ENGINEERING GUIDELINES
FOR
THE EVALUATION OF
HYDROPOWER PROJECTS

FEDERAL ENERGY REGULATORY COMMISSION
OFFICE OF HYDROPOWER LICENSING
JULY 1987
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Preface

These engineering guidelines have been prepared by the Office of Hydropower Licensing (OHL) to provide guidance to the technical Staff in the processing of applications for license and in the evaluation of dams under Part 12 of the Commission's regulations. The Guidelines will also be used to evaluate proposed modifications or additions to existing projects under the jurisdiction of the Federal Energy Regulatory Commission (Commission). Staff technical personnel consist of the professional disciplines (e.g. professional engineers and geologists) that have the responsibility for reviewing studies and evaluating designs prepared by owners or developers of dams.

The guidelines are intended to provide technical personnel of the Office of Hydropower Licensing, including the Regional Office and Washington Office personnel, with procedures and criteria for the engineering review and analysis of projects over which the Commission has jurisdiction. In addition, these guidelines should be used in the evaluation of consultant or licensee/exemptee conducted studies. The guidance is intended to cover the majority of studies usually encountered by Staff. However, special cases may require deviation from, or modification of, the guidelines. When such cases arise, Staff must determine the applicability of alternate criteria or procedures based upon their experience and must exercise sound engineering judgment when considering situations not covered by the guidelines. The alternate procedures, or criteria, used in these situations should be justified and accompanied by any suggested changes for incorporation in the guidelines. Since every dam site and hydropower related structure is unique, individual design considerations and construction treatment will be required. Technical judgment is therefore required in most analytical studies.

These guidelines are not a substitute for good engineering judgment, nor are the procedures recommended herein to be applied rigidly in place of other analytical solutions to engineering problems encountered by staff. Staff should keep in mind that the engineering profession is not limited to a specific solution to each problem, and that the results are the desired end to problem solving.

These guidelines are primarily intended for internal use by OHL staff, but also provide criteria for licensees, exemptees, and applicants for use in any studies presented to the Commission under Parts 4 and 12 of the Regulations (18 CFR, Parts 4 and 12). When any portions of the Guidelines becomes outdated, obsolete, or needs revision for any reason, they will be revised and supplemented as necessary. Comments on, or recommended changes, in these guidelines should be forwarded to the Director of the Division of Inspections for consideration and possible inclusion in future updates. New pages will be prepared and issued with instructions for page replacements.
CHAPTER I

GENERAL REQUIREMENTS

JULY 1987
# Chapter I  
**General Requirements**

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Chapter I

General Requirements

1-1 Purpose and Scope: The purpose of this chapter of the guidelines is to establish the basic requirements for engineering review and studies conducted by Commission Staff during the processing stage of license applications and the review of reports prepared by licensees, exemptees, or independent consultants.

The following Federal Power Act regulations, and Division of Inspections, Office of Hydropower Licensing Operating Manual, provide general policy concerning: the contents, deposition, and evaluation of applications for license, exemption, or preliminary permit; and the supervision of existing permits and licenses.

1-1.1 Regulations

Preliminary Permit or License for Water Power License Subchapter B, Part 4, Subpart D, Section 4.30 and Section 4.80.

Application for License for Major Unconstructed Project and Major Modified Project; and Application for Amendment to License. - Subchapter B, Part 4, Subpart E, Section 4.40 and Subpart H, Section 4.70 and Subpart L, Section 4.200.

Application for License for Major Project - Existing Dam - Subchapter B, Part 4, Subpart F, Section 4.50.

Application for License for Minor Water Power Projects and Major Water Power Projects 5 Megawatts or Less - Subchapter B, Part 4, Subpart G. Section 4.60.

Exemptions of Small Hydroelectric Power Projects of 5 Megawatts or Less - Subchapter B, Subpart K, Section 101.

