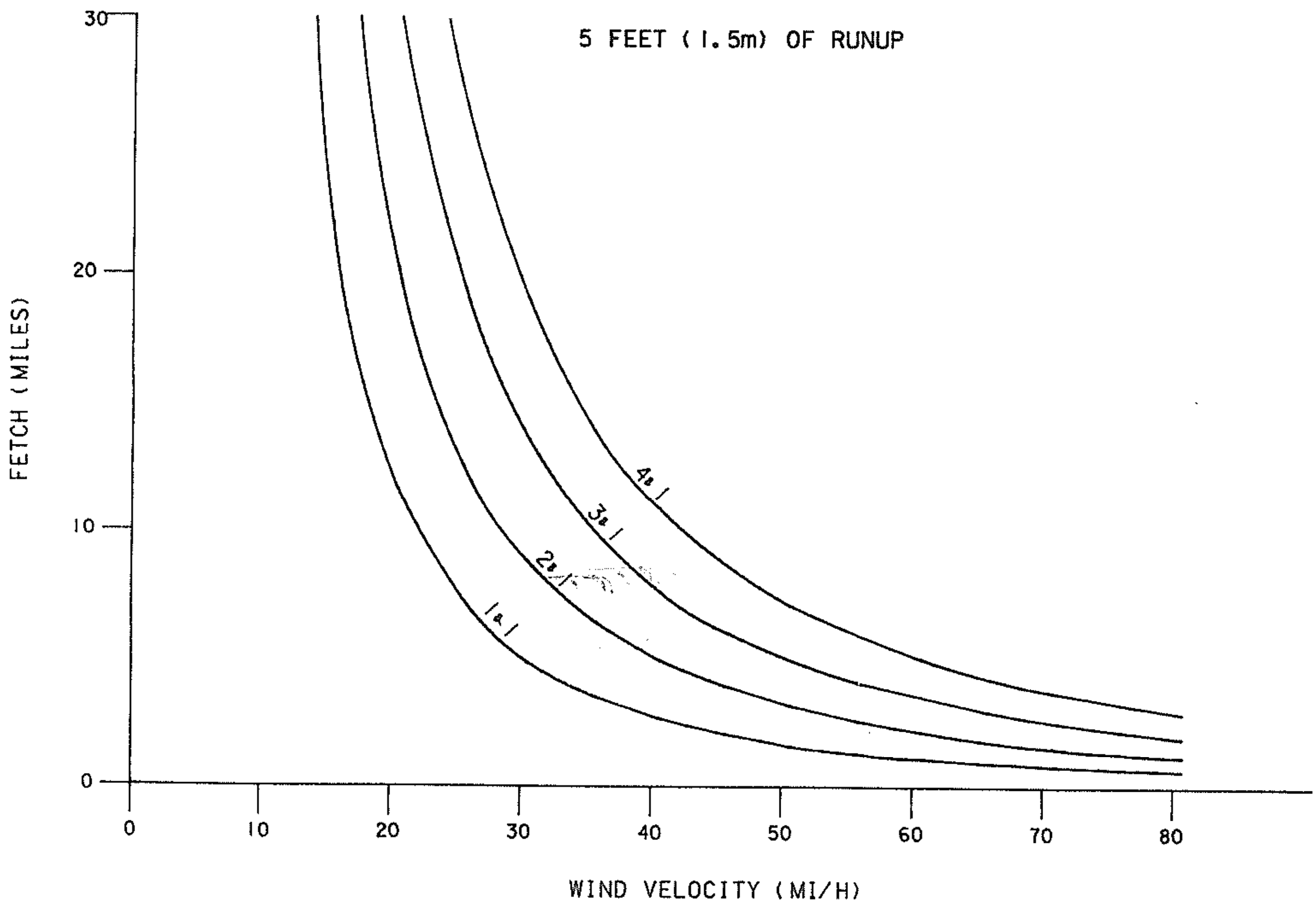


Figure 5. - Wind/fetch relationship to produce 5-ft runup for various slopes of the upstream dam face.

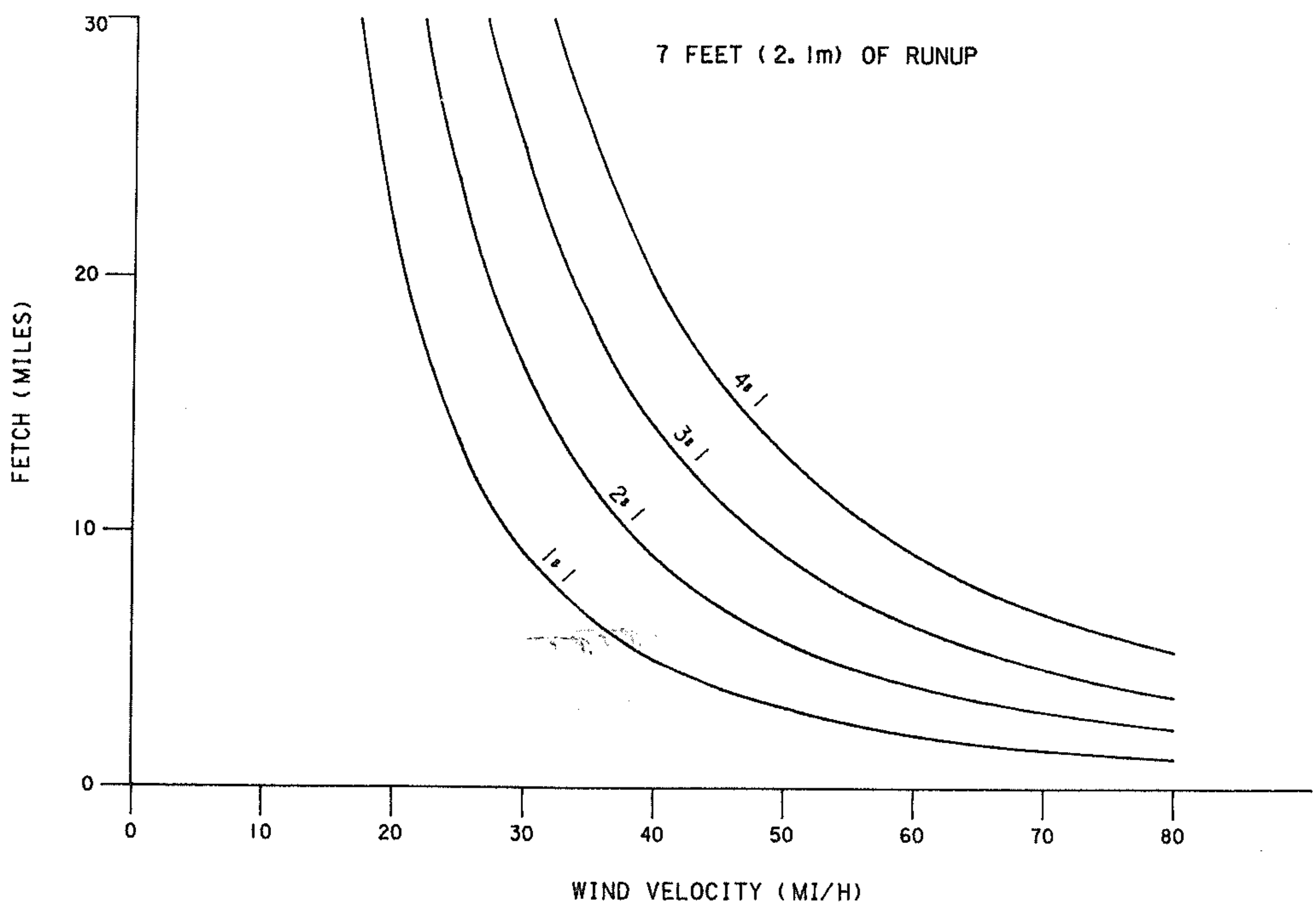


Figure 6. - Wind/fetch relationship to produce 7-ft runup for various slopes of the upstream dam face.

