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Determining Design Basis Flooding at Power Reactor Sites

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Foreword

(This Foreword is not a part of the American National Standard for Determining Design Basis Flooding at Power Reactor Sites, ANSI/ANS-2.8-1981.)

The purpose of this document is to specify standards for determining design basis flooding at power reactor sites. This standard was prepared by Working Group ANS-2.8 of ANS-2 Subcommittee, Site Evaluation, of the American Nuclear Society Standards Committee. The directive to the working group is as follows: "Guidelines are to be developed to establish design basis flooding at power reactor sites as a result of river, stream, or seismically-induced dam failure; surge, seiche, or wave action flooding, or any combination of these events. Methodology will be described for evaluating the worst site-related flood at a power reactor site caused by either a probable maximum flood on streams and rivers and any dam failures resulting therefrom: a seismically-induced dam failure flood; a probable maximum surge and seiche flood; and any attendant wind-generated wave activity associated with these events, or caused by a reasonable combination of less severe events."

This standard covers material that meets the requirements of Section 2.4, Hydrologic Engineering, of Regulatory Guide 1.70, Revision 3, November 1978, "Standard Format and Content of Safely Analysis Reports for Nuclear Power Plants," issued by the Regulatory Staff of the Nuclear Regulatory Commission (NRC). This standard does not cover requirements of said Regulatory Guide on the following Standard Format 2.4 sections:

(1) Probable Maximum Tsunami Flooding - (Addressed by proposed American National Standard Bases for Estimating Tsunami Levels and Forces at Power Reactor Sites, ANS-2.4.)

(2) Low Water Considerations - (Addressed by American National Standard Evaluation of Surface-Water Supplies for Nuclear Power Sites, ANSI/ANS-2.13-1979.)

(3) Dispersion, Dilution, and Travel Times of Accidental Releases of Liquid Effluents in Surface Waters - (Addressed by American National Standard Evaluation of Radionuclide Transport in Ground Water for Nuclear Power Sites, ANSI/ANS-2.17-1980.)

(4) Ground Water - (Addressed by American National Standard Evaluation of Ground Water Supply for Nuclear Power Sites, ANSI/ANS-2.9-1980.)

(5) Technical Specifications and Emergency Operation Requirements.

Before preparing the Safety Analysis Report (SAR) Section 2.4, Hydrologic Engineering, for the licensing of nuclear power plants, the applicant should be aware of hydrologic work which has been done by others in the area of interest. Almost invariably, much work can be saved by utilizing all or parts of studies by local, state, and federal agencies. Such information as dimensioned or dimensionless unit hydrographs, loss rates, lag times, historical floods, geologic and groundwater data, etc., may be obtained from such sources. Sometimes the probable maximum flood has already been derived at the site or at a point near enough to be transposed.

The prime source of such information is the Corps of Engineers. Other federal agencies which may have useful data are the Bureau of Reclamation, Soil Conservation Service, Weather Service, Geological Survey, Tennessee Valley Authority, Environmental Protection Agency, Federal Energy Regulatory Commission (formerly

Federal Power Commission), and the Nuclear Regulatory Commission. Most states have one or more agencies which are concerned with various aspects of water resources. Power companies, particularly those with hydropower capacity, are another source, as are municipal or regional water-supply organizations. Safety Analysis Reports for other nuclear plants in the area may also provide useful information. It is also profitable to discuss the specific site in detail with the hydrology staff of the NRC prior to starting preparation of Section 2.4. In such discussions, the scope of work can often be reduced and methodologies and procedures can be agreed upon, which will save many man-hours and dollars, both for the applicant and for the NRC staff.

This standard was developed by Working Group ANS-2.8 of the American Nuclear Society. In one of the several meetings during the development of the first document, Mr. John Riedel, Chief of the Hydrometeorology Branch of the National Weather Service and Mr. Dwight Nunn, consultant to the NRC, were present to answer questions raised by ANS-2.8 members on hydrometeorology.

The first issue of the standard was approved by the American National Standards Institute Inc. on November 1, 1976 and was published by the American Nuclear Society as American National Standard, Standards for Determining Design Basis Flooding at Power Reactor Sites, N170-1976 (ANS 2.8).

The ANS-2.8 members met on January 30, 31 and February 1, 1979 to review, reaffirm, revise or withdraw N170. This step is required by the American National Standards Institute no later than five years from the date of publication.

This standard was revised by Working Group ANS-2.8 of the American Nuclear Society which had the participation of the following members during the period it prepared and approved the revision:

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The American Nuclear Society's Nuclear Power Plant Standards Committee (NUPPSCO) had the following membership at the time of its approval of this standard:

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Determining Design Basis Flooding at Power Reactor Sites

1. Introduction and Scope

1.1 Scope. This document presents a standard to establish design basis flooding for nuclear safety-related features at power reactor sites. Methodology is described to evaluate the flood having virtually no risk of exceedance that may be caused by: precipitation and snowmelt and any resulting dam failures; seismically induced dam failures; surge or seiche and attendant wind-generated wave activity; or a reasonable combination of these events.

1.2 Discussion. This standard covers that material necessary to develop the design basis flooding for use in evaluation of the adequacy of nuclear power plant site. Water related effects such as water levels, waves, wave forces, ice, erosion, and sedimentation are included to assist in the design of safety related facilities. Where information presentation requirements are stated in this standard, such as "provide," "tabulate," or "describe," such information shall be provided as a part of the documentation for the design basis flood estimate.

1.2.1 Exclusions. This standard does not cover:

(1) Probable Maximum Tsunami Flooding

(2) Low Water Considerations

(3) Dispersion, Dilution, and Travel Times of Accidental Releases of Liquid Effluents

(4) Ground Water

(5) Technical Specification and Emergency Operation Requirements

(6) Channel Diversions

(7) Flooding Protection Requirements partially addressed

(8) Flooding From Pipe Rupture or On-Site Tank Failures.

1.2.2 Stochastic Methods. Stochastic methods of generating data on hydrologic events are not addressed in this standard because, with available records, it is believed that design basis floods of sufficient reliability and low annual probability cannot be predicted using these methods.

2. Definitions

probable maximum precipitation (PMP). Probable maximum precipitation is the estimated depth for a given duration, drainage area, and time of year for which there is virtually no risk of exceedance. The probable maximum precipitation for a given duration and drainage area approximates the maximum which is physically possible within the limits of contemporary hydrometeorological knowledge and techniques.

moving squall line. A moving squall line is a line or narrow band of active thunderstorms having a pressure jump with the cold front providing the initial piston-like impetus, and a mature instability line which is located in the warm sector of a wave cyclone about 50 to 200 miles in advance of the cold front usually oriented roughly parallel to the cold front and moving in about the same direction and speed as the cold front.

probable maximum flood (PMF). The probable maximum flood is the hypothetical flood (peak discharge, volume, and hydrograph shape) that is considered to be the most severe reasonably possible, based on comprehensive hydrometeorological application of probable maximum precipitation and other hydrologic factors favorable for maximum flood runoff such as sequential storms and snowmelt.

probable maximum hurricane (PMH). A probable maximum hurricane is a hypothetical hurricane having that combination of characteristics which will make it the most severe that can reasonably occur in the particular region involved. The hurricane approaches the point under study along a critical path and at an optimum rate of movement which results in most adverse flooding.

probable maximum windstorm (PMWS). The probable maximum windstorm is a hypothetical extratropical cyclone that might result from the most severe combination of meteorological storm parameters that is considered reasonably possible in the region involved. The windstorm approaches the point under study along a critical path and at an optimum rate of movement which will result in most adverse flooding.

probable maximum gradient wind. A probable maximum gradient wind is defined as a probable gradient wind of a designated duration above the surface friction layer, of which there is virtually no risk of being exceeded. The event may be considered to have a probability of occurrence comparable to that of a probable maximum precipitation.

3. Hydrologic Description

3.1 Site and Facilities. The physical characteristics of the site and facilities shall be described in sufficient detail to allow independent analysis of pre- and post-construction drainage patterns.

3.1.1 Area. The land area of the site shall be stated and the site boundaries described.

3.1.2 Location. Location of site with respect to nearby streams, lakes, reservoirs, or ocean in terms of horizontal distances and directions shall be stated. Location may also be stated in terms of miles and directions from the nearest town, city, or U.S. Geological Survey (USGS) control base and meridian. If available, a tie line shall be established between a permanent section corner or base and meridian point stating distance, bearing, and applicable type of projection of plane coordinate system (such as Transverse Mercator or Lambert Conformal Projections) to a well defined corner of the site boundary. The elevation datum and adjustment to be used and relation to other, locally used data shall be defined.

3.1.3 Safety-Related Structures, Systems, and Components.[1] A list of pertinent elevations of safety-related structures shall be provided for comparison with design basis flood levels. It shall be referenced to maps and drawings of such facilities. See Table 1 below. Also see American National Standard Design Basis for Protection of Light Water Nuclear Power Plants Against Effects of Postulated Pipe Rupture. ANSI/ANS-58.2-1980. [1] [2]

3.1.4 Access. Any flood-related problems of access shall be described: to the site from offsite; to safety-related features within the site area.

3.1.5 Changes in Topography. Changes in topography proposed for the plant affecting natural drainage shall be described, indicating purpose of each change. Describe planned site drainage, giving direction and outlet of runoff. Include drainage from roofs of buildings.

3.1.6 Maps. A topographic map of the site, with contour interval appropriate for the topography, shall be provided. The map scale, shown graphically, shall be between 1″ = 100′ and 1″ = 500′. The map shall be capable of reduction to a readable 11″ x 17″ size for the Safety Analysis Report. The map shall show outlines of all major structures. Proposed

Table 1
Example for 3.1.3

Structure	Access	Number of Accesses	Elevation Above Mean Sea Level	Ref. Fig. No.
Intake Pumping Structure	(1) Stairwell entrance	1	705.0	—
	(2) Access hatches	6	705.0	—
Auxiliary and Control Building	(1) Railroad access opening	1	706.0	—
	(2) Doors to turbine building	2	706.0	—
	(3) Doors to turbine building	2	712.0	—

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[1] As described in the U.S. NRC Regulatory Guide 1.59, "Design Basis Flooding for Nuclear Power Plants," August 1977.

[2] Number in brackets refers to corresponding number in Section 12, References.

changes of topography should be shown with contour lines contrasting well with the natural contour lines. If several maps are required, provide an index map.

3.2 Hydrosphere. Water bodies in the area which may influence the site shall be described. These include streams, lakes, shore regions, and ground water. If plant construction will change any drainage areas, pre- and post-construction drainage areas shall be stated. Drainage areas of pertinent watercourses shall be stated in square miles. Mean annual discharge, in the case of a stream, shall be stated if known; otherwise, it should be estimated. Lengths and slopes of pertinent ungaged streams shall be stated. For lakes, the area, average elevation, normal range of elevation, and representative depths shall be provided. Normal tidal range shall be provided for ocean or estuary sites. Other sections of the standard describe determination of extreme high and low water levels. If storm surges and wind-wave calculations are required for lakes or oceans, the required bathymetric maps shall be included in this section, or reference shall be made to the appropriate section where they appear.

3.2.1 Dams and reservoirs. The following information shall be shown for all upstream and downstream dams that may have a significant influence on floods or the plant's safety-related water supply. The information should be in tabular form:

(1) Name of dam and reservoir
(2) Owner and designer, if known
(3) Location in relation to power reactor site in river miles
(4) Drainage area above dam in square miles
(5) Date construction completed
(6) Streambed elevation
(7) Top of dam elevation
(8) Range, volumetric capacity, and purpose of operating levels
(9) Types, capacities, and elevations of spillway and outlet works
(10) Type of dam
(11) Seismic and hydrologic design criteria, if available.

NOTE: Plan, elevation, typical sections, elevation-storage-area data, spillway and outlet rating curves, operating rules, water commitments, and other information may be required under other sections of the standards to support conclusions on flood potential and water supply adequacy.

3.2.2 Watershed Index Map. Where practical, an index map of the topographic quadrangles covering the watershed, shall be provided.

3.2.3 Future Changes. When significant changes are recognized in the hydrosphere, their influence on design basis flooding shall be determined.

3.2.4 Monitoring. Hydrologic, meteorologic, or oceanographic gages installed in connection with plant design or operation should be monitored during the life of the plant for use also in verifying the design basis and other floods.

3.3 Available Retrievable Data

3.3.1 Hydrologic Data. Some data on stream stages and flows, discharge measurements, reservoir stages and contents, sediment concentrations, water quality, station descriptions and watershed area and some statistical analyses are available in digital format from the U.S. Geological Survey in Reston, Virginia and district offices.

3.3.2 Meteorologic Data. Some data on precipitation (including snow depth), temperature, wind, dew point, solar radiation, cloud covers, hours of sunshine, and evaporation, in digital format, and synoptic maps are available from the National Climatic Center in Asheville, North Carolina.

3.3.3 Oceanographic Data. Some data on waves, swells, tides, and currents are available in digital format from NOAA, Silver Spring, Maryland.

3.3.4 Other Sources. Hydrologic, meteorologic, and oceanographic data may be available also from public and private agencies.

4. Plant Safety From Floods

4.1 Flood History. Historical floods in the region of the site shall be identified and used in the analyses where appropriate.

4.1.1 Flood Peaks. Major historical flood events shall be tabulated showing the date of occurrence, observation stations, instantaneous peak discharges, and crest elevations. Provide a brief description of the flood-causing events. References shall be provided.

4.1.1.1 Regional Record. A regional analysis, such as envelope curves, may be used in the absence of flood records near the site.

4.1.1.2 Sources. Streamflow and flood peaks are collected and published by the U.S. Geological Survey in Water-Supply Papers, Hydrologic Investigation Atlases, or Circulars. Examples are:

(1) Major Floods and Flood Summaries-Descriptions of severe floods that have occurred in the United States and summaries of notable floods are published as Water-Supply Papers and Professional Papers.

(2) Flood Maps - Maps of certain localities showing the area inundated in past floods are published as Hydrologic Investigations Atlases. Atlases contain information on local flood heights, frequencies, discharges, and profiles of water surfaces during floods.

(3) Magnitude and Frequency of Floods-Reports on flood frequency and magnitude in major river basins in the conterminous United States are published in Water-Supply Papers.[3]

(4) Surface Water Records - Records of discharge and stage of streams and canals and contents and stage of lakes or reservoirs are published in Water Supply Papers, and the data are also included in an annual series.

In addition, the U.S. Army Corps of Engineers publishes reports of major floods, as does the National Weather Service, Tennessee Valley Authority, and other Government agencies. Information on floods has been published in Weather Bureau technical papers. Unpublished streamflow data may be available from States, Counties, or other political subdivisions, and water and power companies.

4.1.2 Man-made Changes. Works of man shall be identified such as reservoirs and levees that could have had an effect on historic floods. Extensive new studies to demonstrate such effects are not implied.

4.2 Design Flood Bases

4.2.1 Flood-Causing Event. No single flood-causing event is an adequate design base for a power reactor. Usual principal factors are precipitation, antecedent moisture, and wind, but especial factors include dam failures from differing causes. Events that shall be considered to determine the controlling flood elevations are one, or appropriate combination of any of the following as outlined in Section 9, Combined Events Criteria:

(1) Precipitation and snowmelt induced flood
  (a) primary watercourse (most obvious, usually the largest)
  (b) adjacent watercourse (smaller, nearby watercourses)
  (c) site and roof drainage.
(2) Failure of dams and other manmade structures from hydrologic seismic, or other causes upstream, downstream, and onsite.
(3) Landslide
(4) Storm surge
(5) Seiche
(6) Wind wave action
(7) Ice jam
(8) Channel changes and blockages (not covered in this standard)
(9) Tsunami (not covered in this standard)
(10) Volcanic eruption (not covered in this standard)
(11) Glacier (not covered in this standard).

4.2.2 Flood Summary. The flood causes and combinations thereof that determine controlling flood elevation shall be itemized. Other flood conditions that were evaluated to support this conclusion shall be described briefly. Present or refer to the flow and elevation hydrographs and water surface profile of the controlling flood.

4.3 Flooding Potential. Flooding potential and the capability of the plant to accomodate such flooding safely shall be summarized.

4.3.1 Intent of Design. The plant shall be designed: to maintain the function of safety-related structures, systems and components under all flood conditions; or for safe shutdown in advance of flooding. The applicable condition shall be stated.[4]

4.3.2 Provisions For Plant Safety. The ability of safety-related structures, systems, and components to withstand the controlling flood situation shall be stated. Provisions made to prevent flooding of safety-related equipment, such as floor levels, diking, water seals, or submarine doors, shall be stated.

4.3.3 Loading Condition. The hydrostatic and hydrodynamic loading conditions for safety-related facilities during controlling flood situations shall be stated.

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[3] See Appendix A1. (3), for specific titles.

[4] These positions are further described in Section C of Regulatory Guide 1.59 (see footnote 2).

4.4 Effects of Lesser Floods. The structures, structure access, operating areas, and equipment which can be affected by floods less than the controlling flood shall be identified. The frequency and the basis for frequency estimates of such occurrence shall be indicated. The effects of such flooding on the reliability and utilization of plant safety systems and also on plant operation shall be stated. Where emergency action is required to assure the integrity of safety-related systems for floods less severe than design basis, the basis for emergency action implementation may be different and shall be identified.[5]

4.5 Erosion and Scour. Flood design considerations shall include a discussion of the erosion and scouring effects of the controlling flood situation, including the effects of lesser floods, or hydrologic events considered in 4.2.1.

4.5.1 Design Bases. The design bases for the determination of channel velocities, wave heights, setup, runup, embankment, slopes, and thickness and composition of erosion protection layer shall be summarized.

4.5.2 Protection Works. All engineering works for channel training and stabilization or shore protection and other pertinent structures, failure of which will affect plant safety shall be summarized and stated.

4.6 Sedimentation. Flood design considerations shall include a discussion of sedimentation effects on design basis floods, and on lesser floods or hydrologic events considered in 4.2.1.

5. Probable Maximum Flood from Precipitation

5.1 General. A probable maximum flood shall be one of the design bases to assure safety of nuclear power plants. It shall be derived from a combination of circumstances which collectively represent a risk probability which is acceptable for nuclear reactor plant accidents. (See Section 9, Combined Events Criteria.)

5.1.1 Causative Combustion. Probable maximum flood flows and elevation can result from various combinations of sequential precipitation; centering, time and areal distribution, storm duration, and seasonal variations of precipitation; antecedent snowpack and related meteorological parameters; antecedent soil moisture; initial reservoir elevations in regulated watersheds; flood-caused dam failures both upstream and downstream; ice jams; and superimposed wind waves.

5.1.2 Flooding Sources. Location of a nuclear power plant near a primary watercourse should not obscure the potential for plant flooding from less obvious adjacent streams. Plant safety shall be assured against flooding from all sources. Probable maximum flood determination for strictly plantsite drainage is described in Section 11, Plantsite Drainage.

5.1.3 Flood-Dry Site. A flood-dry site is one where safety-related structures are so high above potential flood sources that safety from flooding is obvious or can be documented with minimum analysis. A descriptive statement of circumstances and relative elevations may be sufficient. Analogy may be drawn with comparable watersheds for which probable maximum flood levels have been determined. Approximations of probable maximum flood levels may be used. If a margin of safety is not this assured, the site is not clearly flood-free and a detailed analysis shall be made.

Flood studies for dry sites should be carried only to the degree of detail required to prove that safety-related structures are safe from flooding. All methods and assumptions shall be conservative. Procedures that may be utilized are described below.

5.1.3.1 Approximation Procedures for Natural Flood. One approximation particularly applicable to "hilltop" or "dry" sites for natural floods follows:

(1) Estimate probable maximum flood peak discharge on the basis of drainage area relationships to discharge derived from available probable maximum flood studies in the region. The basis of each study used shall be stated.[6]

(2) Estimate river stage using Manning's equation, average river channel bottom slope, river cross sections, and conservative friction factors.

(3) Elevation should be tested for sensitivity to potential errors in estimated values.

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[5] Pertinent references to determine lesser floods are given in Appendix A1.

[6] Regulatory Guide 1.59, Appendix B, presents alternative methods of estimating probable maximum floods (east of the 103rd meridian).

5.1.3.2 Approximate Procedures for Dam Failure. An approximation particularly applicable to "hilltop" or "dry" sites for dam failure follows:

(1) Select upstream dams within range of influence. Tandem structures should be assumed to fall like "dominoes." Parallel structures may be assumed to have common times of failure.

(2) Assume instantaneous disappearance of each dam under reservoir levels corresponding to the probable maximum flood.

(3) Compute breach wave height, $h = 4(\text{headwater-tailwater})/9$. Transfer downstream without attenuation with the local PMF values. Channel restriction and obstruction downstream from the plantsite should be considered.

5.1.4 Probable Maximum Flood Determination. Subsequent paragraphs describe more accurate methods that shall be utilized when more refined flood and dam failure studies are required.

5.1.5 Base Flow. Mean monthly flow during the month of occurrence of the PMF may be used as the base flow at the beginning of an antecedent storm.

5.2 Probable Maximum Precipitation. Probable maximum precipitation estimates for the United States are available in generalized studies prepared by the National Weather Service. They are presented in 12.1. These should be used whenever applicable.

5.2.1 Maximization Procedures. If the area of interest is not covered by the aforementioned reports, usually because of size of drainage area, transposition and maximization of major historic storms, depth-area-duration studies, or storm sequence studies may be necessary. The use of such procedures in obtaining the probable maximum precipitation shall conform with the methods used in most National Weather Service hydrometeorological reports.[7]

5.2.2 Types of Probable Maximum Precipitation for Consideration. Throughout most of the United States, thunderstorms are generally more critical than cyclonic storms for small areas. Both types of storms should be investigated. For areas where thunderstorms are never critical, documentation for this condition shall be provided.