1-2 Project Classification:

1-2.1 Downstream Hazard Potential

The hazard potential of dams pertains to potential for loss of human life or property damage in the area downstream of the dam in event of failure or incorrect operation of a dam does not indicate the structural integrity of the dam itself, but rather the effects if a failure should occur. The hazard potential assigned to a dam is based on consideration of the effects of a failure during both normal and flood flow conditions.

Dams conforming to criteria for the low hazard potential category generally are located in rural or agricultural areas where failure may damage farm buildings, limited agricultural land, or township and country roads, or have a small storage capacity, the release of which would be confined to the river channel in the event of a failure and therefore would represent no danger to human life.

Significant hazard potential category structures are usually located in predominately rural or agricultural areas where failure may damage isolated homes, secondary highways or minor railroads or cause interruption of use or service or relatively important public utilities, cause some incremental flooding of structures with possible danger to human life.

Dams in the high hazard potential category are those located where failure may cause serious damage to homes, extensive agricultural, industrial and commercial facilities, important public utilities, main highways, or railroads, and there would be danger to human life.

The hazard potential evaluation includes consideration of recreational development and use. Included in the high hazard potential category are dams where failure would cause serious damage to permanently established or organized recreational areas or activities. Also included in the high hazard potential category are dams where failure would result in loss of life of people gathered for an unorganized recreational activity (such as salmon fishermen and kayakers) where concentrated use of a
confined area below the dam is a common annual occurrence during certain times each year. Based on the proximity of the recreational activities to the dam, it may be necessary to require a dam break analysis to evaluate the impact of failure on those recreationists. The evaluation of hazard classification under these circumstances is not as precise as with permanent structures. Therefore, careful analysis of data will be required.

1-2.2 **Project Capacity Classification**

Projects having installed capacity of 1,500 kW or less are classified as minor projects and projects with an installed capacity greater than 1,500 kW are major projects.

1-3 **Study Requirements:**

1-3.1 **General**

The following guidance shall establish the basic requirements for reviews and studies conducted by both the Washington and Regional Offices. It is recognized that unique situations may require deviations from these guidelines, however, they are considered flexible enough to be followed for most of the basic types of reviews and studies anticipated. In any engineering study which is conducted, regardless of whether it is in compliance with these study requirements or not, the technical portion of the studies shall comply with the applicable sections of these guidelines. Appendix I-A includes an outline covering the data and studies necessary to complete the engineering review for a project.

1-3.2 **Regional Office Studies**

1-3.2.1 **General** - The operating manual, prepared by the Division of Inspections (DINS), establishes minimum requirements for reports and field inspections of hydroelectric projects conducted pursuant to the Federal Power Act. The following guidance is intended to be used in conjunction with the operating manual.
1-3.2.2 Prelicense Inspections - A prelicense inspection for any constructed project shall be conducted upon receipt of the application for license and prior to completion of engineering review of the application by the Washington Office.

1-3.2.3 Inspection Reports - Inspections of licensed projects should be conducted in accordance with Chapter II or Chapter III of the Operating Manual, as applicable. The report of inspection should summarize the general condition of the project and note any terms of the license not complied with, as well as matters requiring immediate attention. Items which constitute a changed condition and require special study, or engineering analyses, should be brought to the attention of the Director, Division of Inspections.

1-3.3 Washington Office Studies

1-3.3.1 General - The following guidelines are recommended for use by the Washington Office Staff.

1-3.3.2 License Applications

Review for Deficiencies - All license applications shall be reviewed for compliance with the engineering requirements of FERC regulations. Application deficiencies should be documented so the applicant can be appropriately and timely notified. A preliminary safety and design assessment, economic feasibility study and resource availability analysis shall be conducted prior to the acceptance of the application. Items which should be examined include: the need for project power; the existence (or absence) of an agreement or memorandum of understanding for sale of project power; the impact of changes in fish habitat preservation flow releases on power generation; and the reasonableness of the project construction cost estimate. This study should resolve
any basic questions concerning the ability of the Applicant to build the project and/or sell the project power.