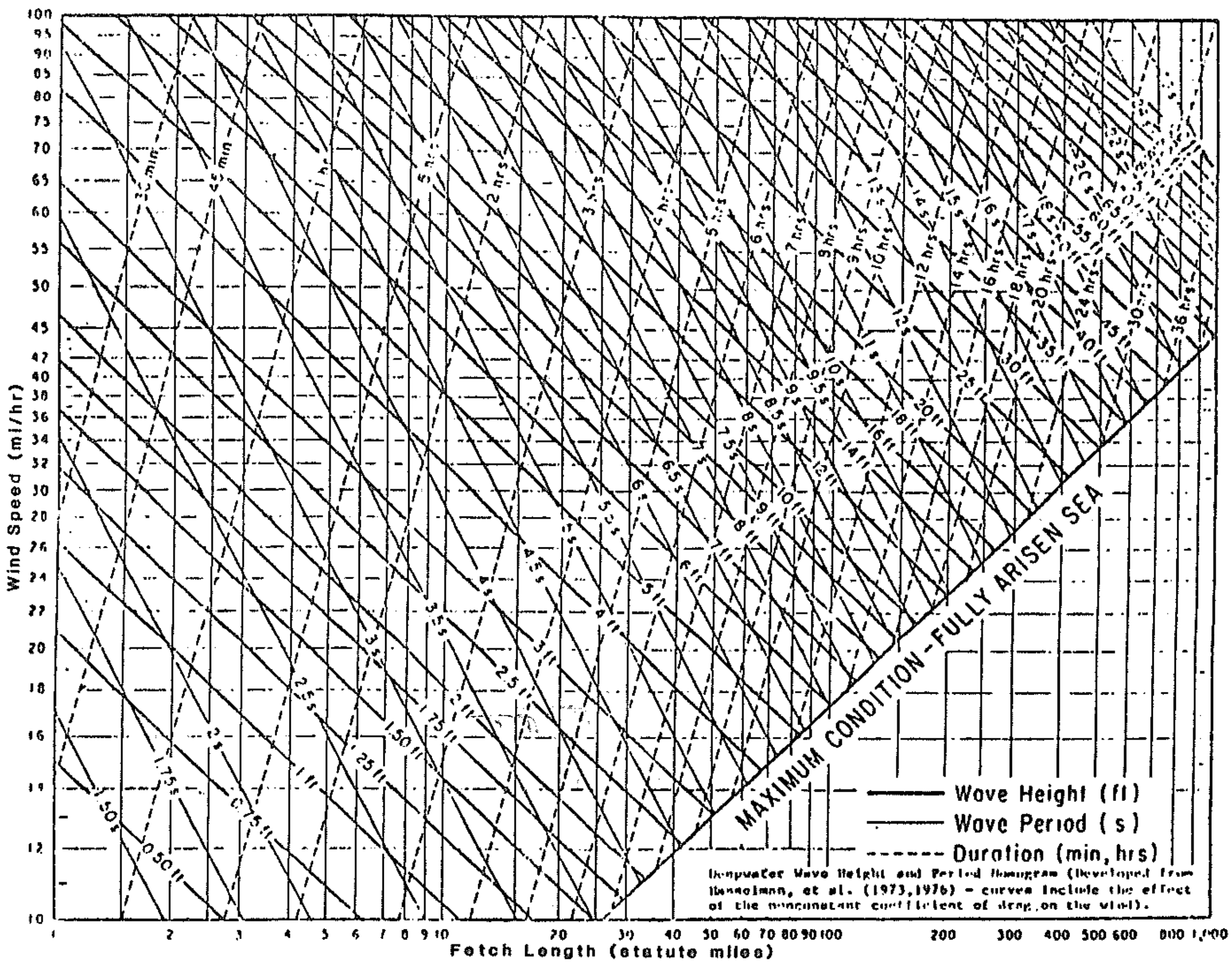


Figure 7. - Generalized correlation of significant wave heights (H_s) with related factors - Deep water conditions.

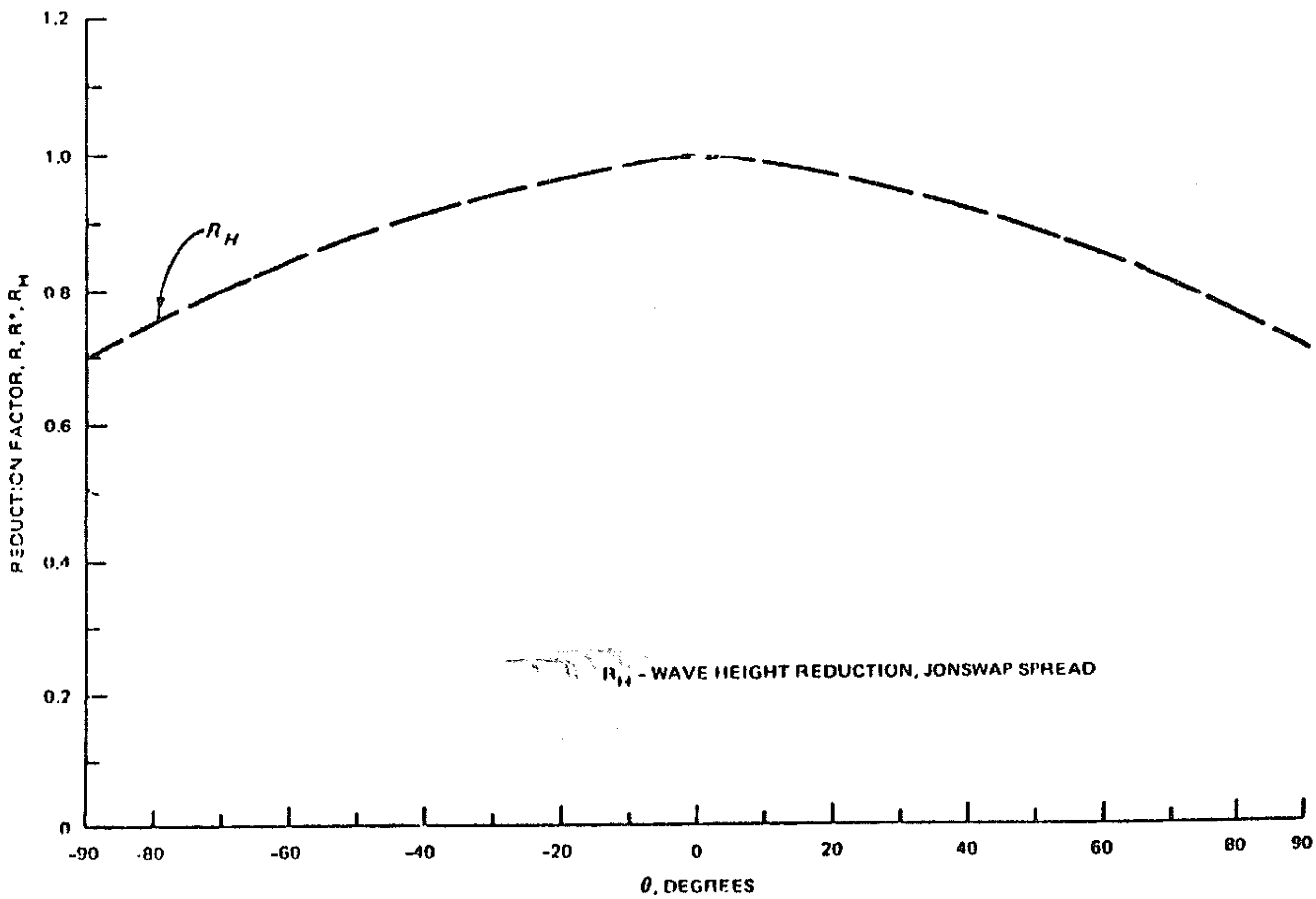
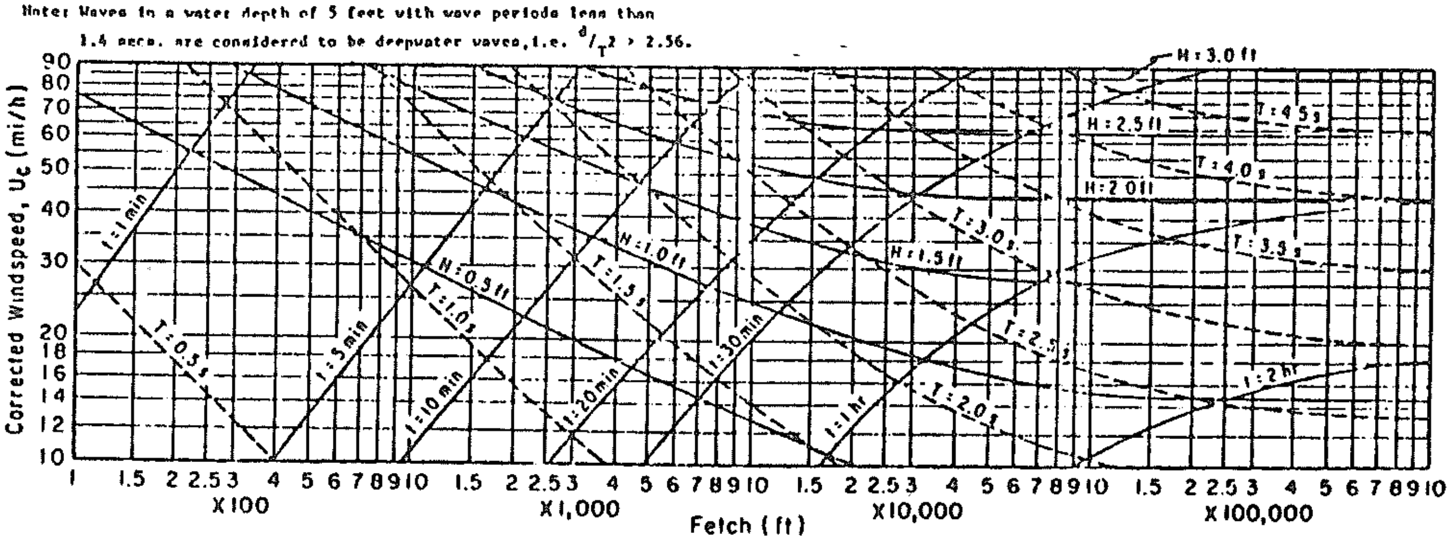
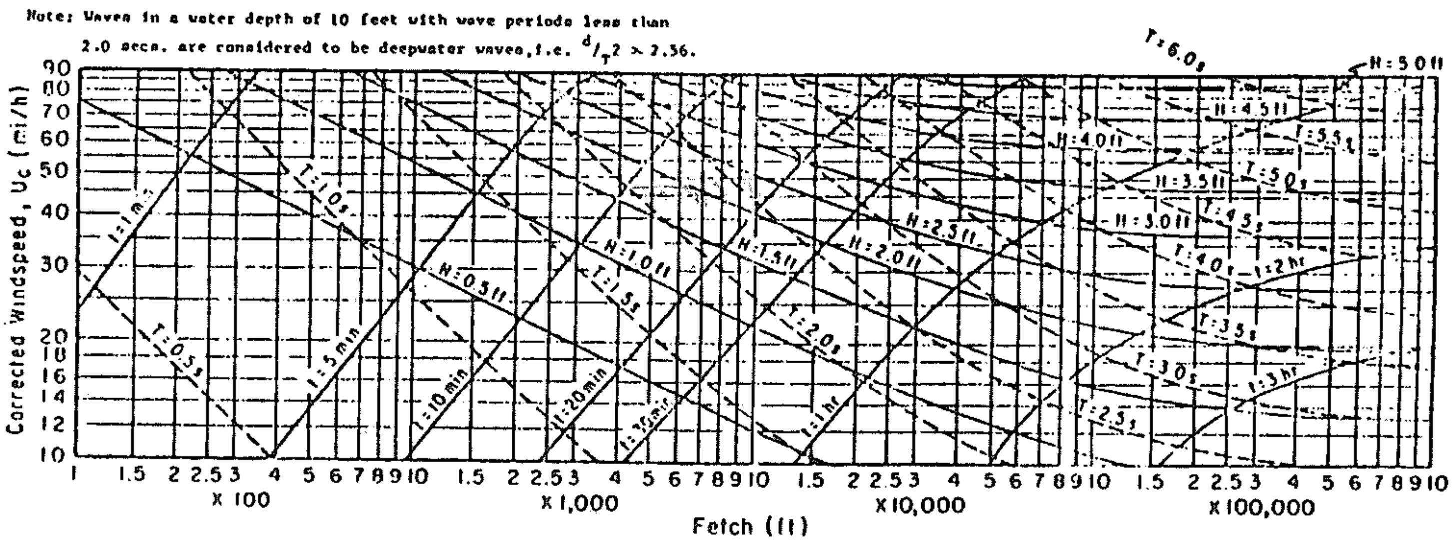