5.2.3 Subareas. For large areas, probable maximum precipitation estimates require a breakdown of depth for individual areas. One method transposes a selected maximized isohyetal pattern to the total drainage area and determines the depth for each subarea. Another method places the most intense rainfall from depth-area-duration curves over the most critical portion of the basin and derives depths for successively larger total areas. Precipitation depths are then calculated for adjacent and mutually exclusive subareas.

It is normally assumed that rain occurs simultaneously over a total watershed. However, if there is sound meteorologic basis for assuming that the design storm will move with time within the basin, resulting influence on flows and elevations at the plant shall be determined.

5.2.4 Time Distribution. Probable maximum precipitation shall have the most critical time sequencing reasonably possible for the region. Guidance in determining time sequences may be obtained from observed storms. Storms often show characteristic groupings of the highest increment in several bursts. The following guidelines are acceptable:

(1) Group the four heaviest 6-hour increments of the probable maximum precipitation in a 24-hour sequence, the next highest four increments in a 24-hour sequence, etc.

(2) For the maximum 24-hour sequence, arrange the four 6-hour increments ranked 1, 2, 3 and 4 (maximum to minimum) in the order 4, 2, 1, 3. Other days may be arranged in similar order.

(3) Arrange the 24-hour sequences so that the highest period is near the center of the storm and the second, third, etc., are distributed in a manner similar to (2) above.

5.2.4.1 Distribution of 6-Hour Increments. The maximum 6-hour increments may further be distributed into smaller time increments as recommended in the references in 12.2, and references (1), (6) and (11) of 12.1.[8]

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[7] Guidance for transposition and maximization are given in the references in Appendix A2.

[8] Additional guides are given in the references in Appendix A4.

5.2.5 Storm Centering. For nonorographic areas, placing the storm center near the downstream end of the drainage area frequently produces the most critical condition for peak discharge. Enough storm center positions shall be considered, however, to assure that the most critical condition has been determined.

For areas with strong orographic effects, the centering of the probable maximum precipitation shall be based on methods of reference (1), (6) and (11) of 12.1, or as more specifically proposed by the National Weather Service.

5.2.6 Seasonal Considerations. Because of differing seasonal characteristics of storms, watershed characteristics, and reservoir operating rules, probable maximum precipitation values for different seasons shall be tested to assure that the controlling storm and resulting flood have been determined.

5.2.7 Associated Meteorological Activities. Meteorological events sequential with a primary rain requiring consideration include rain, snow, snowpack, temperature, and wind.

5.2.7.1 Sequential Storm. A sequential cyclonic storm could precede or follow the probable maximum precipitation with a three- to five-day period between storms, depending upon the meteorological characteristic of the region. A preceding storm is usually the more critical, but an alternative subsequent timing should also be investigated. For a thunderstorm probable maximum precipitation, the sequential thunderstorm could occur on the day before or after the major storm, but the combined storms should not exceed the probable maximum precipitation depth-duration values for a two-day duration. Combinations of precipitation and other events are specified in Section 9, Combined Events Criteria.

For those regions in which specific studies have been developed, the time sequencing of storms should follow the recommendation of those reports. Examples are given in 12.3.

In areas not covered by the references, a study of storms in the region should be used to guide storm sequencing.

5.2.7.2 Time Distribution. Antecedent or subsequent storms should be distributed in accordance with typical regional storms. Such studies are shown in the references in 12.4.

5.2.7.3 Areal Distribution. Antecedent or subsequent storms may have the same configuration and centering as the probable maximum precipitation.

5.2.8 Precipitation Losses and Precipitation Excess. Precipitation excess, needed to compute the flood, may be estimated by subtracting losses from rainfall or directly from multivariable relationships derived from the watershed. Losses should be estimated from watershed records if available and otherwise from regional studies and should be appropriate for the season, anticipated land use, and combination of events collectively being considered. Soil group numbers derived from agricultural soil maps are useful.[9]

5.2.8.1 Documentation. The following shall be documented:

(1) Initial losses used

(2) Assumed or derived infiltration rates or infiltration indexes used

(3) Assumed or derived antecedent soil moisture condition.

5.3 Snowpack and Snowmelt Flood

5.3.1 General. For drainage areas where snowmelt contributes significantly to the controlling flood situation, two conditions shall be considered for antecedent snowpack accumulation:

(1) Probable maximum precipitation on snow

(2) Probable maximum snowpack with rain.

5.3.2 Probable Maximum Precipitation on Snow. Antecedent snowpack prior to a probable maximum precipitation storm should be of 100-year average exceedance for the season of the year during which the probable maximum precipitation occurs.

5.3.2.1 Temperature During Probable Maximum Precipitation. Temperatures as snowmelt factors during the probable maximum precipitation are equal to maximum dewpoints, using the simplifying assumption of a saturated pseudo-adiabatic atmosphere during the event. For large drainage basins it may be necessary to derive several temperature sequences based on the geographic location and topography of the region.

5.3.2.1.1 Distribution. Time distribution of temperatures (dewpoints) shall be fixed by the

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[9] Reference materials are available in Appendix A5.

adopted time distribution of the rain; i.e., the highest temperature (dewpoint) is made coincident with the highest period of rainfall.

5.3.2.1.2 Seasonal Variation. For areas where the all-year probable maximum precipitation does not occur in the same season as the maximum snowpack, two or more months shall be studied, including the season embracing the maximum 100-year snowpack to assure that the controlling combination will be determined.

5.3.2.1.3 References. References (6) and (11) of 12.1 present methods to determine temperatures in their areas of application. For other areas, recorded temperatures can be obtained from National Weather Service (formerly U.S. Weather Bureau) climatological reports.

5.3.2.2 Temperature Prior to Probable Maximum Precipitation. Temperature prior to the onset of the probable maximum precipitation is a factor in determining condition of the snowpack. High antecedent temperatures shall be applied so snowmelt runoff will peak at the plantsite at the same time as that of the probable maximum precipitation runoff. For large drainage basins it may be necessary to derive several temperature sequences based on the geographic location and topography of the region.

5.3.2.2.1 Determination. Highest mean daily temperatures observed prior to major winter rains shall be determined. From an envelope curve of the differences between these prior temperatures and the temperatures during their corresponding storms, temperature differences are determined for addition to the temperature at the beginning of the probable maximum precipitation determined in 5.3.2.1. See reference (6) of 12.1.

5.3.2.2.2 Distribution. The time distribution of prior temperatures may follow the recorded distribution of temperatures enveloped in 5.3.2.2.1.

5.3.2.3 Wind During Probable Maximum Precipitation. If wind is used for maximization of the probable maximum precipitation, the same windspeeds shall also be used for snowmelt wind. If wind is not used in maximization, an envelope of the maximum free air wind with adjustments during major storm periods shall be used. Adjustments shall include seasonal and latitudinal variations and reduction to velocities which would be expected at the surface of the snowpack.

5.3.2.3.1 Distribution. The time distribution of windspeed should follow the same pattern as the temperature and the probable maximum precipitation.

5.3.2.3.2 References. References (6) and (11) of 12.1 present acceptable methods of determining wind during the probable maximum precipitation. For other areas similar procedures may be adopted.

5.3.2.4 Other Meteorological Melt Factors. For the case of probable maximum precipitation on snow, influences of other meteorological melt factors such as relative humidity, solar radiation, and albedo are relatively small. Nominal amounts may be assumed if desired, although in general they need not be considered. The most important melt factors during the probable maximum precipitation are temperature and wind.

5.3.3 Probable Maximum Snowpack With Rain. A 100-year rainstorm of critical duration during the main snowmelt period shall be superimposed on the probable maximum snowpack.

5.3.3.1 Probable Maximum Snowpack. The physical limit of snowpack water equivalent will usually occur during the spring snowmelt season. This can be estimated by detailed studies of total winter season PMP, with assumed conservative percentages falling as snow. From such studies maximum winter snowpack for specific basins may be derived. Variation of snowpack with elevation is determined on the basis of normal variations of precipitation and temperature with elevation (orographic effects). The snow wedge so derived represents the probable maximum flood-producing snowpack. This method is described in the reference of 12.6.[10]

5.3.3.2 Coincident Storm. The equivalent 100-year rainstorm of critical duration that could occur during spring snowmelt season may be estimated from point rainfall using a depth-area relation based on a mean annual precipitation isohyetal map. For areas east of the 105th Meridian, depth-area-duration relations contained in reference (2) and (7), Section 12.1 may be used. Point precipitation data are contained in the references in 12.5.

5.3.3.2.1 Time Distribution. The time distribution of coincident 100-year rainfall on probable maximum snowpack shall include the

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[10] Other methods are described in the references in Appendix A6.

100-year occurrence for all time periods.

5.3.3.2.2 Duration. The duration of the rainfall shall be based on the regional characteristics of the basin. The maximum rainfall rates of 100-year rainstorm shall be timed so that the rain begins at the peak of the snowmelt flood.

5.3.3.3 Air Temperatures. Air temperatures coincident with snowmelt period should be based on an envelope of the highest recorded temperature of the region during snowmelt periods.

5.3.3.4 Dewpoint Temperature. Dewpoint temperatures are used to represent vapor pressures to compute the convection-condensation component of snowmelt equations. In the absence of recorded dewpoint temperature, dewpoint may be assumed to equal air temperature during rain periods. In rainless periods minimum air temperature may be considered equal to dewpoint temperature.

5.3.3.5 Temperature Distribution. The air and dewpoint temperatures may be time distributed according to the mean daily temperature at representative stations with long-term records.

5.3.3.6 Solar Radiation. Watershed insolation values compatible with regional conditions shall be used. Differences between rain and rain-free periods shall be considered. A mean daily average is adequate in distributing solar radiation values over the period of snowmelt.

5.3.3.7 Albedo. Reflectivity, or albedo, values vary from 80 percent for new fallen snow to 40 percent for melting, late-season snow. Time distribution may be assumed linear and progressively decreasing during snowmelt.

5.3.3.8 Wind. Wind values typical of the region should be used; adjustments for topographic features of the watershed may be necessary.

5.3.4 Data Sources. Data on temperature, wind, and solar radiation may be obtained from National Weather Service reports.

5.3.5 Snowmelt Model. Hydrograph synthesis requires a method for evaluating the time delay to runoff for all components of basin storage, including the effect of transitory storage in the snowpack, soil, ground-water aquifers, and surface stream channels. For conservatism in PMF estimates, snowpack storage allowance shall be minimal.[11]

5.3.6 Computation of Snowmelt. Daily quantities of basin snowmelt may be determined either by the energy budget method or the degree-day method.[12] When smaller time increments are required, a fixed distribution pattern may be applied to the daily snowmelt values.

5.3.7 Elevation Effects. Because snowmelt exhibits its principal variation with elevation, the drainage basin should be divided into bands of equal elevation (1,000-foot intervals are commonly used) and the snowmelt, rainfall, and losses computed separately for each band.

5.3.8 Losses. Initial requirements of water to meet the soil moisture deficiencies are generally assumed to be satisfied prior to the occurrence of the conditions specified in 5.3.1. Evapotranspiration loss is disregarded.

5.3.9 Antecedent Conditions. For the conditions of 5.3.1, antecedent precipitation need not be considered; however, optimum conditions to produce maximum runoff shall be assumed including snowpack, meteorological factors, and ground conditions. It shall be assumed that the preceding melt of rainfall has provided drainage channels through the snowpack and has conditioned it to produce runoff without significant delay, so that water excesses from rain and snowmelt during the storm period are immediately available for runoff.

5.3.10 Modeling. Computations of snowmelt are usually performed by computer programs[13] and are generally included in the runoff and streamcourse models of 5.4.

5.3.11 Requirements For Review. To facilitate an independent review and evaluation, the bases and results of the following shall be discussed and summarized.

(1) Initial snowpack characteristics

(a) Area of snow cover

(b) Snowpack depth, water equivalent and distribution with respect to elevation

(c) Snowpack condition with respect to temperature and free water, and variation with elevation

(d) Albedo of the snow surface for basins with significant open areas

(2) Critical sequence of meteorological factors affecting melt

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[11] Commonly used techniques are described in the references in Appendix A6 and A7.

[12] Computations of snowmelt are described in the references in Appendix A6.

[13] Computer programs to model snowmelt are given in Appendix A7.

(3) Rainfall time distribution and total amount

(4) Normal annual precipitation of the basin

(5) Snowmelt rates, utilizing appropriate general equations and including forest cover estimates

(6) Loss and runoff conditions

(7) Verification and synthesis of factors affecting runoff.

5.4 Runoff and Streamcourse Models. A runoff model translates precipitation excess over a watershed to its resulting time variant rate of flow. It is often necessary to divide a watershed into subareas on the basis of size, drainage pattern, installed and proposed regulation facilities, vegetation, soil and cover type, and precipitation characteristics. These subarea runoff models are connected and combined by streamcourse modeling.

5.4.1 Runoff Models. Unit hydrographs and empirical formulas are common runoff models. The model must be verifiable and conservative.

5.4.1.1 Unit Hydrographs. If unit hydrographs are to be used, they shall be derived from rainfall and streamflow records when such records are available. In some cases it may be desirable to install one or more stream and rainfall gages to measure hydrologic data during planning and design of the plant. In the absence of observed records, synthetic derivations may be used.[14]

5.4.1.2 Empirical Formulas. Empirical formulas may be used for watersheds of a few hundred acres in which runoff characteristics are not influenced by regulation. Demonstrably conservative coefficients for empirical formulas shall be used.

5.4.1.3 Other Methods. Rainfall-runoff phenomena are also modeled by other methods.[15] Such method of analysis shall be described.

5.4.1.4 Adequacy of Transposed Parameters. If runoff model parameters are derived from a gaged subarea of the basin, their extension and applicability to other subareas of the basin shall be verified. This may consist of applying models for subareas and verifying their combined flows at a gaging station. Extension of parameters within a watershed or from a nearby watershed should be based upon vegetation type and density and general topographic and geologic similarities or differences between the gaged and ungaged subareas.

5.4.1.5 Linearity of Runoff Model. Peak ordinates of unit hydrographs derived from major historical storms and floods may exhibit larger or smaller values at earlier or later times than those obtained from minor events. Such nonlinear response shall be considered in deriving a runoff model for the high-intensity rainfall or probable maximum precipitation. Adjustment for nonlinearity may be based on recorded watershed performance and judgment. The basis for judgment shall be stated.

5.4.1.6 Verification of Runoff Model. Deterministic simulation models including unit hydrographs should be verified or calibrated by comparing results of the simulation with the highest two or more floods of record for which suitable precipitation data are available.

For ungaged basins adopted relationships between basin parameters and characteristics of the synthetic unit hydrograph shall be verified by regional study. For example, synthetic coefficients may be justified from derived unit hydrographs for similar basins in the region under study which have comparable characteristics.

5.4.2 Streamcourse Model. Floodwater travel in open watercourses shall be modeled by transient flow (unsteady flow) techniques when rate of flow change is rapid, as in the case of surge from dam failure. For normal rates of flow change common to most natural floods, simplified storage routing techniques are adequate.

5.4.2.1 Transient Flow. The transient flow method, also referred to here as the hydraulic method, computes simultaneously the time sequence of both flow and water surface elevation over the full length of stream. The solution requires knowledge of the lateral and longitudinal geometry of the stream, its frictional resistance, a discharge-elevation relationship at one boundary, and the time varying flow or elevation at the opposite boundary.[16]

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[14] Unit hydrograph derivation and application are described in the references in Appendix A8.

[15] Other runoff models are given in Appendix A9.

[16] Transient flow modeling is described in the references in Appendix A10.

5.4.2.2 Storage Routing. The storage routing method is also referred to as the hydrologic method. It relates time sequence of flow, but not elevation, to change in storage. Corresponding elevations are computed in an auxiliary analysis. The approximations, with their attendant cost savings, have practical application in evaluating power reactor plant safety from floods.[17]

5.4.2.2.1 Suitability. The storage routing model may be used when rate of flow change is moderate.

5.4.2.2.2 Flood Elevations. Relationships from which elevations or water surface profiles can be determined from routed flows are described in several hydraulic and hydrologic textbooks.[18]

5.4.2.3 Verification of Streamcourse Model. There is no literal way to verify a streamcourse model with probable maximum flood conditions. However, a model derived from valley dimensions and plausibly derived elevation-flow relationships which has successfully reproduced historical events, stands on its own. Reproduction of more than one large observed floods (if available) should be included.

5.4.3 Combined Models. Mathematical models are available which combine runoff and streamcourse models[19] to simulate the outflow response of an entire watershed of many parts. These are acceptable for estimating the probable maximum flood if performance can be verified in more than one large flood and if adequate data are supplied to permit an independent review and evaluation of the work.

5.4.4 Review Requirements. For review of consistency and adequacy of runoff and streamcourse models, appropriate indexes, coefficients, and dimensional data shall be provided.

5.4.4.1 Runoff Model. The following shall be provided for the review of runoff models:

(1) Size of watershed in square miles
(2) Method used
(3) Rainfall-streamflow records or synthetic parameters used to derive the model
(4) Unit hydrograph ordinates in graphic or tabular form
(5) Equivalent synthetic unit hydrograph coefficients if appropriate
(6) Any other pertinent factors
(7) Verification of the runoff model for more than one large flood.

5.4.4.2 Streamcourse Model. The following shall be provided for the review of streamcourse models:

(1) Length of streamcourse
(2) Method used
(3) Data from which storage routing models were developed including parameters and coefficients
(4) Reach length separations used for profile models or unsteady flow models
(5) Cross sections over a distance of reasonable influence spanning the plantsite, including a map showing location of cross sections
(6) Channel roughness coefficients
(7) Boundary conditions
(8) Verification data and results of model use on more than one large historical flood.

5.5 Hydrologic Dam Failures. The surge from a flood-caused dam failure is as much a component of a plantsite probable maximum flood as the probable maximum precipitation creating the flood. All dams above the plantsite shall be considered for potential failure, but some may be eliminated from further consideration because of low differential head, small volume, distance from plantsite, and major intervening natural or reservoir detention capacity. Others need further study. In dam failure analysis, not only initial condition, but also physical changes during the event need consideration in flood wave analysis.

Consideration of upstream dams should include all water impounding structures whether or not defined as dams in the traditional sense. Mine waste dumps and highway fills across valleys are examples. Their failure surges are as real as failure surges from dams of traditional definition and their failure is more probable.

Analyses of dam failure are complex, many failures not being completely understood. The principal uncertainty involves likely mode and degree of failure. Uncertainties can be circumvented in situations where it can be shown that the complete and sudden disappearance of a dam or dams will not endanger the nuclear plant. Otherwise, reasonable failure postulations are needed.

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[17] Storage routing is described in the references in Appendix A11.

[18] Computation of water surface profile is described in the references in Appendix A12.

[19] Some combined models are given in Appendix A13.

5.5.1 Procedure. Potentially critical dams should be subjected analytically to the probable maximum flood from their own contributing watershed. If a dam can sustain this flood, no further hydrologic analysis is needed. If failure is likely, the degree and mode of failure shall be estimated, and the resulting flood wave, combined with downstream flows that would prevail in this flood, shall be routed to the plantsite. If the watershed controlled by the upstream dam is a major part of the total plantsite watershed, intervening storm depths for computing flows below the dam may be estimated from the outer isohyetals of the storm pattern or extension of depth area curves used to evaluate the dam.

If an upstream dam would likely fail in the probable maximum flood from its own watershed, it shall also be tested in the probable maximum flood applicable to the total plantsite watershed. If judged likely to fail in either case, the resulting flood wave shall be carried downstream to the plantsite for comparison and selection of the critical case.

A third condition needs investigation if a significant part of the plantsite watershed lies below the dams: the PMP centered over the intervening area combined with dam failure waves from the same storm centering.

5.5.2 Domino Failures. A dam within the range of influence which otherwise would be safe in probable maximum floods may fail as a result of such a flood augmented by flood waves from an upstream hydrologic dam failure. Integrity of all dams lying in the path to the plantsite shall be analyzed and failure shall be postulated unless safety from failure can be decumented. Surges from all likely dam failures shall be routed to the plantsite.

5.5.3 Downstream Dams. If failure of a downstream dam would lower probable maximum flood elevations at the plantsite, certainty of such failure shall be documented if credit is to be taken for the flood lowering.

5.5.4 Failure Postulation. Mode, degree, and likelihood of dam failure from floodwater surcharge cannot be predicted with usual engineering accuracy. Failure shall be postulated unless safety from failure due to erosion, sliding or overturning can be documented by engineering computations. Mode and degree of failure should be postulated using conservative judgment based to the extent possible on stability computations.

5.5.4.1 Expedient Evaluation. Uncertainties and time-consuming, costly analyses may be circumvented if postulated instantaneous dam disappearance can be shown to present no flood danger to the proposed plant. Disappearance of dam is the worst case and hence represents the extreme of conservatism.

5.5.4.2 Detailed Analysis. Detailed stability analysis of dams requires documentation of structural dimensions and condition from design plans; construction records; records from installed instrumentation; field surveys; onsite inspections; and special strength testing, coring, and instrumentation.

5.5.4.2.1 Concrete Sections. Concrete gravity dams shall be analyzed against overturning and sliding. With some blocks judged likely to fail and others not, the mode and degree of probable failure can be judged as well as the likely position and amount of downstream debris. From this analysis, the water path and the likely elevation-discharge relationship applying to the failed section can be estimated with reasonable accuracy. Rise of tailwater should be considered in the stability analysis.

5.5.4.2.2 Arch Dams. Arch dams can usually sustain considerable overtopping with failure most likely from foundation and abutment failure. Unless safety can be documented, failure should be postulated. Failure of an arch dam may approach instantaneous disappearance with minimum residual downstream debris.

5.5.4.2.3 Earth and Rockfill. Earth and rock embankments shall be evaluated for breaching from overtopping. Erosion computations require defining an initial breach length. If there are two or more independent embankments, it may be necessary to fail only one if it produces the most critical flood wave.

Initial erosion investigations may begin with two trials:

(a) Probable maximum flood surcharge level plus maximum (1 percent) wave height resulting from sustained 2-year windspeed[20] applied in the critical direction.