Safety and Design Assessment - The safety and design assessment report shall include a summary of the conclusions and recommendations resulting from the engineering data in the license applications and technical review and studies based on such data.

The report shall become the data base for the project, and the safety and adequacy of all future amendments or changes to the project will be evaluated in supplements to the original report.

For constructed, small, low hazard projects, the safety and adequacy shall be based upon the information supplied by the Regional Office in the prelicense inspection report and that contained in the application. Additional studies should only be conducted when deemed necessary due to conditions noted during the site inspection. Inspections performed subsequent to the prelicense inspection should be reviewed for changed conditions which might justify additional studies.

1-3.3.3 Review of Consultants Reports

Review of Board of Consultants Report - In all licenses authorizing major construction the licensee is required to employ a board of qualified independent consultants, approved by the Director, DINS, to review the design, specifications, and construction of the project. The board is expected to assess the construction inspection program, construction procedures and progress, planned instrumentation, the filling schedule for the reservoir, and plans for surveillance during initial filling of the reservoir.
Staff review of consultants reports should examine all recommendations made by the Board. Recommendations which are inconsistent with Commission policy, or previously stated staff positions on a particular problem with the project, should be questioned and the differences resolved.

**Review of Part 12 Inspection Reports** - Reference is made to Chapter IV of the Operating Manual, which establishes the Commission's policy concerning Part 12 inspections. Paragraph 4-7, of Chapter 4, gives specific guidance to Regional Directors concerning the review of Consultants Part 12 reports. The Regional Director's recommendations and comments should be reviewed by Staff in conjunction with any review of a Consultant's report.

**1-3.3.4 Review of Staff Studies** - Independent analyses conducted by any member of the Staff shall be reviewed by another staff member for completeness and soundness of theory. This review shall consist of a check of both the engineering theories used and the mathematical calculations performed. Any deficiencies in the study noted by the reviewer shall be corrected prior to final approval or submittal of the study report.

**1.4 Deviations From the Guidelines:**

**1.4.1 Changes**

Guideline criteria and recommendations which are found to be technically incorrect, or outdated, should be brought to the attention of the Director, Division of Inspections. The Director, Division of Inspections will consult and coordinate any request for changes with the Director, Division of Project Management who has design review responsibilities at the pre-licensing stage. This shall be done in writing,
with the incorrect or outdated passages cited, and shall include the Staff member's recommendations for correcting the deficiency.

### 1.4.2 Deviations

Deviations from the guidelines shall be subject to the approval of the Director, Division of Inspections. The Director, Division of Inspections will consult and coordinate any request for deviations with the Director, Division of Project Management who has design review responsibilities at the pre-licensing stage. The procedures, or criteria, used in lieu of guideline recommendations shall be justified in writing for inclusion in the guideline files, and shall be accompanied by any suggested changes in the guidelines that may be necessary to incorporate such procedures in future revisions.

#### 1.5 References:


2. National Program of Inspection of Non-Federal Dams, Department of the Army, ER 1110-2-106, September 26, 1979, (change 1).

APPENDIX IA

Engineering Review and Studies Outline
General

The following engineering outline includes the data and studies necessary to complete an assessment of a project. This outline is general by necessity so that it can be utilized by the Office of Hydropower Licensing Staff which includes the Regional Office Staff. Therefore, use of this outline should be tailored to the type of engineering review and study being made, and the type of report prepared.
Engineering Review and Studies Outline

A. Summary of Findings

1. Conclusions

1.1 Spillway adequacy (hydraulic)
1.2 Structural stability (of all project features)
1.3 Operation
1.4 Existing conditions (if applicable)
1.5 Resource utilization
1.6 Economic feasibility

2. Recommendations: (should be based upon, or supported by, the conclusions)

2.1 License articles
2.2 Approvals

B. Project Description

1. General Data

1.1 Location of project (include regional vicinity map)
1.2 Name of river, or stream
1.3 Hazard potential
1.4 Seismic zone
1.5 Year construction completed

2. Hydrologic and Hydraulic Data

2.1 Reservoir(s)
2.1.1 Elevation of top of dam
2.1.2 Elevation of normal pool
2.1.3 Storage capacity at normal pool
2.1.4 Storage capacity at top of control structure (gates, etc.)