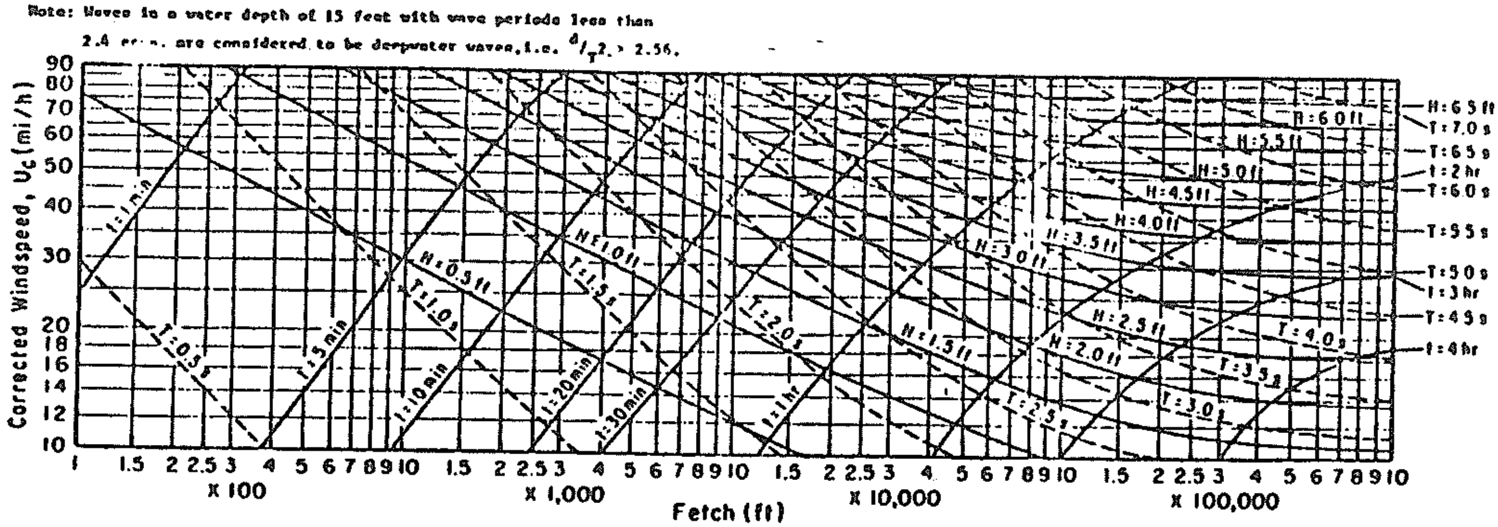
Figure 8. - Wave height reduction due to angular spread.



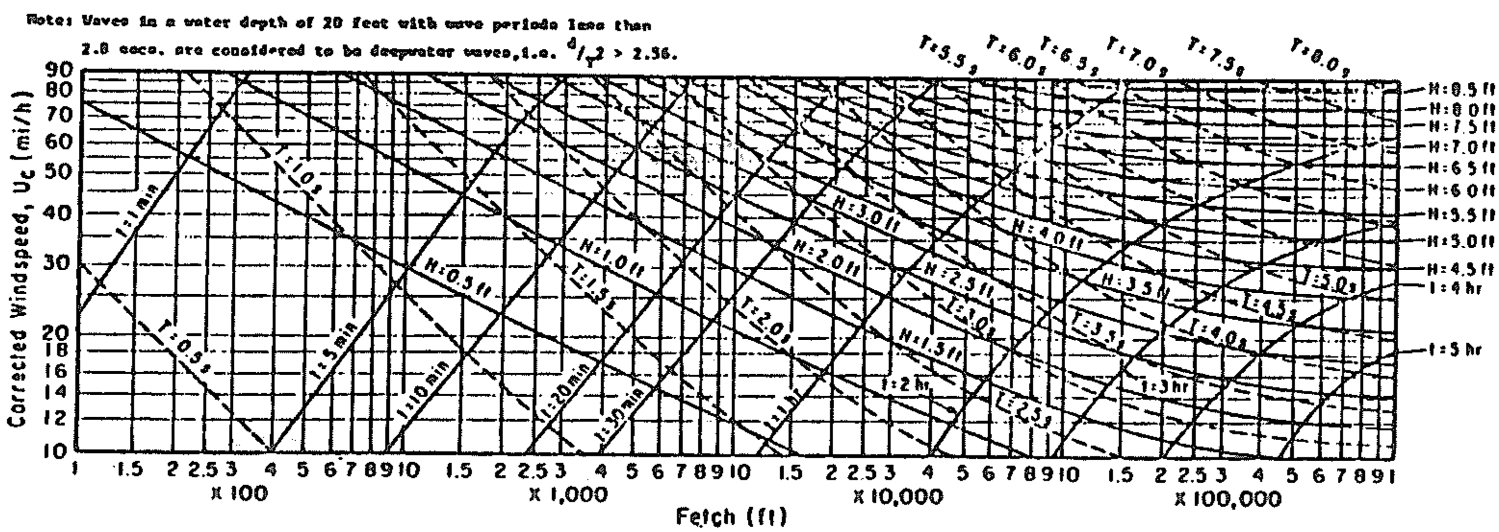
Figures 9. - Forecasting curves for shallow-water waves. Constant depth = 5 feet.



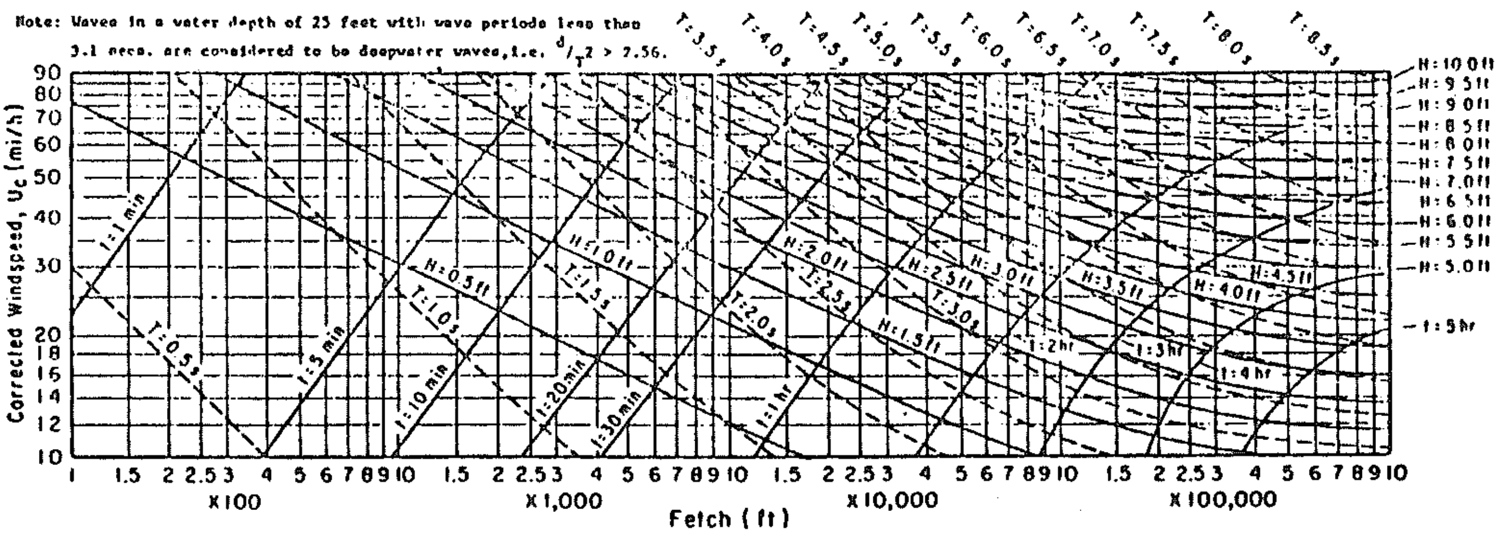
Figures 10. - Forecasting curves for shallow-water waves. Constant depth = 10 feet.