(b) Normal operating level plus maximum (1 percent) wave height based on the probable maximum gradient wind.

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[20] The extreme which has a 0.5 probability of occurrence in any year.

If no failure is demonstrated, the evaluation may end and the embankment may be declared safe. If failure is likely, an initially breached section or notch should be identified or postulated. Subsequent erosion computations to determine the time and rate of failure may ignore wave-induced additive levels.[21] The outflow hydrograph during the period of erosion results from the computation.

Different methods have been proposed to predict the time varying shape and extent of overtopping erosional failure as a function of dam and reservoir characteristics. The degree of conservatism varies widely among the different methods and is a function of each specific problem. It follows that the user must exercise caution in the selection of the method to be used for each site specific problem. Additionally, the sensitivity to parameter changes and event timing shall be thoroughly investigated for each analysis.

5.5.5 Failure Outflow Hydrograph. Outflow from a partially failed nonembankment dam relates to degree and mode of failure, the resulting headwater-discharge relationship, and the geometry and volume of the reservoir. Unsteady flow routing is appropriate to determine reservoir contributions to the outflow hydrograph for long, narrow reservoirs of significant volume. Otherwise, storage routing may be used. For instantaneous dam disappearance, unsteady flow methods are necessary to document plant safety in marginal situations. It may be necessary or desirable to couple the upstream and downstream unsteady flow models at the instance of dam disappearance in order to determine the outflow hydrograph at the failed dam.[22] Sensitivity to variation of estimated parameters shall always be investigated to assure conservatism in the analysis.

5.5.6 Historical Dam Failures. To aid in arriving at reasonable dam-break mechanics, it is recommended that postulated failures be compared with historical dam failures.[23]

5.5.7 Flood Routing. Unsteady flow methods are most suitable for downstream routing of dam failure surges at the plantsite. Where distances are considerable, however, and if intervening channel or reservoir storage can be shown to attenuate surge flows adequately, less costly storage routing may be substituted. Flood routing techniques are referred to in 5.4.2 under streamcourse models.

5.5.8 Reservoir Operation. Information should be provided on mode of operation of all reservoirs involved in dam failure studies. Included should be the area-capacity curves, spillway and outlet rating curves, operating rules, and water commitments.

5.5.9 Controlling Flood Situation. The controlling flood situation shall be the most critical combination of rainflood, snowmelt flood if any, and any related dam-break surge.

5.5.10 Waterborne Objects. Hydrologic safety analysis shall also include the potential influences of waterborne missiles upon dam appurtenances such as gates and their operating machinery, bridges on the dams, lock gates, and other pertinent facilities, and combinations thereof. For example, destruction of a bridge over a spillway may interfere with gate performance, trash may block gates, and drift may destroy gates and thus release a surge of water. Credit for turbine flow during the flood sequence must be documented.

5.5.11 Wave Runup Additives. Safety analyses of hydrologic dam failures should impose concurrent wave runup from appropriate winds during the flood crest at the plant.[24] See Section 9, Combined Events Criteria.

5.5.12 Landslides. Soil stability conditions coincident with severe precipitation or reservoir fluctuations may produce landslides into rivers and reservoirs, resulting in flood waves contributing to hydrologic failure of a downstream dam. Landslides, rockfalls, and icefalls may also create temporary debris dams causing backwater flooding.[25] The subsequent failure of such obstructions can create a downstream flood wave. If the geologic character of the reservoir shoreland demonstrates the unlikelihood of this phenomena, it may be thus dismissed.

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[21] Useful references on erosional failure are given in Appendix A14.

[22] Derivation of a failure outflow hydrograph is described in the references in Appendix A15.

[23] Some references of historical dam failures are given in Appendix A16.

[24] References which are useful in wave runup determination are given in Appendix A17.

[25] Some references are given in Appendix A18.

5.6 Sediment Transport. Aggradation or degradation from sediment transport may increase or decrease flood elevations and should be considered.

5.7 Coincident Wind Wave Activity. Coincident wind shall be used to calculate increase in stillwater levels due to wind-generated setup, wave generation, and runup. Design water levels and static and dynamic forces shall include coincidental effects of wind-generated setup and wave activity superimposed on flood crests.

Wind of enough significance to affect flood safety of power reactor plants would necessarily be from a relatively severe set of meteorological conditions. Such conditions can move into an area during the three to five days required for cyclonic PMP storm to move out of that area. In small watersheds critical winds may be coincident with the PMP storm.

5.7.1 Criteria. Windspeeds to be associated with flood elevations at the power reactor site are specified in Section 9, Combined Events Criteria.

5.7.2 Wind-Generated Setup. Wind-generated setup shall be calculated as described in 7.3.3.1.

5.7.3 Wind-Generated Wave Activity. Wind-generated waves, runup, and static and dynamic forces shall be calculated as described in 7.4.

5.7.4 Design Water Levels. The increase in water levels due to wind wave activity that is to be superimposed on flood levels shall be determined as described in 7.4.6.

5.8 Probable Maximum Flood Summary. Concise summaries of probable maximum flood flows and elevations at the plantsite shall be presented preferably in tabular and graphic form.

5.8.1 Flood Discharges. Principal elements involved in estimating the controlling flood shall be displayed in summary form for convenient observation.

5.8.1.1 Precipitation. The following storm characteristics shall be summarized preferably in one or more tabulations.

(1) Average basin PMP for different seasons and trial storm centerings used to document the controlling storm

(2) Average basin precipitation of sequential storms evaluated or average basin water or melt equivalent of antecedent snowpack

(3) Average basin rainfall loss and precipitation excess for each storm tested

(4) Average rain, rain loss, and precipitation excess for each subarea of the watershed in the controlling PMP storm and in its accompanying sequential storm or the water or melt equivalent in its antecedent snowpack

(5) Time distribution of rainfall in the PMP and sequential storms. A tabulation or curve may be used.

5.8.1.2 Modeling. Methods of runoff and streamcourse modeling to determine discharge at the plant should be described briefly in generic terms with reference to sections which fully describe model dimensions, derivations, and verifications.

5.8.1.3 Dam and Reservoir Influence. For the controlling flood, regulating influence of upstream reservoirs shall be summarized briefly with quantification when practical. Influence of upstream and downstream dam failures shall be summarized briefly with quantification when practical.

5.8.1.4 Base Flow. State the method for estimating base flow and tabulate values used.

5.8.1.5 Flood Discharge Summary. Summarize preferably in tabular form crest flow at the plantsite which would result from each of the various storms tested to document that the controlling flood has been determined. Provide a tabular or graphic hydrograph of the controlling flood, noting thereon the time of any upstream and downstream dam failures.

5.8.1.6 Velocities. Summarize flood velocities near the plantsite.

5.8.2 Flood Elevations Summary. The methods of modeling to estimate plantsite elevations should be described briefly in generic terms with references to sections which fully describe model dimensions, derivation, and verification. Summarize preferably in tabular form plantsite elevations that would result from each of the various storms evaluated to document that the controlling flood elevation has been determined. For the controlling flood level, provide a crest profile generously encompassing the plant location. For the controlling flood, provide a tabular or graphic elevation hydrograph noting thereon the time of any upstream and downstream dam failures.

5.8.3 Wind Additives. A fetch map should be provided, and adopted wind velocities should be stated in reference to sections which document appropriate windspeeds. Resulting wind setup and wave runup on a vertical wall and on pertinent plant yard slopes and structures should be given.

5.8.4 Design Water Levels. Provide appropriate wave runups combined with the controlling stillwater elevation and compare these design water levels with the levels of safety-related facilities.

6. Nonhydrologic Dam Failures

6.1 General. Nuclear reactor safety from flooding must be assured not only in floods from extreme precipitation but in floods from other causes as well. Surges from upstream dam failures from nonhydrologic causes constitute potential threats. Analysis of such failures require multi-disciplinary approach. This section deals with dam failures caused by seismic activity and other causes. Seismic analysis requires consideration of dynamic loading.

6.1.1 Data. Detailed stability analysis requires documentation of conditions of structures. Periodic inspection for safety of dams is required by regulatory jurisdictions such as the Federal Energy Regulatory Commission for licensed projects, and by state agencies, and other regulatory bodies. Such inspection reports may be used in the analysis. Additional data may include strength testing of dam and foundation cores, field survey, consultant inspection, instrumentation, seismic and geologic information.

6.1.2 Procedures. The analyses of dam break may be outlined as follows:

(1) Postulate reservoir levels in a potential coincident flood (see Section 9, Combined Events Criteria)
(2) Determine loading
(3) Analyze stability
(4) Compute or postulate breach
(5) Determine breach outflow hydrograph
(6) Route breach hydrograph
(7) Determine flood level at plantsite.

In dam failure analysis, not only initial condition, but also physical changes during the event need consideration in flood wave determination.

6.1.3 Scope. All dams in the basin above the plantsite should be considered for potential failures. Some dams may be eliminated from detailed consideration because of low differential head, small volume, distance from plantsite, and major intervening natural or reservoir detention capacity. Volcanic eruptions that could cause dam failures should be investigated, but these guidelines do not cover such events.

Section 5, Probable Maximum Flood From Precipitation, describes dam failure concerns, considerations, and procedural techniques related mostly to hydrologic causes, many of which are equally applicable to dam failures from nonhydrologic causes. Among these are:

(1) Potentially flood-dry sites, 5.1.3
(2) Rainfall-time distribution, 5.2.5, and storm centering, 5.2.6
(3) Streamcourse models, 5.4.2, and flood routing, 5.5.7
(4) Domino dam failures, 5.5.2, with the added concerns for failures for nonhydrologic causes
(5) Downstream dam failures, 5.5.3, with added concerns for cooling water supply (not included in the standards)
(6) Dam failure postulations, 5.5.4
(7) Dam failure outflow hydrograph, 5.5.5
(8) Mode of reservoir operation, 5.5.8
(9) Waterborne objects, 5.5.10
(10) Landslides, 5.5.12.

7.3 and 7.4 describe calculation of coincident wind effects. Section 9, Combined Events Criteria, describes combinations of flood-producing events which need consideration for plant safety.

6.1.4 Stability Analysis. If analysis indicates a dam to be unstable, a conservative, likely breach should be postulated and the resulting amount and position of downstream debris should be estimated.

6.2 Seismic Dam Failures. Analytical procedures and guidelines to evaluate the consequences of hydrologic dam failures are discussed in 5.5. Similar approaches for seismic failures may be used when applicable. Appropriate earthquakes are considered in Section 9, Combined Events Criteria.

6.2.1 Earthquake Centering. The epicenter shall be evaluated in positions to assure the maximum failure surge from one or more upstream dams consistent with appropriate tectonic regions and seismic force attenuation.

6.2.2 Loading. Stability of concrete and earth sections should be evaluated against loads due to coincidental earthquakes and floods as described in Section 9, Combined Events Criteria.

6.2.2.1 Timing of Seismic Disturbance. For conservatism, any postulated breach should be timed to coincide with maximum reservoir level from the coincident flood, 9.2.1.2.

6.2.2.2 Domino Failure. Evaluation of dam stability against surges from upstream dam failures may recognize intervening attenuation during travel in rivers and reservoirs. This could influence successive (Domino) failures.

6.2.2.3 Dam Appurtenant Features. Seismic influence on dam appurtenances should be evaluated with respect to its influence on reservoir surcharge and resulting dam stability or breaching by overtopping. Sudden failure of gates from seismic influence should also be evaluated for its resulting downstream flood wave effect.

6.2.2.4 Other Forces. When detailed analysis is required, static forces normally considered in dam design should be considered as loads for dam-break stability analysis in addition to the dynamic forces of earthquakes. To a certain extent they must be based on judgment and experience. The static forces are:

(1) Dead load
(2) External water pressure
(3) Internal water pressure (uplift)
(4) Earth and silt pressure
(5) Ice pressure
(6) Wind pressure
(7) Subatmospheric pressure
(8) Wave pressure
(9) Reaction of foundation.

6.3 Dam Failures From Other Causes.

Potential dam failures from earthquakes are associated with sharply defined natural events of a few moments' duration and failures from extreme floods with natural events of a few hours', days', or weeks' duration. Dam failures from other, onsite causes may result from gradual changes in, under, and adjacent to the dam. With proper inspection and monitoring, gradual changes threatening dam safety may be detected and adequate corrective measures can be taken.

6.3.1 Failure Analysis. For any upstream dam, available records should be evaluated to appraise likelihood of failure. If dam safety cannot be so assured for the normal life of the nuclear plant, the dam shall be postulated to fail in a severe yet credible manner, and the resulting flood wave should be routed to the plantsite. Routing must accommodate induced failures of other dams on the path of the failure flood wave.

6.3.2 Failure Causes. Onsite potential causes of partial or complete dam failure include the following:

(1) Deterioration of concrete due to cracking, weathering, or chemical growth
(2) Deterioration of embankment protection such as riprap or grass cover
(3) Excessive saturation of downstream face or toe of embankment
(4) Excessive embankment settlement
(5) Cracking of embankment due to uneven settlement
(6) Erosion or cavitation in waterways and channels including spillways
(7) Excessive pore pressure in structure, foundation, or abutment
(8) Failure of spillway gates to operate during flood due to mechanical or electrical breakdown or clogging with debris
(9) Buildup of silt load against dam
(10) Excessive leakage through foundation or abutment
(11) Leakage along conduit in embankment
(12) Channels from tree roots or burrowing animals
(13) Excessive reservoir rim leakage
(14) Landslide in reservoir.

6.4 Failure and Failure Surge Analysis.

Hydraulic characteristics of a flood wave released from a dam failure are a function of the mode of failure, size, shape, and position of the breach and extent and position of downstream debris. They are also a function of the volume of water in the reservoir and the changing reservoir inflow and tailwater conditions at and after failure. These and surge routing techniques are described in 5.5.

When dams are found to be unstable, the various elements of a dam should be considered on the basis of the section or sections most likely to fail and the timing and sequence of such failure. The section most likely to fail may be an overtopped earth section; the least stable concrete gravity section; and erodible foundation at the toe of an overtopped section; a concrete gravity section of weaker concrete failing in shear, crushing, or tension; or other similar causes. Failure of one dam element may trigger failure of other sections.

6.4.1 Liquefaction Failures. The rate of liquefaction failure of embankments may be essentially instantaneous and the degree of failure should be considered comparable to complete disappearance unless documentation can be presented supporting a different mode of failure.

6.4.2 Data For Review. Sufficient information shall be provided to allow review of dam stability analyses, causes of dam failures, dam conditions after failures, and flood surge track to the power reactor site. More specifically:

(1) Candidate dams shall be identified by reference to the summary of information of 3.2.1 and watershed maps as appropriate

(2) Flood events made coincident with earthquakes shall be described

(3) Differing, hypothetical centerings of earthquake epicenters shall be described to document that the critical dam failure or combination of failures has been determined

(4) Stability analyses shall be provided to document safety of dams which will not fail and to support likely modes and degrees of failure of those that will

(5) Methods and their documented reliability for tracing failure flood surges to the reactor site shall be described in Section 6, Nonhydrologic Dam Failures, if reference to 5.5 will not suffice

(6) Flood levels at the reactor site, including wind effects, for differing epicenter locations shall be summarized to document that controlling conditions have been determined.

6.5 Summary. A summary of controlling failure situations shall be provided including:

(1) Discharge at the reactor site preferably in hydrograph form noting thereon time of upstream and downstream dam failures

(2) Elevation at the reactor site preferably in hydrograph form noting thereon time of upstream and downstream dam failures

(3) Crest water surface profile generously encompassing the plantsite area

(4) Map showing plant site area that would be flooded

(5) Elevation of safety-related facilities with reference to the controlling elevation, including wind effects.

7. Probable Maximum Surge and Seiche Flooding

7.1 General. Probable maximum surge and seiche flooding resulting from a probable maximum hurricane, probable maximum windstorm, or moving squall line shall be one of the design bases to assure the safety of nuclear power plants. These meteorological events shall be derived from a combination of parameters that would result in a surge or seiche which has virtually no risk of being exceeded in the region involved. In some cases sites can be demonstrated to be flood dry by techniques described in 5.1.3.

7.2 Probable Maximum Winds and Associated Meteorological Parameters

The determination of the probable maximum meteorological winds and parameters associated with the following meteorological events shall be presented in detail:

(1) Probable maximum hurricane

(2) Probable maximum windstorm

(3) Moving squall line.

7.2.1 Probable Maximum Hurricane (PMH). The probable maximum hurricane is defined in Section 2, Definitions.

7.2.1.1 Region of Occurrence. A PMH shall be considered for United States coastline areas and areas within 100 to 200 miles bordering the Pacific Ocean, Atlantic Ocean, Gulf of Mexico, and possessions in the Carribbean Sea. In addition, influence along estuaries and rivers connecting with these bodies of water should also be considered because hurricane storm surges will be transmitted upstream to some degree. In the Pacific Ocean, experience indicates that PMHs are not the controlling events because of other natural phenomena, i.e., tsunamis that can generate higher flood levels.

7.2.1.2 Meteorological Parameters. The PMH parameters are described by PMH winds, atmospheric pressures, and translational speeds.[26] These may be determined from reference 12.7.

7.2.1.3 Special Conditions. For those regions of interest not covered by the zones given in the references of 12.7, reasonable extrapolations and assumptions based on that reference should be used. For such cases, sufficient data and information to justify any assumptions or extrapolations made should be provided.

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[26] PMH parameters may also be determined using the references in Appendix 19.

7.2.1.4 Critical Combination of Parameters. Various combinations of the given ranges of radius of maximum winds, R, and forward speeds, T, as described in the foregoing, and maximum 10-minute sustained 33-foot wind, V_x, which varies with different values of R and T, should be input to the hurricane surge calculation to determine those critical combinations of parameters that would result in the most severe flood condition. A storm of larger R and faster T may generate a higher surge, but the duration of the peak surge activity may be much shorter in comparison with a storm of slower T. Therefore, several combinations of R and T shall be analyzed to determine peak surges and their durations for proper hurricane wave analysis and other considerations that ultimately determine final design flood conditions.

7.2.1.5 PMH Path. The maximum surge at a site usually occurs when the path of the PMH is to the left of the site in the northern hemisphere (when facing in direction of forward movement, i.e., landward) by distance R sin (115°). The path of the PMH is usually normal to the average bathymetric contours. This orients the maximum winds for favorable surge generation, but not necessarily for wave generation. In cases where this path can not physically occur due to the thermodynamics of hurricane generation and movement, then only the possible range in PMH paths shall be considered. The critical wind field direction should be established for maximum wave conditions within the storm rotational limitations. Reasonable routing of the PMH upstream into estuaries and rivers, where applicable, should be treated similarly.

7.2.2 Probable Maximum Windstorm (PMWS). The PMWS defined in Section 2, Definitions, is generally classified as an extratropical windstorm. It is a migratory frontal cyclone occurring in the middle and high latitudes, not a tropical cyclone. Extratropical cyclones usually produce their highest winds in the winter because they are energized mainly by temperature contrast between air masses most prevalent during the cooler seasons. Tornadoes need not be considered for design basis flooding.

7.2.2.1 Region of Occurrence. A PMWS should be considered for the locations along the Pacific Coast and North Atlantic Coast of the United States and large bodies of water such as the Great Lakes.

7.2.2.2 Meteorological Parameters. PMWS parameters are described by wind field pattern, pressure field, and track and forward speed of the storm center. These should be maximized and considered coincidentally to the extent possible as supported by meteorological evidence.

7.2.2.2.1 Parameter Determination. Determination of the various parameters of the PMWS usually involves detailed meteorological analysis of actual large-scale, severe, cyclonic windstorm events for the particular area. Such an analysis often requires storm transpositions with appropriate adjustments, modifications, and extrapolations of data to reflect a more severe meteorological wind system and associated parameters than have actually been recorded, insofar as these are deemed "reasonably possible" of occurrence on the basis of meteorological reasoning. The parameters thus derived are associated with the PMWS because they are determined within the limitations of current meteorological theory and available data and are based on the most effective combination of controlling critical factors.

7.2.2.2.2 Synthetic Windstorm. A single storm should be synthesized from these storms or one cyclone should be chosen and modified to produce the PMWS.[27] Synoptics of the transient nature of the windstorm, such as wind field pattern, pressure field, and forward movement of the storm center, shall be determined. This is used as a windstorm model for the adopted maximum overwater windspeed.

7.2.2.3 Development of Model. Three-hour surface maps of the selected storm should be analyzed for pressure, windspeed, and wind direction during the period the storm is near the area under consideration. Typical pressure and wind fields and isobar and isotach patterns are then used throughout the remainder of the PMWS surge analysis.

7.2.2.3.1 Basic Input. Parameters of the PMWS should be determined by a meteorological study. In lieu of such a study for the Great Lakes Region the following may be used:

(a) Set maximum overwater windspeed at about 100 miles per hour

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[27] References which can be used to study PMWS are given in Appendix A20.

(b) Set lowest pressure within the PMWS to about 950 millibars

(c) Apply a most critical, constant translational speed during life of the PMWS. This may require several trials

(d) Windspeeds over water vary diurnally from 1.3 (day) to 1.6 (night) times the overland speed. This assumption is based upon work by Lemire[28] with some modifications

(e) All winds blow 10 degrees across the isobars over the water body. Decreased friction over the water will cause the wind to approach the isobars, but gradient flow will not be reached because of the inbalance of forces.

7.2.2.3.2 Additional Assumptions. Additional assumptions introduced into the model follow:

(a) A quasi-steady state exists within the PMWS during the entire time that the storm affects the water body

(b) The PMWS center moves along a critical path so the zone of maximum wind travels along the major axis of the water body

(c) The PMWS does not occlude at any time while affecting the water body.

7.2.2.3.3 Overwater Windspeed. To obtain a maximum overwater windspeed of 100 miles per hour, multiply the observed winds of the selected storm by a factor which is the ratio of 100 miles per hour to the maximum windspeed of the synthesized or selected storm. An approximation of the pressure gradient may be determined by multiplying the same factor as that used for the winds.