2.2 Drainage basin
2.2.1 Area
2.2.2 Runoff characteristics
2.2.3 Inflow design floods
2.2.4 Probable maximum flood
2.2.5 Availability of historical stream flow data
2.2.6 Historical floods
2.3 Spillway(s)
   2.3.1 Description
   2.3.2 Type control (number, type and size of gates)
   2.3.3 Crest elevation and length
   2.3.4 Hydraulic capacity at maximum pool

2.4 Outlet works
   2.4.1 Description
   2.4.2 Type control (number, type and size of gates)
   2.4.3 Size and shape of conveyance structure
   2.4.4 Entrance invert elevation
   2.4.5 Exit invert elevation
   2.4.6 Hydraulic capacity (state pool elevation)

2.5 Other drawdown facilities
   2.5.1 Description (penstocks, sluices, etc.)
   2.5.2 Size, shape, and length
   2.5.3 Hydraulic capacity (state pool elevation)
   2.5.4 Discharge capacity of turbine(s)

3. Dam and appurtenant structures data

3.1 Dam(s) or Impoundment Structure(s)
   3.1.1 Description
   3.1.2 Elevation of top of dam
   3.1.3 Height of dam and length at crest
   3.1.4 Materials of construction
   3.1.5 Instrumentation

3.2 Powerhouse(s)
   3.2.1 Description
   3.2.2 Size (structural)
   3.2.3 Size and number of generating units
   3.2.4 Materials of construction
   3.2.5 Instrumentation

3.3 Appurtenant Structures
   3.3.1 Description
   3.3.2 Size and functions of structures
   3.3.3 Pertinent elevations (hydraulic structures)
   3.3.4 Materials of construction
   3.3.5 Instrumentation

4. Foundations

4.1 Regional Geology
   4.1.1 Description
   4.1.2 Rock types and tectonics
   4.1.3 Engineering properties
4.2 Geology (local)
   4.2.1 Description
   4.2.2 Treatments (grouting, drainage, etc.)
   4.2.3 Existing conditions (seepage faults, etc.)
   4.2.4 Seismicity
   4.2.5 Instrumentation
   4.2.6 Engineering properties

4.3 Deficiencies and problem areas

5. Electrical, mechanical, and transmission equipment
   5.1 Description
   5.2 Installed capacity (hp and kW)
   5.3 Name plate ratings of generators and turbines

6. Project operation and resource utilization
   6.1 Manual or automatic operation
   6.2 Annual plant factor
   6.3 Dependable capacity
   6.4 Average annual energy
       6.4.1 minimum, mean, and maximum recorded flows at powerplant
       6.4.2 flow duration curve
       6.4.3 area capacity curve
       6.4.4 tailwater rating curve
       6.4.5 powerplant capability vs. head curve
       6.4.6 hydraulic capacity of powerplant
   6.5 Future development plans

C. Evaluation of Design, Construction, and Performance

1. General - Review the pertinent existing and available engineering data collected in Part B. Staff generated studies, when necessary, should be made. Studies should be summarized in the form of tables whenever possible. Technical studies should be conducted in accordance with the applicable portions of this manual.

2. Unconstructed Projects - The proposed structures shall be examined to determine their site suitability and ability to perform the functions for which they are intended.
If final design of the project has been completed, the design data and calculations for the various project structures should be reviewed to determine if all appropriate loading conditions were considered, and if state-of-art procedures and criteria were used. 1/ If the final design is not available, or is not completed, the preliminary studies shall be reviewed for adequacy. License articles should be recommended requiring the applicant to submit final design drawings and calculations for Commission review prior to construction. For this case, final staff review of the structural and hydraulic designs should be deferred until receipt of these drawings and calculations. Plans and specifications should also be submitted, for staff review. Phased review of the documents may be permitted, however, construction of a project feature may not begin until applicable exhibits or plans and specifications has been accepted. The articles should reserve the Commission's right to require changes in the contract drawings and specifications in order to ensure a safe and adequate project.