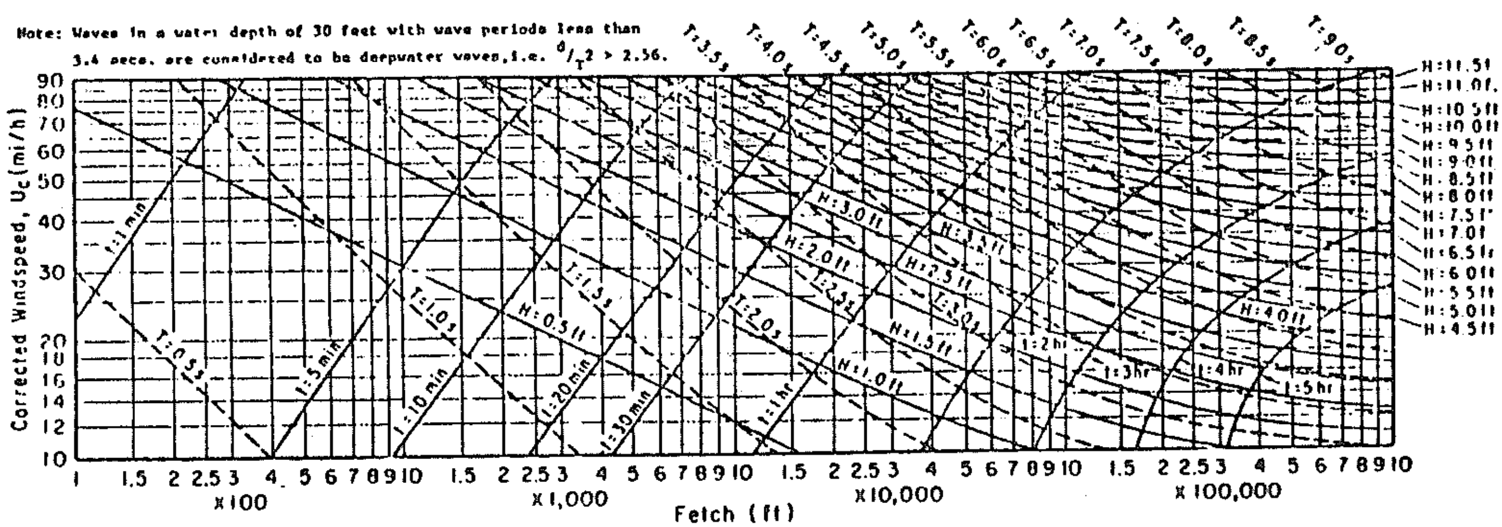
Figures 11. - Forecasting curves for shallow-water waves. Constant depth = 15 feet.



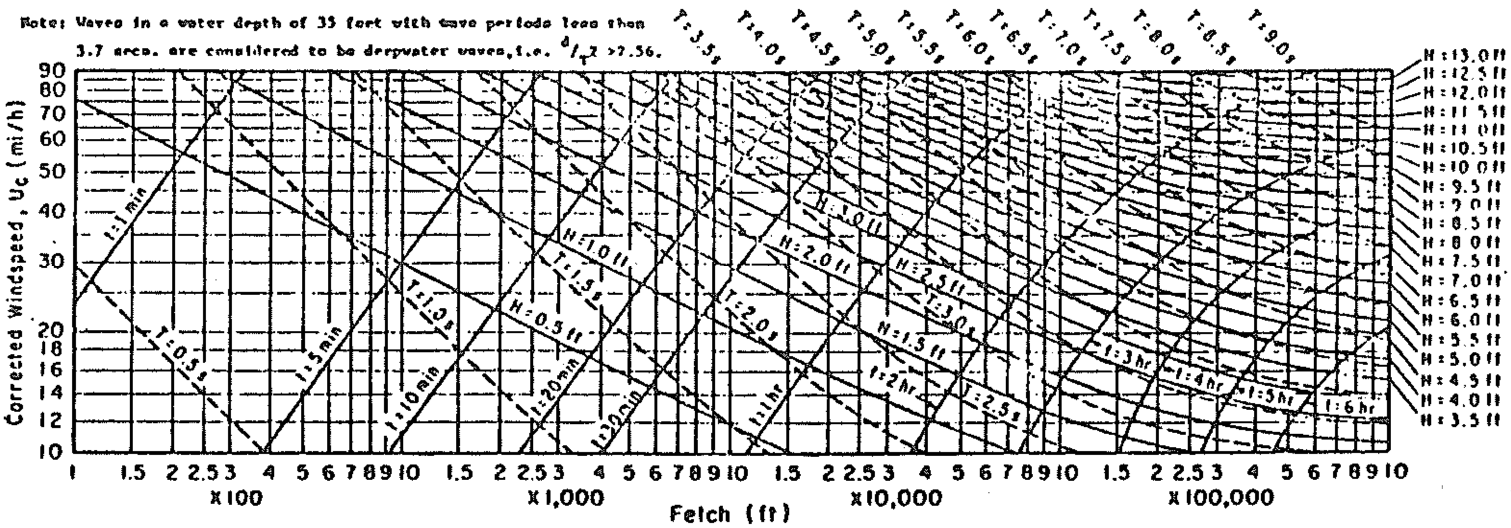
Figures 12. - Forecasting curves for shallow-water waves. Constant depth = 20 feet.



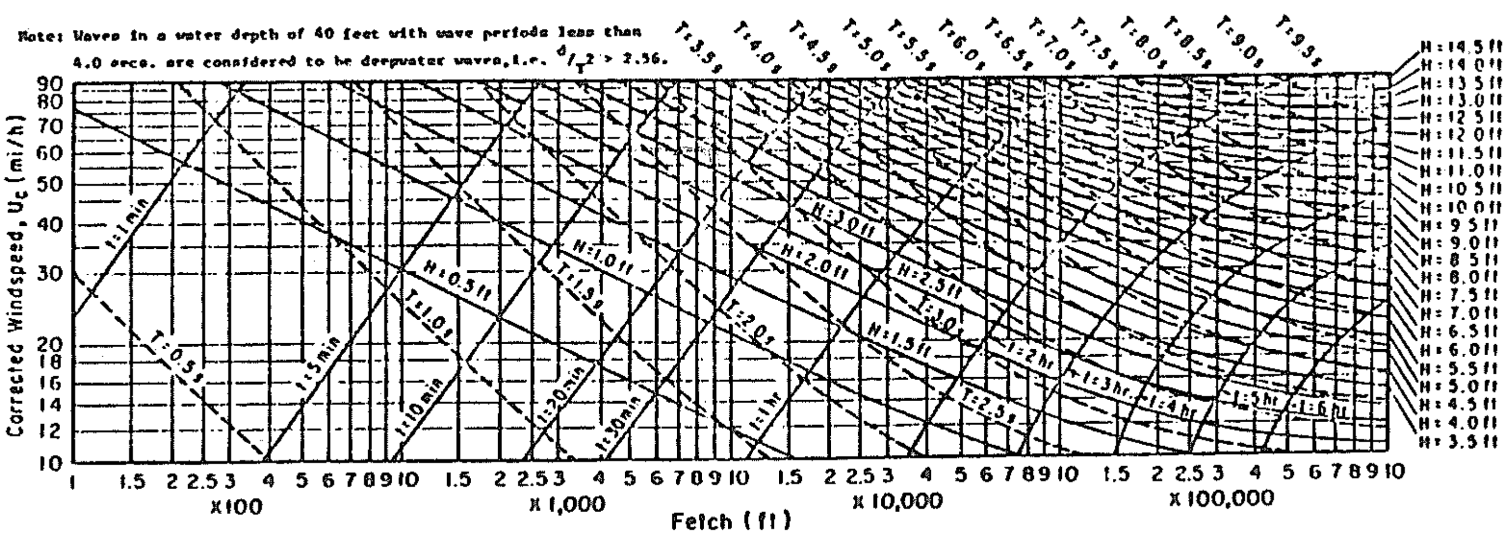
Figures 13. - Forecasting curves for shallow-water waves. Constant depth = 25 feet.



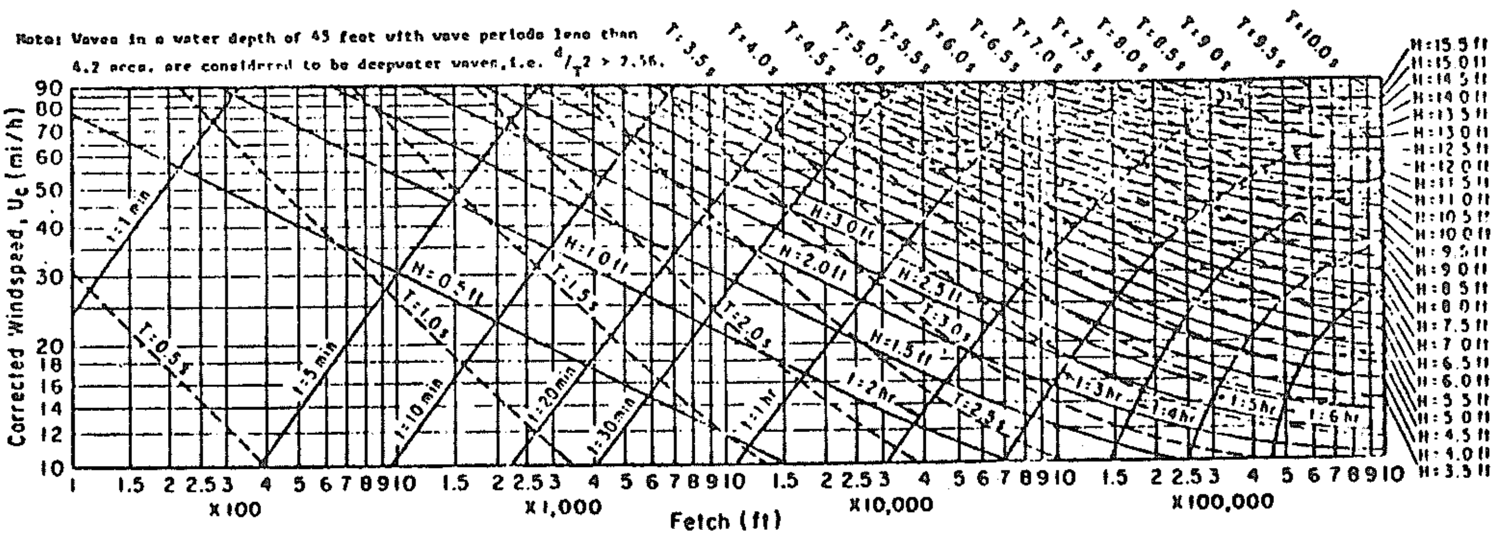
Figures 14. - Forecasting curves for shallow-water waves. Constant depth = 30 feet.