7.2.2.3.4 PMWS Path. The PMWS is assumed to travel along its path maintaining a quasi-steady state wind field and constant translational speed. Hourly values of pressure, windspeed, and wind direction should be estimated for each grid point or zone of the water body.[29]

7.2.3 Moving Squall Line. Some weather systems which are too small to be followed in the standard synoptic network may produce significant effects. Such organized systems as squall lines, masses of thunderstorms, seabreeze systems, and large precipitation cells are in this category. Moving squall line is defined in Section 2, Definitions.

7.2.3.1 Region of Occurrence. A moving squall line should be considered for the locations along Lake Michigan where significant surges have been observed due to such a meteorological event. The possible region of occurrence includes other Great Lakes.

7.2.3.2 Meteorological Parameters. A synoptic model of a squall line[30] will provide thunderstorm high, wake depression and pressure-surge line.

Time-dependent pressure and wind velocity fields associated with the advancing speed of the squall line moving over a water body can be determined from the synoptic model, based on the probable maximum positive excess pressure which is reasonably possible.

A pressure jump of 8 millibars (0.236 in. Hg.) within ten nautical miles width of the squall lines, with a .65-knot windspeed can be used for the probable maximum meteorological conditions.[31]

To maximize the surge level on an open shoreline, the squall line speed should be equal to the resonant speed of the surge, which in turn is dependent on the hydrography of the water body. A range of squall line speeds combined with the most critical line track and angle of squall line with track should be tested for the point under study.

7.2.4 Requirements For Review. To facilitate an independent review, the probable maximum meteorological winds and associated parameters determined for each of the meteorological events (PMH, PMWS, and moving squall line) the following should be included:

(1) Detailed analyses of actual historical storm events in the general region of the site.

(2) Modifications and extrapolations of data to reflect a more severe meteorological wind system than actually recorded, insofar as these modifications and extrapolations are deemed "reasonably possible" of occurring on the basis of meteorological reasoning.

(3) Sufficient bases and information should be provided to assure that the parameters presented are the most severe combination.

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[28] See reference 1, Appendix A20.

[29] An example of this analysis is presented in reference (3) of Appendix A20.

[30] See reference (1), Appendix A21.

[31] References for use in studying moving squall line are given in Appendix A21.

7.3 Surge Water Levels. Abnormal rises in water elevation in nearshore areas of water bodies induced by meteorological windstorm disturbances are generally referred to as storm surge. In open coastal areas the water level rise can be represented by a single peak surge hydrograph which corresponds to the cyclonic windstorm which passed over the point under study. In an enclosed or semienclosed body of water, such as a lake or harbor, the windstorm may cause oscillation of the water surface, and a multipeak surge hydrograph may result. This long-period oscillation of the water body is often called a seiche.

7.3.1 Open Coastal Regions. Open coast is that portion of land directly exposed to and with shorefront conterminous with a major body of water such as the Gulf of Mexico, Atlantic Ocean, Caribbean Sea, etc. The Great Lakes are not considered open coastal bodies of water.

7.3.1.1 Probable Maximum Hurricane (PMH). In calculating open coast storm surge, components described in subsequent sections below should be considered.

7.3.1.1.1 Astronomical Tide. In computing the probable maximum surge level on the open coast, the ten percent exceedance high tide[32] is considered to occur coincidentally with the PHM storm surge. This tide can be determined from recorded tide or from predicted astronomical tide tables (see reference (2) of 12.8). If predicted tides are used, sea level anomaly must be added. A constant tide elevation or the actual predicted periodic tide can be used, provided the calculated peak surge occurs during the peak astronomical tide elevation.

7.3.1.1.2 Sea Level Anomaly (Initial Rise). An anomalous departure of the water surface elevation from the predicted astronomical tide has been observed to occur. This phenomenon is called sea level anomaly and is estimated by comparing long-term recorded and predicted tides. Effect of regional meteorological parameters such as barometric pressure and winds on water level are included in the recorded tide. For determination of the probable maximum surge, the sea level anomaly need not be included when ten percent exceedance high tide is based on recorded tides. If ten percent exceedance high tide is based on predicted tide levels sea level anomaly shall be added. Whichever is lower may be used.

For predicted ten percent exceedance high tide and sea level anomaly the following table may be used.

7.3.1.1.3 Bottom Friction and Wind Stress Coefficients. The proper bottom friction coefficient and wind stress coefficients depend on the area under investigation and on the hurricane storm surge model utilized in the analysis. The formulations of bottom friction term are either linear, quadratic, or time-dependent depending upon the model used. It becomes a significant factor in surge computations in shallow depth areas, and would be less important in the deep stratified water body. Sensitivity analysis, together with calibration and verification procedures shall be exercised for each model to assess a set of reasonably conservative values for use in the determination of the design base flood level. For the computer model listed in reference (1) of 12.8, a bottom friction coefficient between 0.001 and 0.005 is suggested, assuming the depth is taken to the power of 2.0 in the dissipation term of the flux equation. A value of 0.003 for the East and Gulf Coasts is generally acceptable without justification. Other values which would result in lower surge height may be used if sufficient justification, such as verification studies, is provided.

Since the surge is a forced wave, the wind stress terms are critical. The main discrepancy among models is on the parameterization of the wind stress formulation, of which the wind stress coefficient may be constant or variable with wind speed or even varied with temperatures of air and water in the cold regions. Unless these coefficients have been estimated from observations of surge and current velocity at high wind speed levels, wind stress coefficients as discussed in reference (1) of 12.8 should be used unless

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[32] The ten-percent exceedance high tide is the high tide level which is equalled or exceeded by ten-percent of the maximum monthly tides over a continuous 21-year period.

Table 2

Tide Characteristics (Based on 21 Years of Predicted Tides)[33]

Station	10% Exceedance High Tide, Ft. Above MLW	Mean Sea Level, Ft. Above MLW	Sea Level Anomaly Ft.
Eastport, Maine	21.9	9.2	0.5
Portland, Maine	11.2	4.6	0.7
Boston, Massachusetts	11.9	4.4	0.9
Newport, Rhode Island	4.9	1.6	
New London, Connecticut	3.7	1.4	
Bridgeport, Connecticut	8.5	3.4	
Willets Point, New York	9.0	3.6	
The Battery, New York	6.0	2.3	1.0
Sandy Hook, New Jersey	6.2	2.3	
Breakwater Harbor, Delaware	5.6	2.1	
Ready Point, Delaware	6.6	2.8	
Hamilton Roads (Sewell Point), Virginia	3.5	1.3	1.1
Baltimore, Maryland	2.0	0.6	
Charleston, South Carolina	6.8	2.7	1.0
Savannah River Ent., Georgia	9.0	1.2	
Mayport, Florida	6.2	2.3	1.3
Miami Harbor Ent., Florida	3.6	1.3	0.9
Key West, Florida	2.3	0.6	0.9
St. Petersburg, Florida	3.0	1.2	**
Tampa Bay, Florida	2.6	0.7	**
St. Marks River Ent., Florida	4.1	1.8	**
Pensacola, Florida	2.0	0.6	**
Mobile, Alabama	2.3	0.8	**
Galveston, Texas	1.8	0.8	**
San Diego, California	7.6	2.9	***
Los Angeles (Outer Harbor), California	7.1	2.8	***
San Francisco (Golden Gate), California	6.8	3.0	***
Humboldt Bay, California	7.9	3.4	***
Astoria (Tongue Point), Oregon	9.9	4.3*	***
Aberdeen, Washington	12.4	5.6	***
Port Townsend, Washington	9.6	4.8	***

*Datum Below mean river level
**See Appendix C; Table C.1, R.G. 1.59
***Negligible

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[33] Use the above table as follows:

(a) From tide tables (reference 2, of 12.8) select second-order stations on either side of point of interest.

(b) Determine tide adjustment to appropriate first-order station.

(c) Adjust the ten percent exceedance high tide and mean sea level values in the table to obtain these values for the selected second-order stations.

(d) Interpolate between second-order stations for ten percent exceedance high tide and sea level anomaly at point of interest, taking due account of local conditions.

other values can be justified.[34] Further research using better historical data is required to verify wind stress coefficients different from those given in reference (1), 12.8.

7.3.1.1.4 Storm Surge Calculation. The probable maximum hurricane open coast storm surge is calculated using the long wave dynamic equations, as described in the references of 12.8.[35] Computer programs such as described in the references of 12.8 may be used. If a hurricane storm surge model or computer program other than those described above is used, the model or program shall be verified against observed hurricanes and demonstrated to be conservatively applicable for probable maximum hurricane conditions.

Most severe combinations of meteorological parameters with the hurricane moving along a critical path at an optimum rate of movement should be tested for in this analysis. Results of the computation of the probable maximum surge hydrograph shall be presented graphically. Factors for reducing hurricane windspeeds when the storm moves over land should be considered using table 2 of reference (2) of 12.7. For onshore movement reductions should be from table 3.

It is possible that the hurricane which generates the peak open coast storm surge may not be controlling. Hurricanes generating lower peak surges, but causing longer-duration high water levels, or hurricanes generating high windspeeds with their associated wave activity, could conceivably produce higher design water levels. Also, for sites located within a bay, hurricanes generating lower peak open coast surges but longer-duration high water levels could generate higher peak surges within the bay or higher design water levels. Erosion of beach berms and bay entrances should also be considered in this analysis where such erosion can produce worse flood conditions. Therefore, hurricanes other than those generating the peak open coast should be considered.

With any surge model, it might be necessary to correct the surge computations for overland flow.

7.3.1.2 Probable Maximum Windstorm (PMWS). Analysis for calculating storm surge peak water levels and hydrographs is similar to that presented for the probable maximum hurricane. Computer programs used for PMH analysis, with some modifications, may be used for PMWS analysis. Generally, a two-dimensional surge model should be used because the wind field of the the PMWS is not as well documented as that of the PMH.

7.3.1.3 Moving Squall Line. Surge generated by a moving squall line is insignificant compared to surges induced by a PMH or a PMWS in open coastal regions.

7.3.2 Semienclosed Bodies of Water. Semienclosed bodies of water are exemplified by lagoons, estuaries, rivers, and the like.

7.3.2.1 Probable Maximum Hurricane. For analysis of hurricane storm surge in semienclosed bodies of water, the open coast surge is routed through the entrance and up the bay river to sites of interest.

A transient one-dimensional model can be used to compute resonance effects for a narrow water body with bay entrance, while a two-dimensional transient analysis should be used for other shapes of water bodies. No separate computation is required for the open coast surge and routed surge if the area used in the two-dimensional model is large enough to cover the entire hurricane wind field such that water level rise at the model open boundary is negligible.

Hydraulic computations should consist of time-histories of open coast surge, discharge of water through the entrance, surge profile up the bay or river, contribution of setup due to crosswinds, and, if applicable, contribution due to runoff and river flow.

Ten percent exceedance high tide should be used as ambient water level for open coastal areas.

Critical storm translational velocity up the bay or river will generate the maximum surge at the site. Therefore, the combination of PMH parameters generating the highest open coast surge does not necessarily generate the highest surge at sites located in bays or estuaries.[36] For this reason, various combinations of storm parameters should be considered.

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[34] When using the model in reference (1) of Appendix A22, the 1.1 multiplier need not be used. This reference gives details of available historical data to date and their use.

[35] Additional references are given in Appendix A22.

[36] Appropriate techniques for PMH storm surge are given in the references in Appendix A23.

For sites located in bays with low-lying beach berms and low marshes, the problem of berm overtopping and flooding should be considered. Because of rates of inflow through the entrance and overflow across the beach berms and low land, open coast surge hydrographs with a lower peak surge elevation but longer duration may generate the highest surge elevation at sites, as compared to open coast surge hydrographs with a higher peak. Erosion of beach berms and bay entrances should also be considered in this analysis, where such erosion can produce worse flood conditions.

7.3.2.2 Probable Maximum Windstorm (PMWS). Analyses of probable maximum windstorms in semi-enclosed bodies of water are handled in the same manner as that described above for probable maximum hurricanes. The same techniques for routing the open coast PMWS surge, if open coast surge is first computed, through the entrance and up the bay or river to sites of interest are utilized.

7.3.2.3 Moving Squall Line. A moving squall line is not the controlling meteorological event in generating probable maximum surge in a semienclosed water body.

7.3.3 Enclosed Bodies of Water. Enclosed bodies of water are lakes and inland reservoirs.

7.3.3.1 Probable Maximum Hurricane. In enclosed bodies of water, analysis of storm surges due to hurricanes should be performed by using verified one- or two-dimensional mathematical models. When one-dimensional models are used, the transverse wind setup or a transverse seiche component must be calculated and added to the longitudinal wind setup.[37] Selection of coefficients and boundary conditions for the PMH should be based on conservative assumptions. If the water body is sensitive to resonance, transient responses should be considered in a one-dimensional model. The two-dimensional transient mathematical model takes into account the transverse components and resonance effects automatically. If the water body is not sensitive to resonance, analysis should be provided to substantiate insensitivity.

Components of the maximum probable stillwater levels are the ambient water level, the longitudinal wind setup, and the transverse or crosswind setup. The base or ambient water level upon which the surge or seiche is computed is the 100-year recurrence monthly average high water or the maximum controlled water level. Whichever is lower may be used. In determining the 100-year high water, the maximum value of the 12 monthly averages in each year should be obtained for the entire period of record and the yearly maximum values so obtained are then used for frequency analysis.

The critical portion of the hurricane's wind field is used in this analysis after being adjusted for any overland effects.

7.3.3.2 Probable Maximum Windstorm (PMWS). Analysis of the probable maximum windstorm surge elevations on enclosed bodies of water is similar to that for the PMH.

7.3.3.3 Moving Squall Line. Two-dimensional surge model used to determine surge levels for the PMH and PMWS can be adopted for analysis of probable maximum surge elevations due to moving squall lines.[38]

7.3.4 Maximum Stillwater Levels From Surges or Seiches. Probable maximum surge hydrograph estimates of the maximum stillwater level can be produced by one of the probable maximum meteorological events (PMME), such as PMH, PMWS, or moving squall line. The probable maximum surge hydrograph depicting maximum stillwater level should be derived by applying the most critically located PMME (7.2) and by use of the verified surge models discussed in 7.3.1, and 7.3.2, and 7.3.3, along with conservative estimates of other effects external to the storm system. These external effects include astronomical tide, ambient lake level, bottom friction coefficient, surface wind stress coefficients, and sea level anomaly where applicable. In addition, precipitation associated with the PMME can contribute to the rise of the stillwater level when it is assumed to occur simultaneously with the probable maximum surge.

7.3.5 Summary. Considerations of hurricanes, frontal-type (cyclonic) windstorms, moving squall lines, and surge mechanisms which are possible and applicable to the site should be discussed, including the following:

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[37] References describing the calculation of wind setup on enclosed bodies of water are listed in Appendix A24.

[38] Methods listed in references of Appendix A21 and A24 can be used to compute the surge level due to moving squall line.

(1) Ambient water level with reference to the mean low water (MLW) for coastal locations, the 100-year recurrence high water or maximum controlled level for lakes, and sea level anomaly where applicable.

(2) Determination of the controlling storm surge or seiche including the probable maximum meteorological parameters such as the storm track, wind fields, the fetch or direction of approach, bottom effects, and verification with historic events.

(3) Method used for surge or seiche level determination.

(4) Results of the computation of the probable maximum surge hydrograph in graphical presentation.

7.4 Wave Action.[39] Wind-generated activity which can occur coincidentally with a surge or seiche or independently thereof should be discussed. Wave characteristics are dependent upon windspeed, wind duration, water depth, and fetch length. The generated waves are calculated coincidental with the maximum storm surge hydrograph to determine the maximum flood elevations at the site.

7.4.1 Deepwater Wave Generation. Deepwater waves generated by a moving storm with variable windspeed along the most critical traverse line should be considered for open coastal areas. Generation of deepwater waves offshore of the continental shelf due to the approaching PMH should be determined first.[40]

Deepwater-generated waves that have traveled over the continental shelf into shallow water offshore of the site will be subjected to dissipation from friction and to possible breaking due to limited water depths. Therefore, it is possible that deepwater-generated waves will not produce controlling wave conditions at the site, and computations of shallow water wave generation independent of deepwater wave conditions will be required. If maximum breaking wave height is adopted as the design wave height, only wave period is required from the deepwater wave analysis.

Reference listed in 12.9 describes acceptable techniques in calculating the generation of deepwater waves.

7.4.2 Shallow Water Wave Generation. If the maximum breaking wave height is adopted as the design wave height, only the wave period is required from the shallow water wave analysis.

Reference listed in 12.9 describes acceptable techniques in calculating the generation of shallow water waves.

7.4.3 Controlling Offshore Incident Waves. Controlling offshore incident waves are determined by considering the storm surge stillwater hydrograph and the time-histories of transmitted deepwater waves, shallow water waves, and limiting breaking waves. Because maximum stillwater level and maximum offshore generated wave height do not necessarily occur simultaneously, various water levels should be considered in selecting the critical wave conditions. Generally, the maximum breaking wave is the most critical wave height which could occur near the site area. Because the maximum breaking wave is governed by the depth of water, assurance shall be provided against scouring or erosion of the bottom. Estimates of the final total depth and associated maximum breaker height are required if the bottom is subjected to scouring.

The maximum wave height, H_{max}, is defined in this standard as the one percent wave. For deepwater waves, it is considered that $H_{max} = 1.67\ H_s$,[41] while for shallow water waves, if it can be justified, a coefficient less than 1.67 may be used. Maximum wave heights and significant wave heights and periods should be plotted along with the maximum stillwater hydrograph to show their relationships.

7.4.4 Wave Transformation to Site. Controlling offshore incident waves must be transformed to the site. First, any effects due to shoaling, refraction, diffraction, and reflection should be considered. Then, any additional effects due to wave regeneration, wave setup, wave breaking, and wave transmission should be considered. A plot representing time-histories of significant wave heights and periods, maximum wave heights, and maximum stillwater levels should then be prepared for con-

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[39] Wave action covered in this section are those generated by wind activity only. Long period waves caused by seismic activity such as tsunami are covered by another standard. Refer to proposed American National Standard Guidelines for Determining Tsunami Criteria for Power Reactor Sites, ANS-2.4.

[40] Deepwater wave generation can also be calculated using the references in Appendix A25.

[41] H_s is significant wave height defined as the average of highest one third of representative waves.

ditions at the site. Also, a time-history of breaking wave height (maximum supportable wave heights) is plotted. The design wave height envelope is a time-history of wave heights consisting of the smaller of the breaking wave height and the incident wave height at the site. Changes in bathymetry due to wave action should be included in the analyses of wave transformation.

7.4.4.1 **Wave Shoaling and Energy Loss Due to Friction.** When a deepwater wave propagates across the continental shelf into shallow water, changes of wave height due to the combined effects of bottom geometry, bottom friction and percolation, and the continued action of the wind should be considered. Shoaling coefficients and energy loss due to friction can be obtained from reference 12.9.

7.4.4.2 **Refraction.** The change of direction of an orthogonal to the wave front as it passes over relatively simple hydrography can be approximated by Snell's law, reference 12.9.

7.4.4.3 **Diffraction.** Reference 12.9 summarizes analytical and experimental diffraction studies. Graphs presented therein provide the diffraction pattern for various semi-infinite breakwaters or breakwaters with a gap for various incident wave angles.

7.4.4.4 **Reflection.** Wave reflection results in wave amplification and resonance. The problem is more critical when the seawall, bulkheads, and jetties have vertical walls or highly reflective surfaces. Partial reflection is an important factor to consider in partially submerged structure. In many of the problems considered, only partial reflection occurs because some of the energy is absorbed by the structure, or by the beaches in case of harbors, or because a certain amount of energy is transmitted through a permeable structure.

7.4.4.5 **Breaking.** The point and height of breaking are of major importance in planning and designing shore protection measures because this condition often results in maximum wave forces, runup, and overtopping.

Among factors that determine maximum breaker height are depth of water in which the structure is sited, beach slope and bathymetry in front of the structure, including refraction effects, and variables which describe the incident waves in deep water. The phenomenon is highly complicated, and it is not yet possible to be fully described in a mathematical form. To estimate maximum wave height for runup and overtopping, etc., height should be either the maximum breaker height or the maximum wave height available after the modification by refraction, shoaling, and bottom friction of the deepwater wave. The lower value of the two may be used.

7.4.4.6 **Transmission.** Reference 12.9 discuss transmission of waves.[42]

7.4.4.7 **Regeneration.** The ratio of reformed wave height to initial wave height is a function of the energy dissipated in the breaking process. Waves that are transmitted over submerged barriers can also undergo a slight reduction in height without breaking due to energy dissipation. Regeneration of reformed wave or transmitted wave under action of storm wind can cause an increase in wave height as the wave is propagated to the site. This is especially true if the available fetch for wave generation, i.e. the distance between the breakers (or submerged barrier) and the site, is large. For waves breaking on a sloping beach, wave regeneration is generally neglected; however, for wave transmitted over a flooded beach berm, a shore barrier, etc., it must often be considered.

7.4.4.8 **Wave Setup.** Setup depends on the wave height at the point of breaking, the geometry of the beach, and the incoming wave period. In the case of simple beach geometry, formulas in reference 12.9 are available for estimating the maximum setup.

One important feature of wave setup is increase of breaking wave height as a result of increase of effective water depth (mean water depth plus wave setup). Therefore, in estimating water level at the site both wave setup and the modified breaking wave height shall be considered.

7.4.4.9 **Swells.** Free waves arriving from a distant storm (termed swell) may be of significance insofar as they occur at a site coincidentally with a local storm. As waves leave the generation area, they are subject to geometric spreading, frequency dispersion, component interactions, scattering by islands and intervening wave systems, and dissipation. The gross result is that, at a distance, the sensible wave system is characterized by both longer significant period and lower significant height than at the source area.

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[42] References in Appendix A26 also discuss transmission of waves.