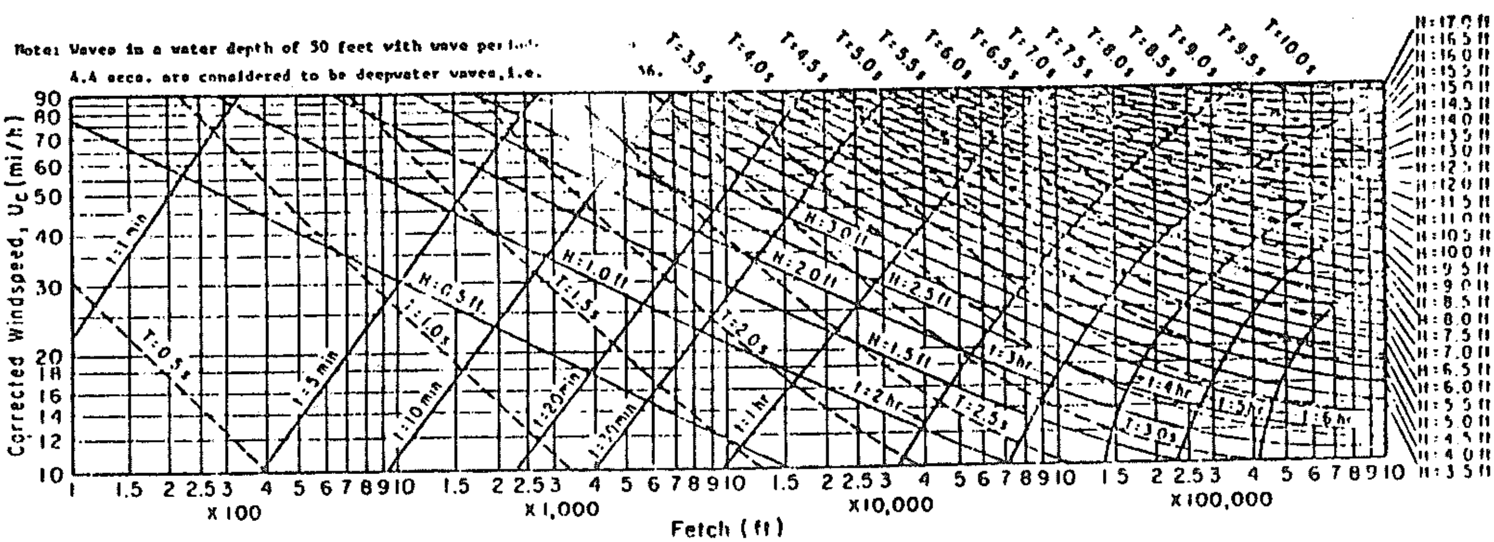
Figures 15. - Forecasting curves for shallow-water waves. Constant depth = 35 feet.



Figures 16. - Forecasting curves for shallow-water waves. Constant depth = 40 feet.



Figures 17. - Forecasting curves for shallow-water waves. Constant depth = 45 feet.



Figures 18. - Forecasting curves for shallow-water waves. Constant depth = 50 feet.

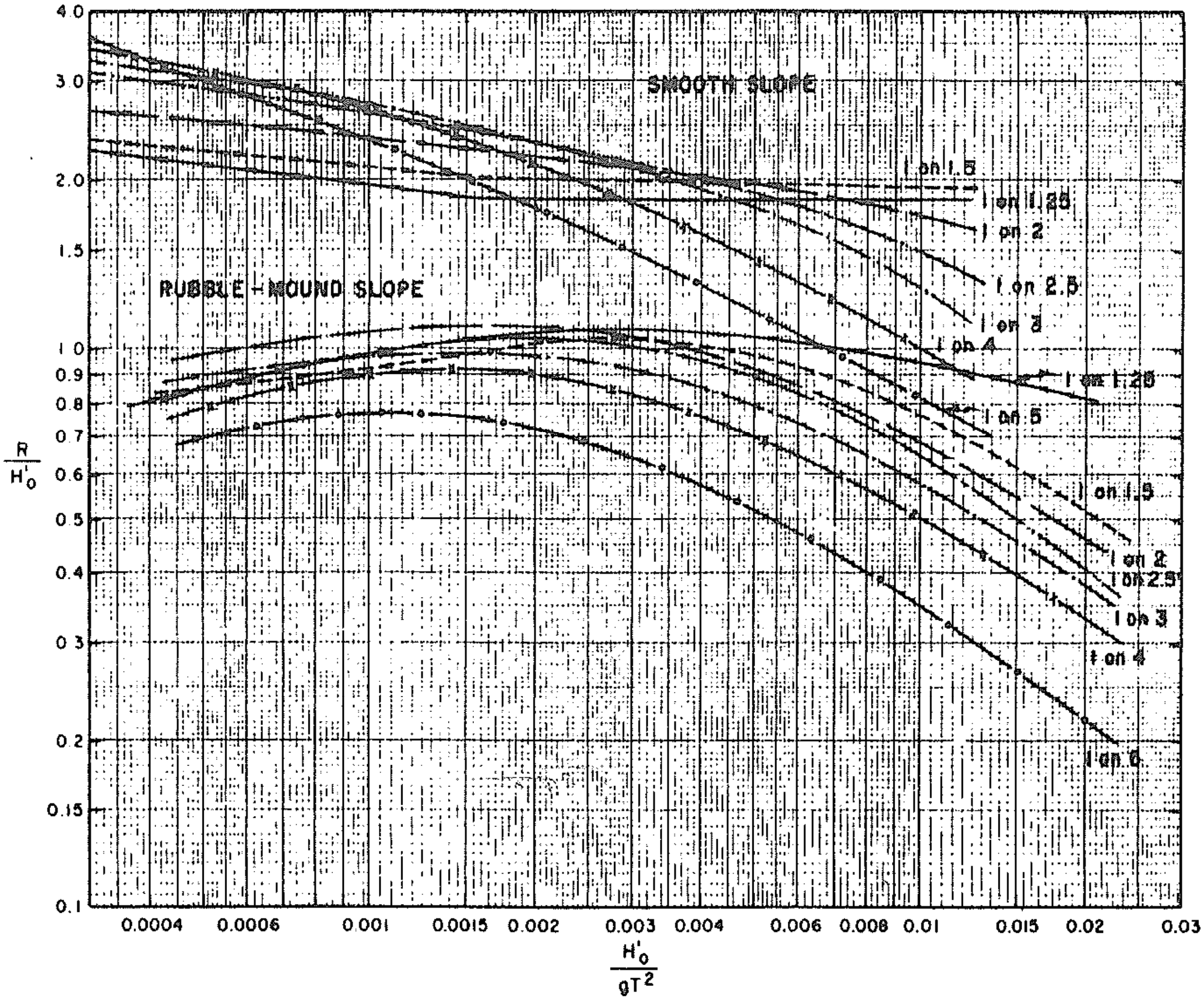


Figure 19. - Comparison of wave runup on smooth slopes with runup on permeable rubble slopes. (Data for $d_s / h'_o > 3.0$)

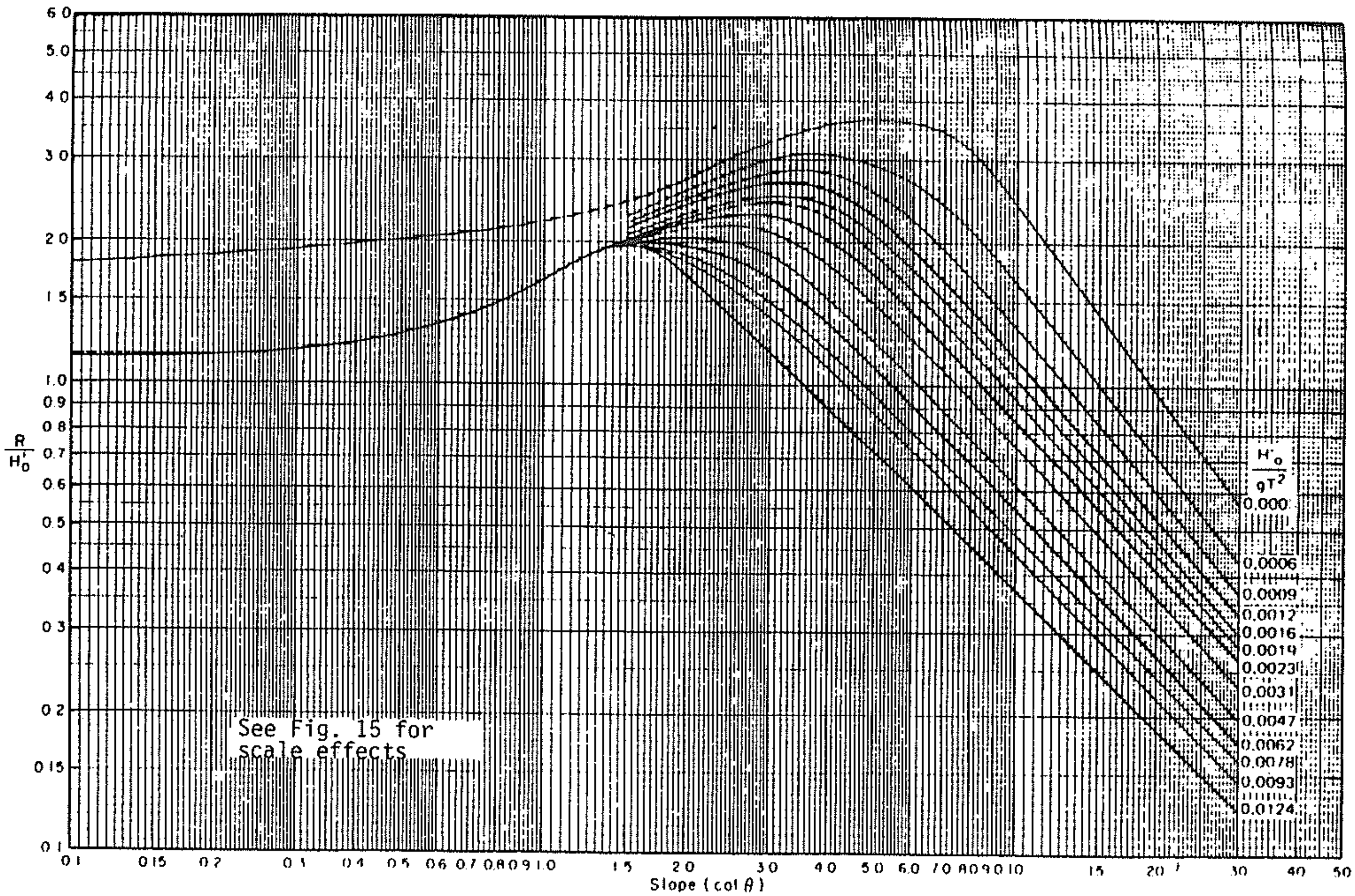


Figure 20. - Wave runup on smooth, impermeable slopes when $d_s / h'_o \geq 3.0$.

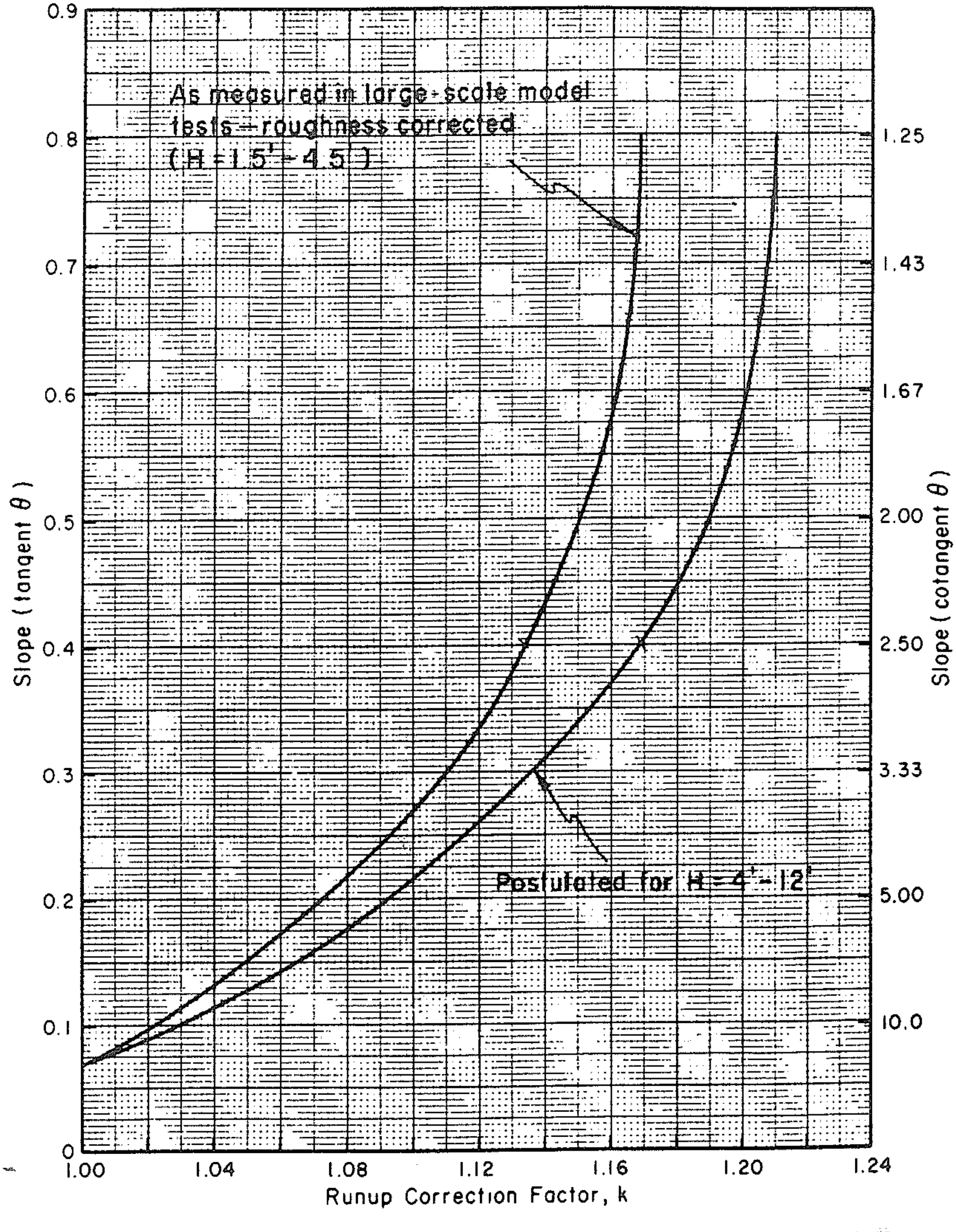


Figure 21. - Runup correction for scale effects.

APPENDIX I. - LIST OF TERMS

Term	
DURELV	The duration that could be expected each year of the reservoir water surface being at or above the given elevation (h).
DURINT	The time that the reservoir would be within a certain interval any year (h).
F	The reservoir fetch is an average horizontal distance over which wind acts to generate waves at a particular point (mi).
g	The acceleration due to gravity (79036.36364 mi/h^2, or 9.806650 m/s^2).
H_s	The significant wave height, the average of the highest one-third of the waves of a given group or spectrum. "Wave height" is the vertical distance between a wave crest and the preceding trough (ft).
IDF	The inflow design flood. Usually the PMF.
L	The deep water wave length is the horizontal distance between similar points on two successive waves (ft).
MWS	Maximum water surface elevation (ft).
P_{D_A}	An annual probability for design. An annual probability of the combined events, wind and water being exceeded.
PMF	The probable maximum flood event.
P_{PMF_A}	The annual probability of the probable maximum flood being exceeded.
P_{R_H}	The hourly probability of the reservoir water surface at or above a given elevation.
$P_{[RR(interval)]_H}$	The hourly discretized probability of the reservoir water being between two elevations as defined by the interval.
$P_{[RR(interval)+W]_H}$	The hourly conditional interval design exceedance probability. The hourly probability of the water surface being at or above a certain elevation due to a wind event occurring while the reservoir is between two elevations as defined by the interval.

P_{W_n}	The probability of the wind event being exceeded any hour.
RUNUP	The movement of water up a structure or beach on the breaking of a wave. The amount of runup is the vertical height that the water reaches above stillwater level (ft).
SETUP	The wind setup is the vertical rise in the stillwater level on the leeward side of a body of water due to wind stresses on the surface of the water (ft). (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 (ft).
Target	An arbitrary elevation for the dam crest for which P_{D_A} is computed (ft).
T_{D_A}	The annual return period for design. The annual return period of the combined events, wind and water, occurring (year).
t_{MIN}	The minimum time required to build up (fully develop) the maximum waves for a given wind velocity and reservoir fetch (h).
V	The wind velocity over land (m/s).
VMPH	The wind velocity over water (mi/h).
VMS	The wind velocity over water (m/s).
Θ	The angle of the upstream face of the embankment dam with the horizon (°).

APPENDIX II. - COMPUTATIONS FOR EMBANKMENT DAM FREEBOARD ANALYSES

A. 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° intervals and extending these radials until they first intersect the shoreline again on the opposite side of the reservoir [9] (see fig. 2). The length of each radial is measured and arithmetically averaged. While 3° 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 24° 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 figure 8 to adjust the wave height 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. It is possible to show that some of the physical arguments on which this is based do not appear valid. Hence, no definition of effective fetch has been formulated in this method. It is possible that fetch width may become important if the fetch is actually very narrow. 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 memorandum.**

For cases where there is an abrupt change in the shoreline geometry or unusual circumstances may cause one to consider that this simple method for calculation of fetch is ambiguous, the U.S. Army Corps of Engineers has documented variations of the methods in section C of reference 9, which can be used.

B. Simultaneous Occurrence of All Freeboard Components

The possibility that some combinations of the components of freeboard will occur simultaneously is extremely low. Maximization of all components and adding them together to determine total freeboard requirements is unreasonable. Only those components which can reasonably occur simultaneously for a particular water surface elevation should be combined. The design crest elevation of the dam should be established to accommodate all combinations of water surface and wind occurrences with other freeboard components that are deemed reasonable. The design crest elevation excludes camber, road surfacing, and associated crown.