Because of the effects of refraction and the tendency for swell sources to be regionally (and seasonally) localized, significant swell tends to arrive from certain preferred directions. In the absence of historical data for a specific region, it will still be possible to estimate the characteristics of swell (including seasonal dependence and frequency) by application of available methods with a knowledge of major storm activity. The average swell for the season being considered is an acceptable ambient condition.

7.4.4.10 Local Storm Effects. If the season of heavy swell does not coincide with the season of local hurricane occurrence, it may be demonstrated that their simulataneous occurrence is a highly improbable event and, therefore, need not be considered. However, if the site is subject to heavy swell and its season coincides with that of major local storms, its incremental effect should be considered. This is not straightforward since swell and local storm waves cannot be simply superimposed, due to the limitations on wave steepness imposed by breaking and the essential nonlinearity of wave behavior in the shallow water zone. An analysis including heavy swell is additionally complicated by consideration of surge and wave-induced setup. Therefore, a thorough discussion showing how these interactions are treated shall be presented. All simplifying assumptions made and degree of uncertainty involved shall be discussed.

7.4.5. Design Waves. Selection of design waves depends on the incoming wave characteristics at the site, the structures being considered, and the available water depths fronting the structures. Waves developed in section 7.4.4 shall be propagated toward the various plant structures. In selecting design waves for various structures, the possible range of wave periods, heights, and approach directions during various times of the storm shall be considered to arrive at critical conditions. In selecting the proper design wave for wave runup and wave forces against safety-related structures, the wave period spectra shall be considered because the significant wave period might not control. In calculating minimum wave periods, the equation for limiting wave steepness in shallow water is used. For maximum wave periods a value of 1.2 times the significant wave period is recommended.

The selection of the design wave for structural stability depends on whether the structure is subjected to the attack of nonbreaking, breaking, or broken waves. If the water depth does not control the wave height, then a nonbreaking wave condition would exist. In this case, the design wave is selected as follows: For rigid structures the design wave should be based generally on H_1, the average of the highest one percent of all waves (H_{max} defined in the standard); for semi-rigid structures the design wave should be selected within the range of H_s to H_1; for flexible structures, such as rubble-mound or riprap structures, the design wave should be based on the significant wave, H_s, except in unusual cases. It shall be shown, however, that in no case will failure or other hazard to safety-related facilities occur from the effects due to H_1 waves.

When the maximum breaking wave is selected as the limiting wave condition, analyses on transformation of the deepwater and shallow water waves are not required.

Where the primary direction of potential wave attack is 45 degrees or less, Mach Stem wave effects should be considered. For such effects it may be necessary to provide protection from waves up to twice the incident wave, as limited by breaking wave.

7.4.5.1 Structure Being Considered. In general, the structures that need to be considered for the wave activities are the intake and discharge structures, protective dike, waterfront bank and shore, auxiliary and control buildings, shield building, and other safety-related facilities, and non-safety-related facilities whose failure could adversely affect safety-realted facilities.

7.4.5.2 Surge Effect and Flood Elevation. The exterior of the safety-related structure shall be protected up to the maximum flood level, which is the result of the most critical combination of wind setup and corresponding wave runup and structural design of floor and wall system.

The interior surge effect due to wave action, such as inside the intake chamber which has openings directly or indirectly leading to the outside open water body, should be considered for the safety and proper function of pump, motor, and related components. The water fluctuation due to wave action on top of the stillwater level of wind setup outside the structure would result in corresponding surge effect inside the structure. The energy flux of inletting water and the maximum interior flood level are functions of the opening dimension, size of the chamber, submergence of the opening, and the design wave characteristics. Intake chambers subject to surging to floor levels must have analyses to verify adequacy of venting or structural design of floor and wall system.

7.4.5.3 Types of Waves Forces. Depending on the type of structure under consideration, the wave forces on the structure should be analyzed for nonbreaking, breaking, and broken waves. Forces due to nonbreaking waves are primarily hydrostatic. Broken and breaking waves exert an additional force due to the dynamic effects of turbulent water and the compression of entrapped air pockets. Dynamic forces may be much greater than hydrostatic forces. Therefore, the structures located where waves break are designed for greater forces than those exposed only to nonbreaking waves. See 7.4.9.2 for forces on structures.

7.4.6 Wave Erosion. The design storm events can potentially result in significant alterations of shorelines fronting the site. An analysis should be performed to determine potential profiles of critical locations of the site.

This analysis should include consideration of composition and grain size of beach materials, beach slopes and vegetative cover, and wave exposure and climatology.[43]

7.4.7 Wave Runup and Overtopping. Wave runup shall be determined. It will depend upon wave characteristics, offshore bathymetry, and structural geometry. A suggested procedure for steep structures is given in section 7.21 of reference 12.9. In case of a gentle slope between shore and stucture, runup can be estimated by a variety of empirical and semi-empirical models. It should be noted that "significant runup" may not correspond to the significant wave (similarly, maximum runup might not correspond to the maximum wave). Wave runup for the entire wave spectrum must be evaluated to identify maximum runup for inundation studies. It should also be noted that resonant wave enhancement in an embayment will strongly affect runup behavior.

If the structure characteristics are such that they are not well represented by any of the cases for which design data are available in the literature, such as given in reference 12.9, present the results of special analyses, physical model tests, or prototype tests, to support runup evaluations. If evaluations are based on physical model, or tests, discuss the scale effects and comparisons provided therefor.

If overtopping of the structure due to design waves is expected to occur, evaluate the time, rate, and quantity of water overtopping. Procedure given in reference 12.9, or similar procedure is acceptable.

7.4.7.1 Consequences of Overtopping. If any overtopping of the structure is likely to occur, discuss the proposed methods and provisions to handle overtopping water so that the safety-related structures and facilities will not be flooded. Include a discussion of windblown and overtopping water spray and its action through openings in structures of facilities that could result in flooding of safety-related structures and facilities.

7.4.8 Design Maximum Flood Elevation. The maximum flood elevations that are most critical with respect to each of the safety-related facilities or structures shall be determined. This shall include the coincidental effects of rainfall, astronomical tide, surge, and wind wave runup including wave setup. A tabulation of the maximum flood elevation and the design flood protection provided for each of the safety-related facilities shall be provided.

Discuss considerations supporting the conclusion that the flood elevations presented are the most critical ones with respect to each of the safety-related facilities. Due consideration should be given to the mutual effects of the structure configuration including the bathymetry around the site and the maximum floodwater levels.

Flood elevations associated with combined events are discussed in Section 9, Combined Events Criteria. It is possible that such elevations could be greater than the design maximum flood elevation from the probable maximum surge and seiche.

7.4.8.1 Surge Hydrograph. A discussion, including calculations of the surge hydrograph shall be provided. It shall correspond to the

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[43] Examples of wave erosion analysis are given in the references in Appendix A27.

most critical of the various alternative assumptions with respect to critical windstorm paths, translational speed of the probable maximum windstorm system, and wind field orientation. The fetch for wind-generated waves shall be provided for each of the safety-related facilities if they are located appreciable distances apart.

7.4.8.2 Routing of Surge Hydrograph. For sites located on streams, rivers, bays, and estuaries subject to surge effects, a discussion of the hydraulic computations used to route the open-coast surge hydrographs to the site shall be provided. The appropriate flow and stage conditions suggested hereinbefore shall be considered.

7.4.8.3 Structure Interiors. In the case of safety-related structures whose interiors are hydraulically connected to the exterior, the design maximum flood elevation on the interior shall also account for the surging effect due to the waves on the exterior open body of water. An example of such possible effect is the inside of the intake chamber. The magnitude of surging effect and the design maximum flood elevation shall be presented and discussed.

7.4.9 Wave Forces. The nature of waves and wave breaking mechanism for the given site conditions and for the whole range of water elevation expected shall be identified and discussed. Types of waves shall include nonbreaking, breaking, and broken. Considerations such as edge waves should be included. Tsunamis are the subject of a separate standard.[44]

7.4.9.1 Identification of Most Conservative Condition. If site topographic conditions and water level ranges are such that only one of the aforementioned types of wave action governs, identify the most conservative condition associated with that type of wave action (conservative from the standpoint of the design structure, i.e., maximum bending moment on wall or overturning moment for stability calculation on shear force or structural members and impact).

Evaluate the wave force and its point of application associated with the most conservative conditions. If more than one type of wave action stated in 7.4.9 is expected to govern the design due to unique site topography and water level range, consider and present the evaluations of all possible types of wave forces and their points of application. The most conservative type from the standpoint of the design of structure shall be used.

7.4.9.2 Forces on Structures. The wave forces on structures such as walls of buildings and rigid seawalls shall be evaluated. The following methods described in reference 12.9 are acceptable:

(1) Forces due to nonbreaking waves-Sainflou method

(2) Forces due to breaking waves -Minikin method

(3) Forces due to broken waves - Combination of hydrostatic and hydrodynamic forces.

Other equivalent and well-accepted methods may also be used.

7.4.9.3 Protective Structures. If rubble-mound structures are provided to protect safety-related structures or facilities, state their design criteria: design waves, overtopping rates, stability, acceptable damage, etc.

Describe the design features of the structure: geometry and elevations, stone sizes and weights, number of layers, and type of stone and its weathering properties. Discuss the reliability of the structure.

Provide analyses or tests in support of the capability of the structure. To evaluate the forces and the stability of the structure, use procedures given in section 7.3 of reference 12.9, or similar procedures.

7.4.9.4 Submerged Structures. In the case of a safety-related structure of facility submerged under water, describe its geometry, its orientation with respect to the direction of waves, submergence depth, etc. Cross-reference relevant discussions of safety-related facilities.

7.4.9.5 Determination of Forces. Calculate the horizontal and vertical (uplift) forces by any of the accepted methods. Cite the literature describing the methods used.

Use of reference 12.9 is suggested.[45] If physical model tests are undertaken, describe the tests, results, and conclusions. Discuss the design provisions made to handle the above calculated forces.

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[44] Refer to proposed American National Standard, Guidelines for Determining Tsunami Criteria for Power Reactor Sites, ANS-2.4.

[45] Additional references to calculate wave forces are given in Appendix A28.

If impermeable seawalls are provided to protect safety-related facilities from wave action, consider uplift force on the wall, in addition to the hydrodynamic force due to the wave action, and other forces such as earth pressures in analyzing the stability of the wall. Provide a discussion of the uplift force, including calculations in support of its magnitude and line of action.

Uplift pressures should be considered as full hydrostatic pressure for walls whose bases are below sea level or for walls with saturated backfill.

If the foundation or body of the wall is permeable, uplift pressure should be conservatively considered as the hydrostatic pressure corresponding to the maximum water elevation including the wave runup height.

7.4.10 **Summary.** Wave period, significant wave height and elevations, and maximum wave height and elevation, all coincident with the water level hydrograph, should be presented. Specific data should be presented on the largest breaking wave height, setup, runup, wave force, and the effect of any overtopping in relation to affected facilities. Water levels at affected facilities and facility protection against static and dynamic effects and splash should be discussed.

7.5 **Seiche.** Oscillations are of two types, free oscillations and forced oscillations.[46]

7.5.1 **Free Oscillations.** When a design site is located within an enclosed or semienclosed bay or harbor, free oscillations of the water body excited by barometric fluctuations, storm surges, variable winds, tsunamis, and, most familiarly, the random wave background, as well as other broadband disturbances (such as a local seismic displacement which could produce a very extreme "sloshing" of the entire basin), shall be considered. The modes of oscillation depend only upon the geometry and bathymetry of the water body. The amplitudes of oscillation depend upon the magnitude of the exciting force and can be calculated, provided that force is properly specified.

7.5.2 **Forced Oscillations.** Forced oscillations arise from the continuous application of an excitation, either over the water column at an entrance or over the fluid surface. The simplest example is that of a train of long-period waves arriving at a coastal embayment, inducing like-period oscillations. Should the frequency of the incoming waves match one of the local free-oscillation modes, resonant amplification leading to large motions may occur. Computer programs have been developed to calculate the oscillation time-history at any point within a bay of arbitrary shape, requiring as input complete specification of the geometry and the forcing wave system.

In an area for which extensive historical observations of long-period wave heights are available, such computations may be unnecessary, since design is based directly upon statistical summaries of the data.

8. Ice Flooding

8.1 **General.** Comprehensive selections of references associated with the solutions of ice problems can be found in several publications.[47] In this section, ice flooding encompasses both water elevations and forces due to ice.

8.2 **Ice Effect.** The following conditions shall be assessed to ensure that safety-related facilities are not adversely affected by the pressure of ice on streams, lakes, canals, and reservoirs:

(1) Ice cover or ice jams in streams and canals causing backwater

(2) Frazil and anchor ice affecting intake screens, racks, pump casings, valves, and control works

(3) Ice-produced forces on intake structures, racks, gates, dams, and control works

(4) Ice ridges on lakes

(5) Windrowed ice piles.

8.3 **Surface Ice.** Surface ice is the primary cause of ice jams and also of direct structural loading.

8.3.1 **Ice Formation or Ice Jams.** Regional ice and ice jam formation history shall be described. Historical maximum events, such as ice thickness, ice-growth period, resulting high level, together with the associated hydrological and physiographic characteristics shall be included. Potential for floods caused by snow or ice slides should also be discussed.

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[46] References which describe the principles and methods of oscillations are given in Appendix A29.

[47] Some references on ice flooding are given in Appendix A30.

8.3.1.1 Protection. Based on the regional history, upstream and downstream ice jam phenomena under the most severe combination of stream geometrical and hydrometeorological condition should be discussed. These combined conditions may be demonstrated analytically or experimentally, or both, to ensure protection of the safety-related facilities from ice-affected floods.

8.3.1.2 Remedial Measures. Possible remedial measures to negate the adverse effects of local ice jams on the plant shall be discussed.

8.3.2 Ice Features. Ice thickness and consequent static and dynamic loads are important to the design of the plant.

8.3.2.1 Ice Thickness. Where required to support an analysis, an estimate of the probable maximum ice thickness shall be provided by conservatively enveloping the regional observations. Information on the maximum and the average rates of ice growth and the lengths of the period of the ice growth compiled in the study shall be included. The frequency and duration of ice buildup should also be considered.

8.3.2.2 Static and Dynamic Load. Critical winter water level estimates and the bases thereof under conditions of a conservative mode of ice fracture and possible rate of ice temperature rise shall be provided. This will be used in the application of probable maximum ice thickness load for the design of safety-related structures and equipment. Discussion should include the ability of protective structures, racks, screens, and supporting members to sustain full static and dynamic forces.

8.3.3 Frazil and Anchor Ice. Regional history of formation of frazil and anchor ice in streams and canals and related problems should be described. Methods and design criteria should be described for control of frazil or anchor ice to obviate blockage of intake racks and screens, clogging of valves and pump casing, and adherence of frazil or anchor ice to gates causing operational difficulties.

9. Combined Events Criteria

9.1 General. No single flood-causing event, predictable by present technology, is adequate as a design flood base for power reactors. This section, therefore, embraces combinations of flood-causing events which, collectively, do provide adequate design flood bases.

9.1.1 Dependency. In any combination of flood-causing events, distinction between dependent and independent events is not sharp. Time sequential meteorological events, for example, are only partially and not invariably dependent and their magnitudes are only partially and not invariably dependent also. In contrast, seismic and meteorological events are clearly independent.

9.1.2 Acceptable Probability. An average annual exceedance probability less than 1×10^{-6} is an acceptable goal for selection of flood design bases for power reactor plants.

Technology is not available to assess precise numeric probabilities of all separate extreme events and their dependency. However, conservative postulations of probabilities for separate events and like postulations of their dependency in combination provide reasonable assurance of combined probabilities of less than 1×10^{-6} and hence are adequate for planning safety from flooding at power reactors.

9.1.3 Effectiveness of Individual Events. Some sequential events may decrease combined probabilities out of scale with their increase on a flood crest, and an event may have varying influence on flood level in different combinations. Saturated soil, for example, will scarcely influence a stream flood crest caused by two large, consecutive rains, but may have a marked increasing influence on a flood crest caused by a single rain. Yet the wet soil event exerts an equal probability influence in both combinations.

In postulating probabilities of combined events, this standard excludes probability influence of flood-causing events that are likely to have only a small influence on flood levels at the power reactor site.

9.1.4 Wind Influence. Windspeeds of differing probabilities are appropriate in differing combinations of flood-causing events. Applicable probabilities are for winds: (a) occurring from the most critical directions on the basis of both historical observations and meteorological reasoning, (b) of durations sufficient to generate full wave heights over effective fetch lengths, (c) which follow heavy precipitation by times about equal to that between flood-causing precipitation and flood peak at the site, and (d) which are seasonally compatible with the precipitation being applied to the watershed.

Wind probabilities meeting these needs have not been analyzed for general nationwide application. Although this kind of wind study applicable at a specific power reactor would be highly desirable, it may not be practical because of lack of data or its format and because of analysis time limitations. If such an analysis is made, the resulting two-year wind should be used where indicated in 9.2.1.1 and 9.2.1.2, which follow.

Wind probabilities in this standard are chosen based on consideration of probability of combined conditions contributing to the event being analyzed.

In lieu of generalized or specific analyses meeting the above needs, two-year annual extreme-mile wind, which recognizes geographic variation of windspeeds, may be used as a starting base. The two-year value for a specific location from Figure 1 would be adjusted from fastest-mile speeds for durations of either one hour or time appropriate to plantsite effective fetch lengths, whichever is less. These adjusted windspeeds should be applied from the most critical direction and coincident with the maximum stillwater level. Figure 1, and tables 3 and 4 for Hawaii and Puerto Rico, of reference (1) of 12.10 are reproduced on the following page for the reader's convenience. Another appropriate reference is given in (2) of 12.10.

9.1.5 Seasonal Compatibility. All events in any alternative combination should apply to a common season.

9.1.6 Storm Optimization. Rainfall events shall be critically centered over the watershed and critically distributed with time. Hurricanes and windstorms shall move at critical speed along the critical track.

9.1.7 Reservoirs. Reservoirs shall be considered to be at the upper level of the operating rule curve at the time of arrival of the flood sequence.

9.1.8 Exceptions. These criteria do not cover flooding caused in any way as a result of landslides, rockfalls, icefalls, volcanic eruptions, tsunamis, ice, glaciers, or dam failure due to nonhydrologic causes other than seismic.

9.2 Summary of Combined Event Criteria. The following combinations of flood-causing events are considered to have an exceedance probability less than 1×10^{-6} and hence are adequate design flood bases for power reactor plants.

9.2.1 Inland Streams. Design basis floods may result from extreme precipitation, sometimes with attendant dam failures, or from dam failures caused by earthquakes, structural defects, and other nonhydrologic causes. Flooding from ice has not been included below because the number of potential sites for which ice flooding could be a limiting condition are considered small.

9.2.1.1 Precipitation Floods. Any of the following alternative combinations of flood-causing events may produce the highest flood level at the plantsite. A sufficient number of the applicable alternatives should be tested or discussed to assure that the highest flood level has been determined. Different seasons for some alternatives may also need to be evaluated.

Alternative I
(1) Mean monthly (base) flow
(2) Median soil moisture
(3) Antecedent (or subsequent) rain = 40 percent of PMP
(4) PMP
(5) 2-year windspeed applied in the critical direction.

Alternative II
(1) Mean monthly (base) flow
(2) Probable maximum snowpack
(3) Coincident 100-year snow season rain
(4) 2-year windspeed applied in the critical direction.

Alternative III
(1) Mean monthly (base) flow
(2) 100-year snowpack
(3) Coincident snow season PMP
(4) 2-year windspeed applied in the critical direction.

9.2.1.2 Seismic Dam Failures. Although the principal cause of a dam failure may be from earthquake, there is a chance, however slight, that the peak of a flood could coincide with the few minutes' duration of the earthquake. The higher of the following two alternative combinations is an adequate design base for seismic dam failure floods.

Alternative I
(1) 25-year flood.
(2) Dam failure caused by safe shut-

Fig. 1

Isotach 0.50 Quantiles, In Miles Per Hour:
Annual Extreme - Mile 30 Ft. Above Ground, 2-Yr. Mean Recurrence Interval

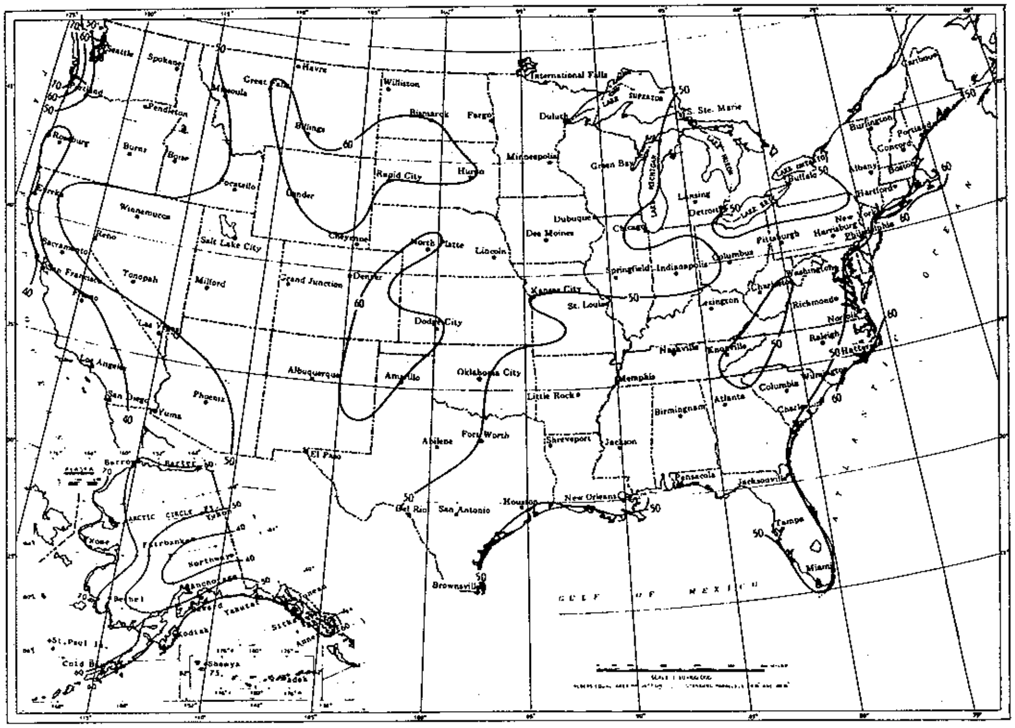

Table 3-Hawaii Fastest Mile Quantiles

1-F (R) (1)	0.50 (2) (2)	0.10 (10) (3)	0.04 (25) (4)	0.02 (50) (5)	0.01 (100) (6)
Leeward Exposure	38	51	60	67	75
Windward Exposure	42	59	70	80	91

Table 4-Puerto Rico Fastest Mile Quantiles

1-F (R) (1)	0.50 (2) (2)	0.10 (10) (3)	0.04 (25) (4)	0.02 (50) (5)	0.01 (100) (6)
Any Exposure	45	65	80	95	110

down earthquake (SSE)[48] coincident with peak of flood.