It is highly unlikely that maximum winds will occur when the reservoir water surface is at its maximum elevation resulting from routing the PMF. Computations of wind-generated wave height and wind setup for intermediate freeboard should incorporate the probability of combined occurrences of reservoir elevation and duration and wind velocity and duration. Consideration should be given to the shape of the reservoir elevation versus time curves during flood events. Although the maximum reservoir elevation associated with many flood events may be expected to last only a few hours, a reservoir elevation close to but lower than the maximum may have a much longer duration. The freeboard analyst must examine many combinations of intermediate reservoir elevations and wind events generating wave runup and setup such that a minimum crest elevation may be determined that would protect the dam from all possible wind and flood events.

C. Methods of Freeboard Analysis

1. Design of small dams. - The 1987 edition of Design of Small Dams [10] contains freeboard guidelines for low-hazard, small embankment dams. Table 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 windspeed of 100 mi/h (160 km/h) for normal freeboard and 50 mi/h (80 km/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." 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 1. - Approximation of minimum and normal freeboard [10]

Longest fetch, mi (km)	Normal freeboard (added to the normal water surface) ft (m)	Minimum freeboard (added to the maximum water surface) ft (m)
< 1 (1.6)	4 (1.2)	3 (0.9)
1 (1.6)	5 (1.5)	4 (1.2)
2.5 (4.0)	6 (1.8)	5 (1.5)
5 (8.0)	8 (2.4)	6 (1.8)
10 (16.1)	10 (3.0)	7 (2.1)

Note: These values were based on a wind velocity of 100 mi/h (160 km/h) blowing over the normal water surface and 50 mi/h (80 km/h) blowing over the MWS. 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.

2. U.S. Army Corps of Engineers. - The U.S. Army Corps of Engineers 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], and its revision, ETL 1110-2-305, dated February 16, 1984 [9]. The Shore Protection Manual, fourth edition [13], published in 1984, contains many of the relationships used here for fetch calculations and wave derivations.

3. Probabilistic method. - As indicated in subsection I.B.2.b., the probabilistic method for computing freeboard requirements must be used for new dams whenever the criteria for the simplified approach are not satisfied.

The probabilistic freeboard analysis involves two basic steps. The first step is, in effect, the derivation of the cumulative probability distribution of a sum of two independent random variables, generically $Z = X + Y$, where in this case X is the water surface elevation corresponding to a flood level (expressed as a fraction of the PMF) and Y is the wind-induced incremental height (due to wave runup and wind setup). The type of calculation required, known as convolution, generally proceeds as follows: the range of one of the random variables (in this case, the water surface elevation) is divided into non-overlapping increments, and the probability of not exceeding a given value of the sum, denoted by z (representing, in this case, a "target" crest elevation, is found by multiplying nonexceedance probabilities, one for each increment (of the water surface elevation). The probability of exceedance of a given crest elevation, denoted here as $p = \text{Prob}\{Z \geq z\}$, is expressed with reference to a 1-hour interval, randomly chosen within a year.

The second step is to convert the hourly exceedance probability of a given design freeboard level z into an annual probability of exceedance. A given freeboard level is not exceeded in a 1-year period if it is not exceeded in any one of the non-overlapping 1-hour segments that make up 1 year.

The calculated "hourly risks," $p = \text{Prob}\{Z \geq z\}$, strongly depend on the assumed probability distributions of the input random variables "X" and "Y." In particular, the range of water surface elevations has a prescribed minimum and maximum value, the latter corresponding to the PMF whose exceedance probability may be arbitrarily set at 10^{-4} per year for relative analysis purposes. In light of these assumptions, it is desirable that a sensitivity analysis be done to determine the relative sizes of contributions to the "hourly risks" p (or "annual risks "P") from different flood levels, i.e., floods corresponding to different fractions of the PMF. It also makes sense to consider the impact on the overtopping risk due to floods that are "multiples of the PMF," associated with mean annual occurrence rates below 10^{-4}.

The remainder of this section presents a step-by-step description of the probabilistic method. It should be noted that the use of this method is greatly facilitated by the use of computers with programs already developed by the Bureau of Reclamation at the Denver Office (D-3620).

a. Analysis of existing wind data. -

(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.

(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_n}, the probability of the wind exceeding a given velocity for a specific number of hours, is derived.

(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 which are closer to the reservoir, those which have topography and surrounding vegetation similar to that of the reservoir, have 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, p_{W_n}, 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.

(4) Wind event curves. - The probability of wind exceeding a given velocity (p_{W_n}) 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_n}) during any wind event.

(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 overland to overwater velocities. The wind velocities for each wind event curve are measured overland and must be adjusted to overwater velocities for use in calculating wave heights, wave runup, and wind setup. The following table demonstrates the relationship between winds blowing over land as compared to wind blowing over water at the same elevation.

Table 2. - Relationship between overland and overwater wind velocities

	Wind velocity (m/s)									
Overland	2.0	4.0	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0
Overwater ($F < 16$ km)	2.4	4.8	7.2	9.6	11.2	12.7	14.3	15.6	17.1	18.0
Overwater ($F \geq 16$ km)	3.3	5.6	7.5	9.6	11.2	12.7	14.3	15.6	17.1	18.0

(6) Minimum wind duration to reach maximum wave heights. - Both wind duration and fetch distance can limit the height of waves caused by a given wind velocity. Waves are assumed to grow continuously under the action of the wind as they move along the fetch. Given the minimum fetch distance needed, the waves will reach a maximum height that can be sustained by the wind velocity. There will be no further increase in wave height regardless of how long the wind blows or how much the fetch exceeds the minimum needed. Conversely, given a limited fetch distance, the maximum wave height for a particular velocity will not be reached because the waves will collide with the shoreline or dam before reaching maximum height.

The duration needed for a given wind velocity to generate the highest waves is designated the minimum duration. The value of fetch is used with figure 7 to obtain minimum durations for the wind velocities (adjusted to overwater velocities) corresponding to the wind event curves. The minimum durations can also be computed as:

$$t_{min} = 1.912\ (F^{0.66} / V^{0.41}) \qquad \text{[Equation 1]}^{4}$$

where:

t_{min} = The minimum duration required to build up the maximum waves (hours).
F = Fetch (miles).
V = Wind velocity (mi/h).

[4]Equivalent metric unit equations are given at the end of this appendix.

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 wind event curve.

(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_u}) versus wind velocity (overwater) should be drawn on semilogarithmic paper. Values of P_{W_u} and their respective overland (converted to overwater) velocities corresponding to the minimum duration for each velocity should be used.

b. Analysis of flood data. - Reservoir flood storage between the top of active conservation or joint-use capacity and the MWS is divided into intervals. Discrete probabilities can be computed for each interval based on flood data. When the wind events are added to these reservoir events, wave runup, and wind setup determine the target crest elevations.

(1) Reservoir events. - All intermediate reservoir events between the top of active conservation or joint-use capacity and the MWS are used in this probabilistic approach. The PMF, 4,000-, 1,000-, 400-, and 100-year flood events are routed so that interpolations can be made to obtain the durations of any reservoir elevation.

(2) Duration of reservoir water surface. - Twenty or so intermediate reservoir water surface elevations are selected between the top of active conservation or joint-use capacity and the MWS. Durations that the water is at or above each elevation for each flood are determined from the elevation versus time information of the flood routings. These durations are plotted versus their annual probability (1/the flood mean return period) on semilogarithmic paper. Lines are drawn through points representing equal reservoir elevations. These lines are divided up into 10 or so increments and the probability times the average duration of each increment is summed to obtain DURELV, the amount of time (in hours) that the reservoir can be expected at or above each elevation each year.

(3) Probability of reservoir water surface intervals. - The hourly probability of the reservoir exceeding the given elevation (P_{R_n}) is calculated by:

$$P_{R_n} = \frac{DURELV}{(24)\ x\ (365)} \qquad \text{[Equation 2]}$$

where:

P_{R_n}	=	The hourly probability of exceeding the given reservoir elevation.
DURELV	=	The total duration that the reservoir would be expected at or above the given elevation in any year (h).

The hourly probability of being within a reservoir interval $P_{(RR(interval))_n}$, is the difference between the P_{R_n} of the two bounding reservoir elevations or:

$$P_{(RR(interval))_n} = \frac{DURINT}{(24)\ x\ (365)} \qquad \text{[Equation 3]}$$

where:

$P_{(RR(interval))_n}$	=	The hourly probability of the reservoir being within a certain interval of two elevations.
DURINT	=	The time that the reservoir would be expected to be within the certain interval any year. The difference between the DURELV of each reservoir elevation bounding the interval (h).

c. Wind effects on water. -

(1) Wave height. - Wind-generated waves in large bodies of water are not uniform in height but consist of spectra of waves with various heights [11, 12]. A well-defined relationship exists between the significant wave height (H_s) and the heights of the other waves in the spectrum. The relationship is shown in table 3. From this tabulation, it can be seen that H_s represents the average height of the highest one-third of the waves in a given spectrum. Likewise, the average wave height of the highest 10 percent of the waves in a given spectrum is 1.27 H_s, and the average wave height of the highest 1 percent of the waves in a given spectrum would be approximately 1.67 H_s.

Table 3. - Common wave height relationships

Percent of total number 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)	(3)	(4)
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	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

The maximum wave height ratio to be used to compute wave runup should be selected 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 or will not slough or soften excessively, or when public traffic will not be interrupted, a wave height equal to the average height of the highest 10 percent of the waves (1.27 x height of significant wave) [9] should be used to compute runup. 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.

The height of significant wave due to each wind event is determined from figure 7 or calculated from the relationship:

$$H_s = 0.0177 \ (V)^{1.23} \ (F)^{0.5} \qquad \text{[Equation 4]}$$

where:

H_s = Height of significant wave, in feet
V = Wind velocity, in miles per hour
F = Fetch, in miles

Wave heights for waves computed for fetches that are not normal to the dam axis should be reduced according to a factor derived from figure 8. The angular spread is the angle between the central radial of the nine radials used to compute the fetch and a line normal to the dam axis. The reduction factor is multiplied to the significant wave height to obtain a reduced significant wave height.

(2) Wave length and wave period. - The deep water wave length in feet can be computed by the relationship:

$$L = 5.12 \ T^2 \qquad \text{[Equation 5]}$$

in which T = the wave period in seconds obtained from figure 7 or the following equation:

$$T = 0.559 \ (0.589 (v)^{1.23} \ (F))^{0.33} \qquad \text{[Equation 6]}$$

It may be assumed that the wave period T is the same for all wave heights in a given wave spectrum.

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 should be used. Wave height, wave period, and minimum duration for shallow water waves can be obtained from figures 9-18 or can be computed from similar relationships given in reference 13, chapter 7.

(3) Wave runup. - If a deepwater 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 stillwater elevation. Runup, from a wave, on an even embankment with a riprap surface is given by:

$$R = \frac{H}{0.4+\left(\frac{H}{L}\right)^{0.5}\cot\theta} \qquad \text{[Equation 7]}$$

where:

R = The vertical component of wave runup in feet
H = The wave height in feet
L = The wave length in feet
θ = The angle of the dam face from horizontal

This equation should be used only for dam slopes of 5:1 (horizontal:vertical) or steeper. The wave height used to compute runup should be selected on the basis of permissible overtopping by runup. The selection of the P_{W_u} value to use in determining the wind velocity for the calculation of wave height has been described previously in this ACER Technical Memorandum.

For embankment dams with soil-cement, rounded cobbles and boulders for riprap, or other protective surfaces not as rough or irregular as dumped angular riprap, the runup computed by the above equation should be multiplied by a factor of up to 1.5 (for the smoothest embankment surfaces, such as soil cement), depending on the relative smoothness of the surface.

Equation 7 should not be used for computing runup for rockfill dams. Rockfill acts more like a rubble mound structure and has a different effect on energy dissipation than riprap placed on an impervious embankment. Runup for rockfill dams may be determined from figure 19. These data were taken from figure 7-20 of reference 13, volume II.

For smooth, impermeable slopes of concrete and other smooth surface dams with water depth at the dam toe (d_s) greater than three times the wave height (H), the relationship between wave runup and wave height can be determined from figure 20. These data were taken from figure 7-12 of reference 13.

Results predicted by figure 20 are probably less than the runup on prototype structures because of scale effects due to the inability to scale roughness effects into small-scale laboratory tests. Runup values should be adjusted for scale effects by using a factor obtained from figure 21.

(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 leeward end of the fetch. This effect is termed "wind-tide" or "wind setup."

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

$$S = \frac{V^2 F}{1400\ D} \qquad \text{[Equation 8]}$$

where:

V = The design wind velocity over water in miles per hour
F = Wind fetch in miles
D = Average water depth in 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 effective fetch.

d. Probability relationships. -

(1) General. - For each interval of intermediate reservoir water surface elevations due to floods, numerous $P_{(RR(interval)+W)_H}$ are calculated. This is a conditional exceedance probability where the hourly wind probability, P_{W_H}, is combined with the hourly probability of the reservoir water surface interval, $P_{(RR(interval))_H}$:

$$P_{(RR(interval)+W)_H} = P_{(RR(interval))_H} \times P_{W_H} \qquad \text{[Equation 9]}$$

where:

$P_{(RR(interval))_H}$ = The hourly discrete probability of the reservoir water surface being within a certain interval (see equation 5).

P_{W_H} = The hourly exceedance probability of the wind event occurring.

The annual probability for design, P_{D_A}, is the probability that a given crest elevation will be equaled or exceeded by a combination of flood and wind events in any 1 year. It is the sum of the probabilities of all possible wind and flood events that could raise the water to the target crest elevation.

It is derived by the following two equations. Equation 10 is the hourly probability for design,

where:

$$P_{D_H} = 1 - (1-P_{(RR(interval)+W)_H})\,(1-P_{(RR(interval)+W)_H}) \ldots \qquad \text{[Equation 10]}$$

P_{D_H}	=	The hourly probability for design. The hourly probability that water will rise up to or over the target crest elevation.
$P_{(RR(interval)+W)_H}$	=	The hourly exceedance probability of the reservoir water surface being within a given interval simultaneously occurring with a wind event that creates setup and runup to bring water up to a target crest elevation.

The hourly probability for design is converted to an annual probability by equation 11:

$$P_{D_A} = 1 - (1-P_{D_H})^{n} \qquad \text{[Equation 11]}$$

where:

P_{D_A}	=	The annual probability for design. The annual probability that water will rise up to or over the target crest elevation.
n	=	The number of independent wind events, measured in hours, per year. Ignoring the statistical dependence, n = 8760, the number of hours in a year, yields a conservative estimate of the annual probability for design for a given target crest elevation.

The proper value of "n" (less than 8760) is difficult to determine. Every hour of wind at a site is not always independent of the previous or next hour of wind. One might try to estimate a typical duration (in hours) of either a wind or flood event and divide 8760 by this number to obtain a "n." But, it is useful and sufficient to take n = 8760 and state that it will yield too high a value for the annual probability for design of a target crest elevation resulting in the selection of a conservative, higher design crest elevation for a given probability.

(2) Target crest elevations. - The set of target elevations for the dam crest (each higher than the maximum water surface) is chosen somewhat arbitrarily and the probability of reaching each of these elevations from all possible reservoir intervals is computed. These elevations are reached by runup and setup generated by wind. Wind events of various magnitude produce various runup and setup which are added to the MWS to obtain the set of target crest elevations. Any of these target crest elevations can be reached by wind-generated waves when the reservoir water surface is within any

interval. The lower the interval, the higher the wind velocity has to be. The probability of reaching a target crest elevation from one interval due to a particular wind event is computed in equation 9. The probabilities are adjusted as one considers the wind event necessary to reach the same target crest elevation from another reservoir interval. For one target crest elevation, the above calculation is performed many times to obtain the probability of reaching the target elevation from all reservoir intervals within flood storage.

For each reservoir water surface interval considered, a tabulation of $P_{(RR(interval)+W)_n}$ required to achieve each target probable crest elevation is made. After all of the computations have been made for every reservoir water surface elevation interval, the P_{D_A} is calculated (see equations 10 and 11) for each target crest elevation. A plot is made of P_{D_A} versus the crest elevation on semilogarithmic paper. The crest elevation satisfying normal and intermediate freeboard requirements as well as one condition of minimum freeboard is defined by the choice of P_{D_A}. A recommended value of P_{D_A} is 0.0001.

e. The interrelationship between minimum, normal, and intermediate freeboard in the probabilistic method. - 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 PMF. Thus, the probabilistic approach should not apply any velocity winds less than typical to the uppermost reservoir interval to derive the lowest possible target crest elevation and a minimum freeboard above the MWS. The typical wind which is applied to the MWS applies the lowest wind velocity considered in the computations. Ordinarily, the lowest wind velocity considered is that which has an exceedance probability equal to 10 percent (P_{W_n} = 0.1). A minimum freeboard is determined by adding the runup and setup caused by this wind event to the MWS.

To create waves that would reach to the top of the dam from reservoir elevations far below the crest (say, nearer the normal water surface than the maximum water surface), very high wind velocities are often needed. Contributions to freeboard from extreme wind events are expected to be insignificant, however, because their probabilities are so low. It is more important to assign realistic maximum winds to reservoir intervals down near the normal water surface. It is suggested that this be in the range of 60 to 100 mi/h.

Intermediate freeboard is that distance between the dam crest and any reservoir interval within flood storage. Minimum and normal freeboard are the special cases of intermediate freeboard where the reservoir interval considered is at the top or near the bottom of flood storage. More appropriately, intermediate freeboard is the calculation of the runup and setup necessary to reach a particular one of the target dam crest elevations from any reservoir interval between maximum water surface and top of active conservation or joint-use capacity.

f. Metric equivalents of the presented equations. -

$$t_{min} = \frac{F^{0.666}}{V^{0.41}} \qquad \text{[Equation 1]}$$

where:

t_{min} = The minimum duration required to build up maximum waves (h)
F = Fetch (km)
V = Wind velocity (m/s)

$$H_s = 0.011452\ V^{1.23}\ F^{0.5} \qquad \text{[Equation 4]}$$

where:

H_s = Height of the significant wave (m)
V = Wind velocity (m/L_s)
F = Fetch (km)

$$L = 1.56\ T^2 \qquad \text{[Equation 5]}$$

where:

L = The deep water wave length (m)
T = Wave period (s)

$$T = 0.556\ V^{0.41}\ F^{0.333} \qquad \text{[Equation 6]}$$

where:

T = The wave period (s)
V = Wind velocity (m/s)
F = Fetch (km)

$$R = \frac{H}{0.4+(H/L)^{0.5}\cot\theta} \qquad \text{[Equation 7]}$$

where:

R = Wave runup (m)
H = Wave height (m)
L = Wave length (m)
θ = angle of the dam face from the horizontal (°)

$$S = \frac{V^2 F}{4850\ (D)} \qquad \text{[Equation 8]}$$

where:

S = Wind setup (m)
F = Fetch (km)
V = Wind velocity (m/s)
D = Average water depth (m)

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