(3) 2-year windspeed applied in the critical direction.

Alternative II
(1) One-half PMF.
(2) Dam failure caused by operating basis earthquake (OBE)[48] coincident with peak of flood.
(3) 2-year windspeed applied in the critical direction.

9.2.2 Open and Semienclosed Bodies of Water. Surges, seiches, tsunamis and tides would produce maximum flood levels along the shores of open and semi-enclosed water bodies. On streams tributary to such water bodies, floods on the stream also contribute to flood levels. Tsunami determination and application are not included in these standards. When a surge event is hurricane caused, associated precipitation should be taken into account.

9.2.2.1 Shore Location. Following is an adequate design base:
(1) Probable maximum surge and seiche with wind wave activity
(2) Ten percent exceedance high tide.

9.2.2.2 Streamside Location. The following alternative combinations should be evaluated to determine the maximum controlling level:

Alternative I
(1) One-half PMF
(2) Surge and seiche from the worst regional hurricane or windstorm with wind wave activity
(3) Ten percent exceedance high tide.

Alternative II
(1) PMF
(2) 25-year surge and seiche with wind wave activity
(3) Ten percent exceedance high tide.

Alternative III
(1) 25-year flood
(2) Probable maximum surge and seiche[49] with wind wave activity
(3) Ten percent exceedance high tide.

Alternative IV (for drainage areas of less than 300 square miles in hurricane areas)
(1) PMF
(2) PMH
(3) Ten percent exceedance high tide.

9.2.3 Enclosed Bodies of Water. Surges and seiches applied to antecedent water levels determine flood levels at shore locations on enclosed bodies of water. Along streams tributary to the water body, floods in the stream also contribute to flood levels.

When a surge event is hurricane caused, associated precipitation should be taken into account.

9.2.3.1 Shore Locations. The following combination provides an adequate design base:
(1) Probable maximum surge and seiche with wind wave activity
(2) 100-year or maximum controlled level in water body, whichever is less.

9.2.3.2 Streamside Location. The following alternative combinations should be evaluated to determine maximum flood level:

Alternative I
(1) One-half PMF
(2) Surge and seiche from the worst regional hurricane or windstorm with wind wave activity
(3) 100-year or maximum controlled level of water body, whichever is less.

Alternative II
(1) PMF
(2) 25-year surge and sieche with wind wave activity
(3) 100-year or maximum controlled level of water body, whichever is less.

Alternative III
(1) 25-year flood
(2) Probable maximum surge and seiche with wind wave activity

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[48] Title 10, Code of Federal Regulations, Part 100, "Reactor Site Criteria," Appendix A, "Seismic and Geologic Siting Criteria for Nuclear Power Plants," Government Printing Office, Washington, D.C.

[49] The surge and seiche resulting from a probable maximum windstorm, a probable maximum hurricane, or a moving squall line.

(3) 100-year or maximum controlled level of water body, whichever is less.

9.2.4 Non-Hydrologic and Non-Seismic Dam Failures. No specific guidance or specific event combinations are provided in this standard because of uncertainty in postulating a realistic dam failure from non-hydrologic and non-seismic causes. Refer to 6.3.

10. Cooling Water Canals and Reservoirs

10.1 General. This section provides guidance on the design of safety-related canals, reservoirs and related structures. If these are part of the ultimate heat sink, further guidance is available.[50]

10.2 Canals. Purpose of the canals, whether safety-related or not, and the design capacity with appropriate elevations, shall be stated.

10.2.1 Maps. Maps and cross sections should be provided to define the location, elevations, shape, size, and slope of canals. Drainage areas and cross-drainage structure locations shall be defined.

10.2.2 Appurtenant Structures. Drawings of structures appurtenant to canals, such as intake and discharge structures, energy dissipators, gates, valves, lining, riprap, and cross-drainage structures shall be provided.

10.2.3 Design Criteria. Design criteria,[51] including hydraulic, hydrologic, soils, and seismic considerations shall be discussed, including wind wave criteria and resultant freeboard requirement above stillwater levels. If safety-related, the two-year windspeed criteria of Section 9, Combined Events Criteria, shall be used. During times of normal water levels, the probable maximum gradient wind shall be used. Freeboard shall not be less than wind setup plus wave runup.

10.2.4 Failure. Cause, mode, consequence, and probability of canal failure shall be discussed.

10.2.5 Sedimentation. Provision for preventing sediment buildup in the canal shall be discussed.

10.3 Reservoirs.[52] Purpose of the reservoirs, whether safety-related or not, and the design capacity with appropriate elevations shall be stated. Guidance is available if the ultimate heat sink is a reservoir.[50]

10.3.1 Purpose. Purpose of the reservoirs, whether safety-related or not, basis for the design capacity, and proposed mode of operation shall be described.

10.3.2 Location. A topographic map showing location of the reservoir with relation to other plant elements and the contributing drainage area shall be provided.

10.3.3 Capacity Curve. Elevation-area-capacity curves extended to the PMF level (or to top of dam if the design flood is less than the PMF) shall be provided.

10.3.4 Drawings. Engineering design drawings and descriptions of the dam and discharge and withdrawal facilities, including low-level outlet, spillway, gates, valves, intakes, pump structures, discharge chute, energy dissipators, and erosion protection shall be provided.

10.3.5 Probable Maximum Flood. If not already contained in Section 5, Probable Maximum Flood From Precipitation, derivation of the probable maximum flood (PMF) shall be described including the PMP, runoff model and its verification, losses, snowmelt, and antecedent or subsequent rains.

10.3.5.1 Flood Routing. Inflow, outflow, and reservoir level hydrographs shall be provided and routing techniques should be described.

10.3.5.2 Maximum Water Level and Velocity. Maximum water levels and velocities shall be discussed in reference to safety-related facilities.

10.3.6 Non-Safety-Related Reservoir. If a reservoir is not safety-related, its spillway design flood, if less than the PMF, shall be described together with the effect of the PMF on the dam and reservoir.

10.3.7 Wind Wave Criteria and Freeboard. Procedures and criteria used to determine wind wave effects and freeboard above stillwater levels shall be described. If a reservoir is safety-related, the two-year wind-speed criteria of Section 9, Combined Events Criteria, shall be used. During times of normal water levels, the probable maximum gradient wind shall be used. Freeboard shall not be less than wind setup plus wave runup.

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[50] NRC Regulatory Guide 1.27 "Ultimate Heat Sink for Nuclear Power Plants," Revision 1, March 1974.

[51] Some useful references on canal design are given in Appendix A31.

[52] Some references to determine design bases for dams and spillways are given in Appendix A32.

10.3.8 Sedimentation.[53] Sources and estimated amount of sediment, reserved sediment space, and provision for removing sediment or preventing it from reaching the reservoir should be discussed. Where applicable, measures to preclude blockage of intakes, including frequency and methods of removing sediment, shall be discussed.

11. Plant Site Drainage

11.1 General. The effects of local probable maximum precipitation on the plant site drainage shall be determined and summarized.

11.2 Factors for Consideration. Potential flooding of safety-related facilities due to one or more of the following factors should be considered:

(1) Sheet flow over the areas immediately adjacent to safety-related facilities, including roof drainage

(2) Side-hill drainage running toward the plantsite

(3) Temporary ponding in the plant area because of site topography, site grading or installed hydraulic controls, and

(4) Natural watercourses or man-made drainage channels that run through or adjacent to the site, the drainage area of which may also include areas outside of the plant boundary. (If already considered in 4.2.1 and 5.4, refer thereto.)

11.3 Site Drainage Area. Drainage area contributing to floods on the site shall be delineated on a topographic map of sufficient scale and quality to allow an independent evaluation of pre- and post-construction drainage patterns.

11.3.1 Site Characteristics. Site characteristics such as runoff factors and losses shall be determined as in 5.4. Assumed values shall be based on regional characteristics, but changes in site characteristics due to project development shall be reflected in the assumption. Losses are highly sensitive to site development. When in doubt assume little, if any, loss.

11.3.2 Drainage System. Sufficient details of the site drainage system should be provided (1) to allow an independent review of rainfall and runoff effects on safety-related facilities, and (2) to judge the adequacy of design criteria.

11.3.3 Drainage Plans. Drainage plans must show existing and proposed contours of the area(s) to be graded. The plan shall also include practices for erosion control, slope stabilization, safe disposal of runoff water and drainage, such as waterways, lined ditches, reverse slope benches (include grade and cross section) grade stabilization structures, retaining walls, and surface and subsurface drains.

11.3.4 Local Criteria. Hydraulic and hydrologic design criteria of drainage systems developed for local jurisdiction should be discussed.

11.4 Roof Drainage. Roof drainage contributions to the surface runoff during the PMP event shall be evaluated to determine the worst case for site surface drainage effects.

11.5 Grades and Drains. The effects of surface grading, drainage facilities, roads, railroad tracks, and terracing shall be evaluated for local PMP to determine potential flooding of safety-related facilities or preventing access thereto.

11.6 Local Intense Precipitation. Estimates of local probable maximum precipitation are presented in the references of 12.1 For California, estimates are presented in reference (1) of 12.1.

11.6.1 Short Duration Rainfall. For drainage areas where the time of concentration is small, estimates of short duration rainfall given in the references of 12.11 may be useful.

11.6.2 Point Rainfall. In areas where the basic data use point rainfall as average 10-square mile rainfall, any increase in intensities for areas less than the 10-square miles should be documented.

11.6.3 Rainfall Intensity-Duration-Frequency. Rainfall intensities for the selected and critically arranged time increments should be presented. References in 12.11 may be useful.

11.6.4 Time Distribution. Rainfall increments should be distributed critically assuming coincidental occurence of all duration values.

11.7 Model. Runoff and streamcourse models for the site may be derived in accordance with the guideline of 5.4.

11.7.1 Overland Flow. For strictly overland flow or "sheet flow," the runoff model may use flow formulas from standard hydraulic textbooks.[54]

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[53] References which discuss sedimentation are given in Appendix A33.

[54] References describing overland flow are given in Appendix A34.

11.7.2 Hydraulic Solutions. The applicant shall site references, treatises, model study reports, or prototype tests used in the solution of hydraulic or hydrologic design problems.

11.8 Snow and Ice Accumulation. Provide a discussion of the effects of snow and ice accumulation on site facilities where such accumulation of melting conditions could coincide with local probable maximum (winter) precipitation and cause flooding or other damage to safety-related facilities. If already discussed under Section 8, Ice Flooding, refer thereto.

11.9 Summary. Provision for safe site drainage should be summarized in brief form for each examination by providing the following:

(1) Map of site area showing the drainage facilities, direction of runoff, and outfall of site drainage water. Provide the site watershed areas in acres or square miles.

(2) Description of the runoff and stream-course models in generic terms including appropriate coefficients.

(3) PMP amounts, any appropriate snow, losses, and precipitation excess used to estimate site drainage needs.

(4) Site drainage flow and volume, and capacity of drains, channels, and outlets.

(5) Discussion of designed characteristics and maintenance procedures to ensure effectiveness of all drainage facilities.

(6) Identification of access routes to safety-related structures with pertinent flood elevations.

(7) Controlling site drainage flood elevations.

12. References

[1] American National Standard Design Basis for Protection of Light Water Nuclear Power Plants Against Effects of Postulated Pipe Rupture, ANSI/ANS-58.2-1980. American Nuclear Society, La Grange Park, Ill.

12.1 Probable Maximum Precipitation. Specific probable maximum precipitation estimates for the United States are as follows:

(1) U.S. Department of Commerce, "Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages," Hydrometeorological Report No. 49, NOAA, National Weather Service, Silver Spring, Maryland, September, 1977.

(2) U.S. Department of Commerce, "Probable Maximum Precipitation Estimates, United States East of the 105th Meridian Hydrometeorological Report No. 51, NOAA, National Weather Service, Washington, D.C., June, 1978.

(3) U.S. Department of Commerce, "Probable Maximum Precipitation and Snowmelt Criteria for Red River of the North Above Pembina, and Souris River Above Minot, North Dakota," Hydrometeorological Report No. 48, NOAA, National Weather Service, Washington, D.C., May 1973.

(4) U.S. Weather Bureau, "Generalized Estimates of Probable Maximum Precipitation and Rainfall-Frequency Data for Puerto Rico and Virgin Islands," Technical Paper No. 42, Washington, D.C., 1961.

(5) U.S. Weather Bureau, "Generalized Estimates of Probable Maximum Precipitation West of the 105th Meridian," Technical Paper No. 38, Washington, D.C., 1960.
Note: This report shall be used only for areas not covered by references (6) and (11) and only for drainage areas larger than 200 square miles in the area covered by reference (1).

(6) U.S. Weather Bureau, "Interim Report-Probable Maximum Precipitation in California," Hydrometeorological Report No. 36, Washington, D.C., October, 1961 with revisions of October, 1969.

(7) U.S. Weather Bureau, "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian," Hydrometeorological Report No. 33, Washington, D.C., 1956.

(8) U.S. Weather Bureau, "Meteorological Criteria for Extreme Floods for Four Basins in the Tennessee and Cumberland River Watersheds," Hydrometeorological Report No. 47, Silver Spring, Maryland, May, 1973.

(9) U.S. Weather Bureau, "Probable Maximum and TVA Precipitation for Tennessee River Basins up to 3,000 Square Miles in Area and Durations to 72 Hours, "Hydrometeorological Report No. 45, Silver Spring, Maryland, May, 1969.

(10) U.S. Weather Bureau, "Probable Maximum Precipitation in the Hawaiian Islands," Hydrometeorological Report No. 39, Washington, D.C., 1963.

(11) U.S. Weather Bureau, "Probable Maximum Precipitation, Northwest States," Hydrometeorological Report No. 43, Washington, D.C., November, 1966.

(12) U.S. Weather Bureau, "Probable Maxi-

mum Precipitation Over South Platte River, Colorado, and Minnesota River, Minnesota," Hydrometeorological Report No. 44, Washington, D.C., 1969.

(13) U.S. Weather Bureau, "Probable Maximum and TVA Precipitation Over the Tennessee River Basin Above Chattanooga," Hydrometeorological Report No. 41, Washington, D.C., 1965.

(14) U.S. Weather Bureau, "Probable Maximum Precipitation Rainfall-Frequency Data for Alaska," Technical Paper No. 47, Washington, D.C., 1963.

(15) U.S. Weather Bureau, "Probable Maximum Precipitation, Susquehanna River Drainage Above Harrisburg, Pennsylvania," Hydrometeorological Report No. 40, Washington, D.C., 1965.

(16) U.S. Department of Commerce, "Seasonal Variation of 10-Square-Mile Probable Maximum Precipitation Estimates, United States East of the 105th Meridian," Hydrometeorological Report No. 53, NOAA, National Weather Service, Silver Spring, Maryland, April 1980.

12.2 Time Distribution. Typical time distributions for various regions are found in the following:

(1) U.S. Army Corps of Engineers, "Standard Project Flood Determinations," EM 1110-2-1411, revised March 1965, Washington, D.C., Plate 11.

(2) U.S. Department of Commerce, "Time Distribution of Precipitation in 4- to 10-Day Storms - Arkansas-Canadian River Basins," NOAA Technical Memorandum NWS HYDRO-15, National Weather Service, Silver Spring, Maryland, June, 1973.

12.3 Sequential Storms. Specific regional studies of time sequencing of storms are as follows:

(1) U.S. Weather Bureau, "Meteorological Criteria for Extreme Floods for Four Basins in the Tennessee and Cumberland River Watersheds," Hydrometeorological Report No. 47, Silver Spring, Maryland, May, 1973.

(2) U.S. Weather Bureau, "Meteorology of Flood-Producing Storms in the Ohio River Basin," Hydrometeorological Report No. 38, Washington, D.C., 1961.

(3) U.S. Weather Bureau, "Meteorology of Hypothetical Flood Sequences in the Mississippi River Basin," Hydrometeorological Report No. 35, Washington, D.C., 1959.

(4) U.S. Department of Commerce, "Time Distribution of Precipitation in 4- to 10-Day Storms—Ohio River Basin," NOAA Technical Memorandum NWS HYDRO-13, National Weather Service, Silver Spring, Maryland, July, 1972.

(5) U.S. Weather Bureau, "Interim Report -Probable Maximum Precipitation in California," Hydrometeorological Report No. 36, Washington, D.C., October, 1961 with revision of October, 1969.

(6) U.S. Weather Bureau, "Probable Maximum Precipitation, Northwest States," Hydrometeorological Report No. 43, Washington, D.C., November, 1966.

(7) U.S. Weather Bureau, "Rainfall Frequency Atlas of the United States for Durations From 30 Minutes to 24 Hours and Return Periods From 1 to 100 Years," Weather Bureau Technical Paper No. 40, Washington, D.C., 1964.

(8) U.S. Weather Bureau, "Probable Maximum and TVA Precipitation Over the Tennessee River Basin Above Chattanooga,"Hydrometeorological Report No. 41, Washington, D.C., 1965.

(9) U.S. Weather Bureau, "Probable Maximum and TVA Precipitation for Tennessee River Basins up to 3,000 Square Miles in Area and Durations to 72 Hours," Hydrometeorological Report No. 45, Silver Spring, Maryland, May, 1969.

12.4 Regional Time Distribution. Typical regional time distribution are given in the following reports:

(1) U.S. Army Corps of Engineers, "Generalized Standard Project Rain-Flood Criteria, Southern California Coastal Streams," The Hydrologic Engineering Center, Sacramento, California, March, 1967.

(2) U.S. Army Corps of Engineers, "Standard Project Flood Determinations," EM 1110-2-1411, Washington, D.C., Revised March, 1965.

(3) U.S. Army Corps of Engineers, "Standard Project Rain-Flood Criteria, Sacramento-San Joaquin Valley, California," U.S. Engineer District, Sacramento, California, April, 1957, Revised 1958.

12.5 Coincident Storm. The following may be used to determine storms coincident with the snowmelt:

(1) MILLER, JOHN F., "Probable Maximum Precipitation and Rainfall-Frequency Data for Alaska," Technical Paper No. 47, U.S. Weather Bureau, Washington, D.C., 1963.

(2) National Weather Service, *Precipitation Frequency Atlas of Western United States,* NOAA Atlas 2, Volume 1, Montana; Volume II, Wyoming; Volume III, Colorado; Volume IV, New Mexico; Volume V, Idaho; Volume VI, Utah; Volume VII, Nevada; Volume VIII, Arizona; Volume IX, Washington; Volume X, Oregon; and Volume XI, California; Washington, D.C., 1974.

(3) U.S. Weather Bureau, "Rainfall Frequency Atlas of the United States for Durations From 30 Minutes to 24 Hours and Return Periods From 1 to 100 Years," Weather Bureau Technical Paper No. 40, Washington, 1961.

(4) U.S. Weather Bureau, "Two- to Ten-Day Precipitation for Return Periods of 2 to 100 years in Alaska," Weather Bureau Technical Paper No. 52, Washington, D.C., 1965.

(5) U.S. Weather Bureau, "Two- to Ten-Day Precipitation for Return Periods of 2 to 100 years in the Contiguous United States," U.S. Weather Bureau Technical Paper No. 49, Washington, D.C., 1964. (No longer valid for Western United States. An approximation can be obtained by use of values from NOAA Atlas 2 and the relation of figure 6 of Technical Paper No. 49 to obtain 2-year 10-day values. The 100-year 10-day values are obtained by use of a 100-year to 2-year ratio developed by use of figures 30 and 35. Intermediate durations can then be obtained by use of figure 3.)

12.6 Snowpack and Snowmelt. The following materials are useful in determining snowpack and snowmelt:

(1) U.S. Department of Commerce, "Meteorological Conditions for the Probable Maximum Flood on the Yukon River Above Rampart, Alaska," Hydrometeorological Report No. 42, U.S. Weather Bureau, Environmental Science Services Administration, Washington, D.C., May, 1966.

12.7 Probable Maximum Hurricane. Literature to assist in this analysis is as follows:

(1) U.S. Department of Commerce, "Meteorological Criteria for SPH and PMH Wind Field, Gulf and East Coast of the United States," NOAA Technical Report NWS-33, National Weather Service, Washington, D.C., September 1979.

(2) U.S. Weather Bureau, "Meteorological Characteristics of the Probable Maximum Hurricane, Atlantic and Gulf Coast of the United States," Interim Report, HUR 7-97, Washington, D.C., May, 1968. (Also see HUR 7-97A, December, 1968.)

12.11 Short Duration Rainfall. The following publications may be useful to determine short duration rainfall:

(1) National Weather Service, "5-to-60-Minute Precipitation Frequency for the Eastern and Central United States," NOAA Technical Memorandum NWS HYDRO-35, Silver Spring, Maryland, June 1977.

(2) National Weather Service, *Precipitation Frequency Atlas of Western United States,* NOAA Atlas 2, Volume 1, Montana; Volume II, Wyoming; Volume III, Colorado; Volume IV, New Mexico; Volume V, Idaho; Volume VI, Utah; Volume VII, Nevada; Volume VIII, Arizona; Volume IX, Washington; Volume X, Oregon; and Volume XI, California; Washington, D.C., 1974.

(3) U.S. Army Corps of Engineers, "Standard Project Rain-Flood Criteria, Sacramento-San Joaquin Valley, California," U.S. Engineer District, Sacramento, Caifornia, April, 1957, Revised 1958.

(4) U.S. Weather Bureau, "Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years," Weather Bureau Technical Paper No. 40, Washington, D.C., 1964. (Does not apply to areas covered by reference 2.)

(5) U.S. Weather Bureau, "Thunderstorm Rainfall," Hydrometeorological Report No. 5, Washington, D.C., 1947.

12.8 Open Coast Surges. The following may be used to calculate open coast PMH surges.

(1) BODINE, B. R., "Storm Surge on the Open Coast: Fundamentals and Simplified Prediction," Technical Memorandum No. 35, U.S. Army Corps of Engineers, Coastal Engineering Research Center, Fort Belvoir, Virginia, May, 1971. Limitations are discussed in (1a).

(1a) FSAR, Docket No. 50-302, Attachment of Letter "Regulatory Position on Crystal River Nuclear Station" from A. Schwencer of USAEC to J. T. Rogers of Florida Power Corporation, October 12, 1973.

(2) U.S. Department of Commerce, NOAA, Tide Tables.

12.9 Waves. The following should be used for analyses of water waves.

(1) U.S. Army Corps of Engineers, "Shore Protection Manual", Coastal Engineering Research Center, Fort Belvoir, Virginia, 1977.

12.10 Wind. Extreme wind for combined events may use the following references:

(1) THOM, H. C. S., *New Distribution of Extreme Winds in the United States,* Journal of the Structural Division, American Society of Civil Engineers, ST7, July, 1968.

(2) U.S. Army Corps of Engineers, "Wave Runup and Wind Setup on Reservoir Embankments," ETL 1110-2-221, Office of the Chief of Engineers, Washington, D.C., November 29, 1976.

Appendix

(This Appendix is not a part of American National Standard for Determining Design Basis Flooding at Power Reactor Sites, ANSI/ANS-2.8-1981, but is included for information purposes only.)

Bibliography

A1. Lesser Floods. The following may be used to derive lesser floods, for 4.4:

(1) Beard, L.R., "Statistical Methods in Hydrology," U.S. Army Engineer District, Corps of Engineers, Sacramento, California, January, 1962.

(2) Dalrymple, T., "Flood Frequency Analyses," Manual of Hydrology: Part 3, Flood Flow Techniques, U.S. Geological Survey Water Supply 1543-A, Washington, D.C., 1960.

(3) U.S. Geological Survey, "Magnitude and Frequency of Floods," - Reports on flood frequency and magnitude in major river basins in the conterminous United States are published in Water-Supply Papers. Volumes have the general title "Magnitude and Frequency of Floods in the United States," subtitled with the name of the basin.

(4) U.S. Water Resources Council, "Guideline for Determining Flood Flow Frequency," Bulletin No. 17A of the Hydrology Committee, Washington, D.C., June, 1977.

A2. Maximization Procedures. Following references contain guidance for transposition and maximization:

(1) Meyers, V. A., "Criteria and Limitations for the Transposition of Large Storms Over Various Size Watershed," Symposium on Consideration of Some Aspects of Storms and Floods in Water Planning, Texas Water Development Board Report No. 33, November, 1966.

(2) World Meteorological Organization, "Manual for Estimation of Probable Maximum Precipitation," WMO Operational Hydrology Report No. 1, Geneva, 1973.

A3. Depth-Area-Duration. Determining depth-area-duration curves is described in the following references:

(1) Linsley, R. K., Kohler, M., and Paulhus, J. L. H., *Applied Hydrology.* McGraw-Hill Book Company, Inc., New York, 1949.

(2) World Meteorological Organization, "Manual for Depth-Area-Duration Analysis of Storm Precipitation," WMO-No. 237, TP 129, Geneva, 1969.

A4. Time Distribution. The following may also be used in time distribution of storms for various regions:

(1) U.S. Department of Commerce, "Five-to-Sixty Minute Precipitation Frequency for the Eastern and Central United States," NOAA Technical Memorandum NWS HYDRO-35, National Weather Service, Silver Spring, Maryland, June, 1977.

(2) U.S. Department of Commerce, "Precipitation Frequency Atlas of Western United States," NOAA Atlas 2, Volume 1, Montana; Volume II, Wyoming; Volume III, Colorado; Volume IV, New Mexico; Volume V, Idaho; Volume VI, Utah; Volume VII, Nevada; Volume VIII, Arizona; Volume IX, Washington; Volume X, Oregon; Volume XI California; NOAA, National Weather Service, Washington, D.C., 1974.

A5. Precipitation Losses and Excess. Estimates of losses and excess may use the following:

(1) Musgrave, G. W. and Holtan, H. N. "Infiltration," Section 12, *Handbook of Applied Hydrology,* McGraw-Hill Book Company, Inc., New York. Ven Te Chow, Editor, 1964, pp. 12-1 to 12-30.

(2) Soil Conservation Service, Section 4, "Hydrology," National Engineering Handbook, U.S. Department of Agriculture, Washington, D.C., August, 1972.

(3) U.S. Army Corps of Engineers, "Unit Hydrograph and Loss Rate Optimization," Computer Program 23-J2-L211. The Hydrologic Engineering Center, Davis, California, August, 1966.

(4) U.S. Army Corps of Engineers, "Flood Hydrograph Package," Computer Program 723-X6-L2010, HEC-1. The Hydrologic Engineering Center, Davis, California, August, 1974.

(5) U.S. Army Corps of Engineers, "Flood-Hydrograph Analyses and Computation," EM 1110-2-1405, Washington, D.C. August 31, 1959, pp. 6-7.

A6. Snowpack and Snowmelt. The following materials are useful in determining snowpack and snowmelt:

(1) Bureau of Reclamation, "Effect of Snow Compaction From Rain on Snow," Engineering Monograph No. 35, 1966.

(2) U.S. Army Corps of Engineers, "Generalized Snowmelt Runoff Frequencies," Technical Bulletin No. 8, Sacramento District, September, 1962.

(3) U.S. Army Corps of Engineers, "Runoff from Snowmelt," Engineering and Design Manual, EM 1110-2-1406. Washington, D.C., 1960.

(4) U.S. Army Corps of Engineers, "Snow Hydrology," Summary Report of Snow Investigations, North Pacific Division. Portland, Oregon, June, 1956.

(5) U.S. Department of Commerce, "Snow Hydrology," PB 151660, undated.

(6) World Meteorological Organization, "Estimation of Maximum Floods," Technical Note No. 98 WMO - No. 233, TP. 126, Geneva, 1972, pp. 117-135.

A7. Snowmelt Models. Computer programs to model snowmelt are as follows:

(1) Anderson, E. A., and Crawford, N. H., "The Synthesis of Continuous Snowmelt Runoff Hydrographs on a Digital Computer." Technical Report No. 36, Department of Civil Engineering, Stanford University, Stanford, California, June 1964.

(2) National Weather Service, "National Weather Service River Forecast System—Snow Accumulation and Ablation Model," NOAA Technical Memorandum NWS HYDRO-17. Silver Spring, Maryland, November, 1973.

(3) Rockwood, D. M., "Streamflow Synthesis and Reservoir Regulation," Engineering Studies Project 171, Technical Bulletin No. 22, U.S. Army Division, North Pacific Division, Portland, Oregon, January, 1964.

(4) U.S. Army Corps of Engineers, "Basin Rainfall and Snowmelt Computation," Computer Program No. 723-G1-L2260. The Hydrologic Engineering Center, Davis, California, July, 1966.

A8. Unit Hydrographs. Suitable techniques for deriving and applying unit hydrographs are described in the following references:

(1) Barnes, B. S., "Unitgraph Procedures," Bureau of Reclamation, U.S. Department of the Interior, Denver, Colorado, November, 1952.

(2) Bernard, M. M., "An Approach to Determinate Stream Flow," Proceedings, American Society of Civil Engineers, Volume 60, January, 1934, pp. 3-18.

(3) Chow, V. T., *Handbook of Applied Hydrology,* Chapter 14. McGraw Hill Book Company, Inc., New York, 1964.

(4) Clark, C. O., "Storage and the Unit Hydrograph, " Transactions, American Society of Civil Engineers, Volume 110, Paper No. 2261, 1945, pp. 1419-1488.

(5) Gray, D. M., "Synthetic Unit Hydrographs for Small Watersheds," Proceedings, American Society of Civil Engineers, Paper No. 2854, Volume 87, No HY4, July, 1961, pp. 34-54.

(6) Linsley, R. K., Kohler, M. A., Paulhus, J. L. H., *Hydrology for Engineers.* McGraw-Hill Book Company, New York, Second Edition, 1975.

(7) Newton, D. W. and Vinyard, J. W., "Computer Determined Unit Hydrograph From Floods," Journal of the Hydraulics Division, American Society of Civil Engineers, HY5, September, 1967, pp. 219-235.

(8) Snyder, F.F., "Synthetic Unit-Graphs," Transactions, American Geophysical Union, August, 1938, pp. 447-454.

(9) Soil Conservation Service, "Hydrology," National Engineering Handbook, Section 4, U.S. Department of Agriculture, August, 1972.

(10) U.S. Army Corps of Engineers, "Flood-Hydrograph Analyses and Computations, EM 1110-2-1405, Washington, 1959,

(11) U.S. Army Corps of Engineers, "Hydrograph Analysis," Hydrologic Engineering Methods for Water Resources Development, Volume 4, The Hydrologic Engineering Center, Davis, California, October, 1973.

A9. Other Runoff Models. Other methods of modeling the rainfall-runoff phenomena are presented in the following references:

(1) Crawford, N. H., and Linsley, R. K., "Digital Simulation in Hydrology: Stanford Watershed Model IV," Technical Report No. 39, Department of Civil Engineering, Standford University, Stanford, California, July, 1966.

(2) Eagleson, D. S., *Dynamic Hydrology,* McGraw-Hill Book Company, Inc., New York, 1970, Chapter 15.

(3) Hydrocomp Simulation Program, Hydrocomp, Inc., Mountain View, California.

(4) Mitchell, W. D., "Model Hydrographs," Water Supply Paper 2005, U.S. Geological Survey, 1972.

(5) Rockwood, D. M., "Streamflow Synthesis and Reservoir Regulation," Engineering Studies Project 171, Technical Bulletin No. 22, U.S. Army Division, North Pacific Division, Portland, Oregon, January, 1964, pp. 1-15.

A10. Transient Flow. References which describe transient flow techniques include:

(1) Amein, N. and Fang, C. S., "Implicit Flood Routing in Natural Channels," Journal of the Hydraulics Division, American Society of Civil Engineers, Volume 96, HY12, Proceeding Paper 7773, December, 1970, pp. 2481-2500.

(2) Baltzer, R. A. and Lai, C., "Computer Simulation of Unsteady Flow in Waterways," Journal of the Hydraulics Division, American Society of Civil Engineers, Volume 94, No. HY4, Proceeding Paper 6048, July, 1968, pp. 1083-1117.

(3) Chow, V. T., *Open Channel Hydraulics,* McGraw-Hill Book Company, Inc., New York, 1959, Chapter 20.

(4) Fread, D. L., "DWOPER: NWS Operational Dynamic Wave Model," Hydrologic Research Laboratory, National Weather Service, NOAA, Silver Spring, Maryland, 1980.

(5) Garrison, J. M., Granju, J. P., and Price, J. T., "Unsteady Flow Simulation in Rivers and Reservoirs," Journal of the Hydraulics Division, American Society of Civil Engineers, Volume 95, No. HY5, September, 1969, pp. 1559-1576.

(6) Gilcrest, B. R., "Flood Routing," Chapter X, *Engineering Hydraulics,* Hunter Rouse, Editor, John Wiley, Inc., New York, 1950.

(7) Henderson, F. M., *Open Channel Flow,* McMillan Company, New York, 1966, Chapters 8 and 9.

(8) Streeter, V. L., and Wylie, E. B., *Hydraulic Transients,* McGraw-Hill Book Company, Inc., New York, 1967, Chapter 5.
(9) Strelkoff, T., "Numerical Solution of Saint-Venant Equation," Journal of the Hydraulics Division, American Society of Civil Engineers, Volume 96, No. HY1, Proceeding Paper 7043, January 1970, pp. 223-252.
(10) U.S. Army Corps of Engineers, "Gradually Varied Unsteady Flow Profiles," Computer Program 723-G2-L7450, The Hydrologic Engineering Center, Davis, California, June, 1976.

A11. Storage Routing. Following references describe the storage routing method:

(1) Styner, W., and Mockus, V., "Flood Routing," Chapter 17 of Soil Conservation Service National Engineering Handbook, Section 4, Hydrology, U.S. Department of Agriculture, August, 1972.
(2) U.S. Army Corps of Engineers, "Routing of Floods Through River Channels," EM 1110-2-1408, Office of the Chief of Engineers, Washington, D.C., March 1, 1969.

A12. Flood Elevations. Water surface profiles can be determined with the following references:

(1) Chow, V. T., *Open Channel Hydraulics,* McGraw-Hill Book Company, Inc., New York, 1959.
(2) U.S. Army Corps of Engineers, "Backwater Curves in River Channels," EM 1110-2-1409, Office of the Chief of Engineers, Washington, D.C., December 7, 1959.
(3) U.S. Army Corps of Engineers, "Gradually Varied Unsteady Flow Profiles," Computer Program 723-G2-L7450, The Hydrologic Engineering Center, Davis, California, June, 1976.
(4) U.S. Army Corps of Engineers, "Water Surface Profiles," Computer Program 723-X6-L202A, HEC-2, The Hydrologic Engineering Center, Davis, California.

A13. Combined Models. Computer programs which combine runoff and streamcourse models are as follows:

(1) Crawford, N. H. and Linsley, R. K., "Digital Simulation in Hydrology: Stanford Watershed Model IV," Technical Report No. 39, Department of Civil Engineering, Stanford University, Stanford, California, July, 1966.
(2) Hailey, B. M., Perkins, F. E., and Eagleson, P. S., "A Modular Distributed Model of Catchment Dynamics," Ralph M. Parsons Laboratory for Water Resources and Hydrodynamics, Report 133, Massachusetts Institute of Technology, Department of Civil Engineering, Cambridge, Massachusetts, 1970.
(3) Hydrocomp Simulation Program, Hydrocomp Inc., Mountain View, California.
(4) National Weather Service, "National Weather Service River Forecast System Procedures," NOAA Technical Memorandum NWS HYDRO-14, Silver Spring, Maryland, December, 1972.
(5) Rockwood, D. M., "Streamflow Synthesis and Reservoir Regulation," Engineering Studies Project 171, Technical Bulleting No. 22, U.S. Army Engineer Division, North Pacific Division, Portland Oregon, January, 1964.
(6) U.S. Army Corps of Engineers, "Flood Hydrograph Package," Computer Program 723-X6-L2010, HEC-1, The Hydrologic Engineering Center, Davis, California, August, 1974.

A14. Erosional Failure. Some references which may prove useful in developing an erosional failure model are as follows:

(1) Cristofano, E. A., "Method of Computing Erosion Rate for Failure of Earthfill Dams," U.S. Bureau of Reclamation, Engineering and Research Center, Denver, Colorado, June, 1973.

(2) Fread, D. L., "DAMBRK: The NWS Dam-Break Flood Forecasting Model," Hydrologic Research Laboratory, Office of Hydrology, National Weather Service, NOAA, Silver Spring, Maryland, 1980.

(3) Grzywienski, A., "Failure of Conventional Dams by Overtopping," The Institution of Civil Engineers, Proc. Paper No. 7313, Volume 48, pp. 35-50, January, 1971.

(4) Harris, G. W. and Wagner, D. A., "Outflow From Breached Earth Dams," Thesis to the Faculty of the University of Utah, Utah, June, 1967.

(5) Newton, D. W. and Cripe, M. W., "Flood Studies for Safety of TVA Nuclear Plants, Hydrologic Embankment Breaching Analysis," Tennessee Valley Authority, Knoxville, Tennessee, January, 1973.

(6) Soil Conservation Service, "Guidelines for Structure Classification, Procedure for Determining Downstream Effects Following Breaching of an Earth Dam," EWP Technical Guide No. 24, SCS, South Regional Technical Service Center, Engineering and Watershed Planning Unit, Fort Worth, Texas, December 8, 1969.

(7) Tennessee Valley Authority, "Computer Program for Dam Breaching," TVA, Knoxville, Tennessee, November, 1973.

A15. Failure Outflow Hydrograph. Methods which may prove useful in determining outflow hydrograph are presented in the following references:

(1) Dressler, R. F., "Hydraulic Resistance Effect Upon the Dam-Break Functions," Journal of Research of the National Bureau of Standards, Volume 49, No. 3, Research Paper 2356, September, 1952, pp. 217-225.

(2) Fread, D. L., "DAMBRK: The NWS Dam-Break Flood Forecasting Model," Hydrologic Research Laboratory, Office of Hydrology, National Weather Service, NOAA, Silver Spring, Maryland, 1980.

(3) Gundlach, D. L., and Thomas, W. A., "Guidelines for Calculating and Routing a Dam-Break Flood," Research Note No. 5, The Hydrologic Engineering Center, Davis, California, January, 1977.

(4) Price, J. T. and Garrison, J. M., "Flood Waves From Hydrologic and Seismic Dam Failures," Tennessee Valley Authority, Knoxville, Tennessee, January, 1973.

(5) Price, James T., Lowe, G. W., and Garrison, J. M., "Hydraulic Transients Generated by Partial and Total Failures of Large Dams," Tennessee Valley Authority, Knoxville, Tennessee, August, 1974.

(6) Su, Shih-Tun and Barnes, A. H., "Geometric and Frictional Effects on Sudden Releases," Journal of the Hydraulics Division, American Society of Civil Engineers, Volume 96, No. HY11, Proceeding Paper 7650, November, 1970, pp. 2185-2200.

(7) U.S. Army Corps of Engineers, "Floods Resulting From Suddenly Breached Dams: Conditions of Minimum Resistance," Hydraulic Model Investigation, Miscellaneous Paper No. 2-374, Report 1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, February, 1960.

(8) U.S. Army Corps of Engineers, "Floods Resulting From Suddenly Breached Dams: Conditions of High Resistance," Hydraulic Model Investigation, Miscellaneous Paper No. 2-374, Report 2, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, November, 1961.

(9) U.S. Army Corps of Engineers, "Military Hydrology Bulletin No. 9: Flow Through a Breached Dam," prepared in connection with Research and Development Project No. 8-97-10-003, Engineer Research and Development Division, Office of the Chief of Engineers, Washington, D.C., June 1957 (use for initial estimate only or where there is a large margin of safety).

(10) U.S. Department of Commerce, "Proceedings of Dam-Break Flood Routing Model Workshop" Sponsored by U.S. Water Resources Council in Bethesda, Maryland on October 18-20, 1977, National Technical Information Service, PB-275437, Springfield, Virginia, October 1977.

(11) Whitham, G. B., "The Effects of Hydraulic Resistance in the Dam-Break Problem," Proceedings of the Royal Society of London, Series A. Mathematical I Physical Sciences, Volume 227, Royal Society Burlington House, Picadilly, London, February 8, 1955.

A16. Historical Dam Failures. Some of the references which describe historical dam failures are as follows:

(1) Babb, A. O. and Mermel, T. W., "Catalog of Dam Disasters, Failures and Accidents," PB 179243, U.S. Department of the Interior, Bureau of Reclamation, Washington, D.C., 1968.

(2) Gruner, E., "Dam Disasters," The Institution of Civil Engineers, Proceeding Paper No. 6648, Volume 24, January, 1963, pp. 47-60.

(3) Hamilton, D. H., and Meehan, R. L., "Ground Rupture in the Baldwin Hills," Science, American Association for the Advancement of Science, Volume 172, April 23, 1971, pp. 333-344.

(4) International Commission on Large Dams, "Lessons From Dam Incidents," 1973.

(5) Kirkpatrick, G. W., "Evaluation Guidelines for Spillway Adquacy," Proceedings of the Engineering Foundation Conference, Pacific Grove, California, November 18 - December 3, 1976.

(6) "Report of the Committee on the Cause of Failure of the South Fork Dam," Transactions, American Society of Civil Engineers, Volume XXIV, June, 1891.

(7) Scott, K. M., and Gravlee, G. C. Jr., "Flood Surge on the Rubicon River, California - Hydrology, Hydraulics, and Boulder Transport," U.S. Geological Survey Open-File Report, May 2, 1967.

(8) Wiley, A. J. (Chairman) et al., "Report of the Commission Appointed by Governor G. C. Young to Investigate the Causes Leading to the Failure of the St. Francis Dam near Saugus, California," California State Printing Office, Sacramento, 1928.

(9) United States Committee on Large Dams, "Lessons from Dam Incidents, USA," published by the American Society of Civil Engineers, 1975.

(10) U.S. Department of the Interior, Teton Dam Failure Review Group, "Failure of Teton Dam," A Report of Findings, Bureau of Reclamation, Denver Federal Center, Denver, Colorado, April, 1977.

(11) Independent Panel to Review Cause of Teton Dam Failure, "Report to U.S. Department of the Interior and State of Idaho on Failure of Teton Dam," December, 1976.

A17. Wave Runup. As applicable, the following references may be used to determine wave runup additives to dam failure analysis:

(1) Saville, T., McClendon, E. W., and Cochran, A. L., "Freeboard Allowances for Waves in Inland Reservoirs," Journal of the Waterways and Harbors Division, American Society of Civil Engineers, Volume 88, No. WW2, May, 1962, pp. 93-124.

(2) U.S. Army Corps of Engineers, "Waves in Inland Reservoirs," Summary Report on Civil Works Investigation Projects CW-164 and CW-165, Technical Memorandum No. 132, Beach Erosion Board, Washington, D.C., November, 1962.

(3) U.S. Army Corps of Engineers, Policies and Procedures Pertaining to Determination of Spillway Capacities and Freeboard Allowances for Dams," Engineering Circular No. 1110-2-27, Change 1, Office of the Chief of Engineers, Washington, D.C., February 19, 1968, pp. 12-17.

(4) U.S. Army Corps of Engineers, "Wave Runup and Wind Setup on Reservoir Embankments," ETL 1110-2-221, Office of the Chief of Engineers, Washington, D.C., November 29, 1976.

(5) U.S. Army Corps of Engineers, "Reanalysis of Wave Runup on Structures and Beaches," Technical Paper 78-2, Coastal Engineering Research Center, Fort Belvoir, Virginia, March 1978.

(6) U.S. Army Corps of Engineers, "Wave Runup on Rough Slopes," Engineering Technical Aid No. 79-1, Coastal Engineering Research Center, Fort Belvoir, Virginia, July 1979.

A18. Landslides. The following references may be appropriate:

(1) Davidson, D. D. and McCartney, B. L., "Water Waves Generated by Landslides in Reservoirs," Journal of the Hydraulics Division, ASCE, Volume 101, No. HY12, Proc. Paper 11791, December, 1975, pp. 1489-1501.

(2) Fread, D. L., "DAMBRK: The NWS Dam-Break Flood Forecasting Model," Hydrologic Research Labaroatory, Office of Hydrology, National Weather Service, NOAA, Silver Spring, Maryland, 1980.

(3) Koutitas, C. G., "Finite Element Approach to Waves Due to Landslides," Journal of the Hydraulics Division, ASCE, Volume 103, No. HY9, Proc. Paper 13218, September, 1977, pp. 1021-1029.

(4) Wiegel, R. L., et al., "Water Waves Generated by Landslides in Reservoirs," Journal of the Waterways and Harbors Division, ASCE, Volume 96, No. WW2, Proc. Paper 7279, May, 1970, pp. 307-334.

A19. Probable Maximum Hurricane. Literature to assist in this analysis includes the following:

(1) Graham, H. E. and Nunn, D. E., "Meteorological Consideration Pertinent to Standard Project Hurricane, Atlantic and Gulf Coasts of the United States," National Hurricane Research Project Report No. 33, U.S. Weather Bureau, Washington, D.C., 1959.

(2) Harris, D. L., "Characteristics of Hurricane Storm Surge," Technical Paper No. 48, U.S. Weather Bureau, Washington, D.C., August, 1959.

(3) National Weather Service, "Revised SPH Criteria for the Atlantic and Gulf Coasts of the U.S.," Memorandam HUR 7-120, NOAA, Silver Spring, Maryland, June, 1972.

(4) Neumann, C. J., Cry, G. E., Caso, E. L., and Jarcinen, B. R., "Tropical Cyclones of the North Atlantic Ocean, 1871-1977," National Weather Services, NOAA, Silver Spring, Maryland, June, 1978.

A20. Probable Maximum Windstorm. The following may be used in studying PMWS:

(1) Lemire, F., "Winds on the Great Lakes," Meteorological Branch, Department of Transport, CIR-3560, TEC-380, 1971.

(2) Ludlum, D. M. Weather Record Book, Princeton, New Jersey, 1972.

(3) PSAR - Nine-Mile Point Nuclear Station - Unit 2, Docket No. 50-410, "Two-Dimensional Transient Surge Analysis on Lake Ontario," Niagara Mohawk Power Corporation.

(4) Stone and Webster Engineering Corporation, "Development and Verification of a Synthetic Northeaster Model for Coastal Flood Analysis," prepared for Federal Insurance Administration, Department of Housing and Urban Development, Washington, D. C., 1978.

A21. Moving Squall Line. The following may be used in studying moving squall line:

(1) Fujita, T., "Results of Detailed Synoptic Studies of Squall Lines," Tellus, Volume 7, 1955, pp. 405-436.

(2) Hughes, L. A., "The Prediction of Surges in the Southern Basin of Lake Michigan: Part III, The Operational Basis for Prediction," Monthly Weather Review, Volume 93, No. 5, May, 1965.

(3) Irish, S. M., "The Prediction of Surges in the Southern Basin of Lake Michigan: Part II, A case Study of the Surge of August 3, 1960," Monthly Weather Review, Volume 93, No. 5, May, 1965.

(4) Platzman, G. W., "A Numerical Computation of Surge of June 26, 1954, on Lake Michigan," Technical Report No. 1 to U.S. Weather Bureau (Now NOAA) Contract CWB 9270, University of Chicago, Illinois, June, 1958.

(5) Platzman, G.W., "A Procedure for Operational Prediction of Wind Setup on Lake Erie," Technical Report to U.S. Weather Bureau (Now NOAA), 1967.

(6) Platzman, G. W., "The Prediction of Surges in the Southern Basin of Lake Michigan: Part 1, The Dynamical Basis for Prediction," Monthly Weather Review, Volume 93, No. 5, May, 1965.

A22. Open Coast Surges. The following may be used to calculate open coast PMH surges:

(1) FSAR, Docket No. 50-302, "Verification Study of Dames and Moore's Hurricane Storm Surge Model With Application to Crystal River Unit 3 Nuclear Plant, Crystal River, Florida," Florida Power Corporation, Florida, July 13, 1973.

(2) Jelesnianski, C. P., "Bottom Stress Time-History in Linearized Equations of Motion for Storm Surges," Monthly Weather Review, Volume 98, No. 6, June, 1970.

(3) Marinos, T. and Woodward, J. W. "Estimation of Hurricane Surge Hydrographs," Journal of the Waterways and Harbors Division, ASCE, Volume 94, WW2 Proc. Paper 5945, May, 1968, pp. 189-216.

(4) Pararas-Carayannis, George, "Verification Study of a Bathystrophic Storm Surge Model," Technical Memorandum No. 50, U.S. Army Corps of Engineers, Coastal Engineering Research Center, Fort Belvoir, Virginia, May, 1975.

(5) Pearce, B. R., and Pagenkopf, J. R., "Numerical Calculation of Storm Surges: An Evaluation of Techniques" presented at the May 5-8, 1975, Seventh Annual Offshore Technology Conference, Held at Houston, Texas.

(6) Stone and Webster Engineering Corporation, "Two-Dimensional Coastal Storm Surge Model, SWECO 7501-A," U.S. Nuclear Regulatory Commission, Washington, D.C., April, 1977.

(7) Thacker, W. C., "Irregular Grid Finite - Difference Techniques for Storm Surge Calculations for Curving Coastlines," presented at the 10th Liege Colloquim on Ocean Hydrodynamics, May 8-12, 1978, Liege, Belgium.

(8) Wanstrath, J. J., Whitaker, R. E., Reid, R. O., Vastano, A. C., "Storm Surge Simulation in Transformed Coordinates," Volume 1, Theory and Application, U.S. Army Corps of Engineers, Technical Report No. 76-3, 1976, pp. 166.

(9) Yeh, G. T., and Yeh, F. F., "A Generalized Model for Storm Surges," Proceedings, Fifteenth International Conference on Coastal Engineering, Volume 1, Chapter 54, Honolulu, Hawaii, July 7-11, 1978, pp. 921-933.

A23. Semi-Enclosed Bodies of Water Surges. The references listed below give appropriate techniques for handling PHM storm surge analysis in semi-enclosed bodies of water:

(1) Bretschneider, C. L. and Collins, J. I., "Prediction of Hurricane Surge: An Investigation for Corpus Christi, Texas, and Vicinity," NECSO Technical Report, SN-120, prepared by National Engineering Science Company for U.S. Army Engineering District, Galveston, Texas.

(2) Leendertse, J. J., "Aspects of a Computational Model for Long-Period Water Wave Propagation," RM 5294-PR, The Rand Corporation, Santa Monica, California, May, 1967.

(3) National Weather Service, "Estimation of Hurricane Storm Surge in Apalachicola Bay, Florida," NOAA Technical Report, Unpublished, Silver Spring, Maryland.

(4) Reid, R. O. and B. R. Bodine, "Numerical Model for Storm Surges in Galveston Bay," Proceedings, Journal of Waterways and Harbors Division, American Society of Civil Engineers, February, 1968, pp. 32-56.

(5) TRACOR, Inc., "Estuarine Modeling: An Assessment," Water Pollution Control Research Series 16070 DZV 02/71, Environmental Protection Agency, Water Quality Office, Washington, D. C., February, 1971.

(6) Tsai, Y. J. and Y. C. Chang, "Two-Dimensional Transient Hydrothermal Mathematical Model, Proceedings of the First World Congress on Water Resources, International Water Resources Association, Volume IV, September 24-28, 1973.

(7) U.S. Army Corps of Engineers, "Hurricane Surge Predictions for Chesapeake Bay," Miscellaneous Paper No. 3-59, Beach Erosion Board, Washington, D. C., September, 1959.

(8) U.S. Army Corps of Engineers, "Hurricane Surge Predictions for Delaware Bay," Miscellaneous Paper No. 4-59, Beach Erosion Board, Washington, D. C., November, 1959.

A24. Enclosed Bodies of Water Setup. The following may be used in calculating wind setup in enclosed bodies of water:

(1) Hellstrom, B., "Wind Effect on Lakes and Rivers," Ing. Vetenskaps Akad., Handl., No. 158, 1941.

(2) Hunt, I. A., "Winds, Wind Setups and Seiches on Lake Erie," U.S. Lake Survey, Research Report 1-2, U.S. Army Corps of Engineers, Washington, D. C., January, 1959.

(3) Keulegan, G. H., "Wind Tides in Small Closed Channels," National Bureau of Standards, (U.S.), Research Paper No. 2207, F. Res. Nat. Bur. Std., C., 46 (5) (1951).
(4) Linghaar, H. K., "Wind Tides in Inland Waters," Proceedings, Mid-Western Conference on Fluid Mechanics, 1951.
(5) Platzman, G. W., "A Procedure for Operational Prediction of Wind Setup on Lake Erie," Technical Report No. 11, to Environmental Science Services Administration, The University of Chicago, November, 1967.
(6) PSAR Docket No. 50-410, Nine Mile Point Nuclear Station - Unit 2, "Two-Dimensional Transient Surge Analysis on Lake Ontario," Niagara Mohawk Power Corporation.
(7) Tsai, Y. J., and Y. C. Chang, "Two-Dimensional Transient Hydrothermal Mathemetical Model," Proceedings of the First World Congress on Water Resources, International Water Resources Association, Volume IV, September 24-28, 1973.

A25. Deepwater Wave Generation. The following may be used to calculate generation of deepwater waves:

(1) Bretschneider, C., "The Nondimensional Stationary Hurricane Wave Model," Offshore Technology, 1972.
(2) Resio, D. T., Garcia, A. W., and Vincent, C. L., "Preliminary Investigation of Numerical Wave Models," Symposium on Technical, Environmental, Socioeconomic and Regulatory Aspects of Coastal Zone Management, ASCE, San Francisco, California, March 14-16, 1978, pp. 2085-2104.
(3) Wiegel, R. L. Oceanographical Engineering, Prentice Hall, New Jersey, 1964.
(4) Wilson, B. W., "Deepwater Wave Generation by Moving Wind Systems," Transactions, American Society of Civil Engineers, Volume 128, Part IV, 1963, pp. 104-131.
(5) Wilson, B. W., "Graphical Approach to the Forecasting of Waves in Moving Fetches," Technical Memorandum No. 73,, Beach Erosion Board, U.S. Army Corps of Engineers, Washington, D. C., April, 1955.

A26. Wave Transformation. The following may be used in analyzing wave transformation to the site:

(1) Collins, J. I., "Prediction of Shallow Water Spectra," Journal of Geophysical Research, Volume 77, May, 1972.
(2) Cross, R. H. and Sollete, C. K., "Wave Transmission by Overtopping," Proceedings, Journal of the Waterways and Harbors Division, American Society of Civil Engineers, Volume 98, No. WW3, August, 1972.
(3) Diskin, M. H. and Vayda, M. K., "Piling-Up Behind Low and Submerged Permeable Breakwaters," Proceedings, Journal of the Waterways and Harbors Division, American Society of Civil Engineers, Volume 96, No. WW2, May, 1970.
(4) Lording, P. T. and Scott, J. R., "Armor Stability of Overtopped Breakwater," Proceedings, Journal of the Waterways and Harbors Division, American Society of Civil Engineers, Volume 97, No. WW2, May, 1971.

A27. Wave Erosion. Examples of wave erosion analyses are presented in the references below:

(1) Florida Power and Light Company, "Amendment No. 37, Preliminary Safety Analysis Report, St. Lucie Plant Unit 2," Docket No. 50-389, August, 1975.
(2) Florida Power and Light Company, "Amendment No. 41, Preliminary Safety Analysis Report, St. Lucie Plant Unit 2," Docket No. 50-389, October, 1975.

A28. Wave Forces. The following are references on wave forces:

(1) Brater, E. F. and McKnown, J. S., "Wave Forces on Submerged Structures," Journal of the Hydraulics Division, American Society of Civil Engineers, Volume 84, HY6, Paper No. 1833, November, 1958.

(2) Chakrabarti, S. K., "Wave Forces on Submerged Objects of Symmetry," Journal of the Waterways, Harbors, and Coastal Engineering Division, American Society of Civil Engineers, Volume 99, No. WW2, Paper No. 9698, May, 1973, pp. 147-164.

(3) Garrison, C. J. and Chow, P. Y., "Wave Forces on Submerged Bodies," Journal of the Waterways, Harbors, and Coastal Engineering Division, American Society of Civil Engineers, Volume 98, No. WW3, Paper No. 9098, August, 1972, pp. 375-392.

(4) Ippen, A. T., *Estuary and Coastline Hydrodynamics,* McGraw-Hill Book Company, Inc., New York, 1966.

(5) Wiegel, R. L., *Oceanographic Engineering,* Prentice Hall, Inc., New York, 1964.

A29. Seiche. The following sources describe principles and methods of computation of harbor and bay oscillations:

(1) Chen, H. S. and Mei, C. C., "Oscillations and Wave Forces in a Man-Made Harbor in the Open Sea", Proceedings of the 10th symposium on Naval Hydrodynamics, Cambridge, Massachusetts, 1974 pp. 573-596.

(2) Houston, J. R. "Long Beach Harbor Numerical Analysis of Harbor Oscillations, "Miscellaneous Paper H-76-20, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, 1976.

(3) Lamb, H., *Hydrodynamics,* (1932 ed), Dover, New York, 1945.

(4) Raichlen, F., "Long-Period Oscillations in Basins of Arbitrary Shapes," Proceedings, American Society of Civil Engineers Specialty Conference on Coastal Engineering, 1965, pp. 115-145.

(5) Wilson, B. W., Jen, Y., and Hendrickson, J. A., "Surge in the Southeast Basin, Long Beach Harbor, California," Proceedings of the 13th Coastal Engineering Conference, Vancouver, B.C., Canada, July, 1972.

A30. Ice Flooding. The following references may be used in evaluating ice flooding:

(1) Bryce, J. B., "Ice and River Control," Journal of the Power Division, American Society of Civil Engineers, Volume 94, P02, Proceeding Paper 6246, November, 1968, pp. 177-181.

(2) Burgi, P. H. and Johnson, P. L., "Ice Formation - A Review of the Literature and Bureau of Reclamation Experience," USBR REC-REC-71-8, September, 1971.

(3) "Ice Pressure Against Dams: A symposium," Transactions, American Society of Civil Engineers, Volume 119, 1954, pp. 1-42.

(4) Korzhavin, K. N., "Action of Ice on Engineering Structures," CRREL-TL260, U.S. Army Corps of Engineers, Cold Regions Research Engineering Laboratory, Hanover, New Hampshire, December, 1971.

(5) Logan, T. H., "Prevention of Frazil Ice Clogging of Water Intakes by Application of Heat," USBR, REC-ERC-74-15, Denver, Colorado, September, 1974.

(6) Michel, Bernard, "Ice Pressures on Engineering Structures," CRREL-111-B1b, U.S. Army Corps of Engineers, Cold Regions Research Laboratory, Hanover, New Hampshire, June, 1970.

(7) Michel, Bernard, "Wind Regime of Rivers and Lakes," CRREL-111-Bla, U.S. Army Corps of Engineers, Cold Regions Research Engineering Laboratory, Hanover, New Hampshire, April, 1971.

(8) Pariset, E., and Hausser, R., "Frazil Ice and Flow Temperature under Ice Covers," The Engineering Journal (Canada), January, 1961, pp. 46-49.

(9) Pariset, E., Hausser, R., and Gaynon, A., "Formation of Ice Covers and Ice Jams in Rivers," Journal of the Hydraulics Division, American Society of Civil Engineers, Volume 92, HY6, November, 1966, pp. 1-24.

(10) Task Committee on Hydromechanics of Ice of the American Society of Civil Engineers, "River-Ice Problems: A State-of-the-Art Survey and Assessment of Research Needs," Journal of the Hydraulics Division, American Society of Civil Engineers, Volume 100, No. HY1, January, 1974.

(11) U.S. Army Corps of Engineers, "Bibliography on Cold Regions Science and Technology," published periodically, Cold Regions Research Engineering Laboratory, Hanover, New Hampshire.

(12) Williams, G. P., "Probability Charts for Predicting Ice Thickness," The Engineering Journal, June, 1963, pp. 31-35.

A31. Canals. The following publications may be used in providing methods of hydraulic design bases for canals:

(1) Chow, V. T., *Open Channel Hydraulics,* McGraw-Hill Book Company, Inc., New York, 1959.

(2) Ferrell, P. W. and Borland, W. M., "Design of Stable Canals and Channels in Erodible Material," Transactions, American Society of Civil Engineers, Volume 123, Paper 2913, 1958, pp. 101-115.

(3) U.S. Army Corps of Engineers, "Additional Guidance for Riprap Channel Protection," EM 1110-2-120, Office of Chief of Engineers, Washington, D. C., May 1971.

(4) U.S. Army Corps of Engineers, "Engineering and Design of Flood Control Channels," EM 1110-2-1601, Office of the Chief of Engineers, Washington, D. C., July 1, 1970.

(5) U.S. Army Corps of Engineers, "Hydraulic Design Criteria," Waterways Experiment Station, various publications and dates.

(6) U.S. Army Corps of Engineers, Waterways Experiment Station, various hydraulic model studies, various dates.

A32. Reservoirs. The following materials may be used to determine design bases for dams and spillways:

(1) Beach Erosion Board, "Wave in Inland Reservoirs," Summary Report on Civil Works Investigation, Projects CW-64 and CW-165, Technical Memorandum No. 132, Office of the Chief of Engineers, Washington, D. C., November, 1962.

(2) Bureau of Reclamation, Design of Small Dams, U.S. Department of the Interior, Washington, D.C., Second Edition, 1973.

(3) Cochran, A. L., "Quantitative Estimates of Wave-Overtopping of Levees and Flood Walls," Memorandum for Record, Hydrology and Hydraulics Branch, Enginering Division, Civil Works, Office of the Chief of Engineers, January 15, 1962.

(4) Saville, T., McClendon, E. W., and Cochran, A. L., "Freeboard Allowances for Waves in Inland Reservoirs," Journal of the Waterways and Harbor Division, American Society of Civil Engineers, Volume 88, No. WW2, May 1962, pp. 93-124.

(5) Sherard, J. L., Woodward, R. J. Gizienski, S. F., and Clevenger, W. A., Earth and Earth-Rock Dams, John Wiley and Sons, Inc., New York, 1963.

(6) U.S. Army Corps of Engineers, "Hydraulic Design of Spillways," EM 1110-2-1603, Office of the Chief of Engineers, Washington, D.C., March 31, 1965.

(7) U.S. Army Corps of Engineers, "Hydraulic Design of Reservoir Outlet Structures," EM 1110-2-1602, Office of the Chief of Engineers, Washington, D.C., August 1, 1963.

(8) U.S. Army Corps of Engineers, "Policies and Procedures Pertaining to Determination of Spillway Capacities and Freeboard Allowance for Dams," Engineering Circular No. 1110-2-27, Change 1, Office of the Chief of Engineers, Washington, D.C., February 19, 1968.

(9) U.S. Army Corps of Engineers, "Hydraulic Design Criteria," Waterways Experiment Station, various publications and dates.

(10) U.S. Army Corps of Engineers, Waterways Experiment Station, various hydraulic model studies, various dates.

A33. Sedimentation. The following materials may be used for studying sedimentation, or sediment transport:

(1) Gottschalk, L. C., "Reservoir Sedimentation," Section 17-1, *Handbook of Applied Hydrology,* Chow, V. T., Editor, McGraw-Hill Book Company, Inc., New York, 1964, pp. 17-1 to 17-34.
(2) Linsley, R. K., Kohler, M. A., Paulhus, J. L. H., Applied Hydrology, McGraw-Hill Book Company, Inc., New York, 1949, pp. 316-355.
(3) Vanoni, V. A., Editor, "Sedimentation Engineering," ASCE - Manuals and Reports on Engineering Practice No. 54, prepared by ASCE Task Committee for the Preparation of the Manual on Sedimentation of the Sedimentation Committee of the Hydraulics Division, American Society of Civil Engineers, New York, 1975.

A34. Overland Flow. The following are some references that describe overland flow:

(1) Crawford, N. H., and Linsley, R. K., "Digital Simulation in Hydrology: Stanford Watershed Model IV," Technical Report No. 39, Department of Civil Engineering, Stanford University, Stanford, California, July, 1966, pp. 16-17.
(2) Izzard, C. F., "Hydraulics of Runoff from Developed Surfaces," Proceedings, Highway Research Board, Volume 26, 1946, pp. 129-150.
(3) Overton, D. E. and Meadows, M. E., *Storm Modeling,* Academic Press, New York, 1976.