3. Constructed Projects - If available, the original design and design data should be examined to determine if all appropriate loading conditions were considered. The design criteria should be reviewed to determine if changed conditions at the site have created any need for changes in the criteria such as loadings, flows, etc. Any updated design data, such as newly developed floods, regional seismicity studies, changes in material properties, etc., should be studied to determine their influence on the structure. The data should be reviewed to determine if they are correct and if the latest state-of-art analysis procedures and information have been considered.

If original design data is not available, an independent analysis of the project is required.

D. Hydroelectric Power and Resources Utilization Evaluation

1. General
2. Unconstructed Projects
   economic feasibility study
   evaluation of proposed operation
   resource utilization
   future plans

1/ The term state-of-art as used in these guidelines refers to engineering analyses that are generally accepted methodologies by the engineering profession.
resource utilization
evaluation of operation
utilization of power generated
power value
dependable capacity
future developments
flow duration

3. Constructed Projects
resource utilization
evaluation of operation
utilization of power generated
power value
dependable capacity
future developments
flow duration
CHAPTER II
SELECTING AND ACCOMMODATING INFLOW
DESIGN FLOODS FOR SPILLWAYS
CHAPTER II
SELECTING AND ACCOMMODATING INFLOW DESIGN FLOODS FOR SPILLWAYS

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Chapter II
Selecting and Accommodating
Inflow Design Floods for Spillways

2-1 Purpose and Scope

The purpose of these guidelines is to define the appropriate inflow design flood to be used in the review of spillway and appurtenant structure designs by FERC staff and to conform to the provisions of the Federal Guidelines for Dam Safety.

These guidelines are not intended to provide a complete manual of all procedures used for estimating inflow design floods for spillways, because the selection of procedures is dependent upon available hydrologic data and individual watershed characteristics. To comply with these guidelines, all studies submitted to the Commission should be performed by a highly competent engineer experienced in hydrology and hydraulics, and should contain a summary of the design assumptions, design analyses, and methodology used to evaluate the inflow design flood.

2-2 Definition of Terms

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

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

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

Hazard - A situation which creates the potential for adverse consequences such as loss of life, property damage, and adverse social and environmental impacts. Impacts occur in a defined area downstream of a dam from flood waters released through spillways and outlet works of the dam or waters released by partial or complete failure of the dam. They may also be for an area upstream of the dam from effects of backwater flooding or effects of landslides around the reservoir perimeter.

Hydrograph - A graphical representation of the streamflow stage or discharge as a function of time at a particular point on a watercourse.
**IDF (Inflow Design Flood)** - The flood hydrograph used in the design of a dam and its appurtenant works particularly for sizing the spillway and outlet Works, and for determining maximum height of dam and temporary storage requirements.

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

**One Percent Chance Flood** - A flood that has 1 chance in 100 of being equaled or exceeded in a specified time period, usually 1 year.

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

**PMF (Probable Maximum Flood)** - The most severe flood that can be expected at a site as a result of the PMP developed from probable hydrologic and meteorologic conditions.

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

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

**Spillway** - A hydraulic structure used to convey water from a reservoir which may be either gated or ungated. Definition of specific types of spillways follow:

- **Service Spillway**. A spillway that is designed to provide continuous or frequent regulated or unregulated releases from a reservoir without significant damage to either the dam or its appurtenant structures.

- **Auxiliary Spillway**. Any secondary spillway which is designed to be operated very infrequently and possibly in anticipation of some degree of structural damage or erosion to the spillway during operation.

- **Emergency Spillway**. A spillway that is designed to provide additional protection against overtopping of dams and is intended for use under extreme conditions such as misoperation or malfunction of the service spillway.
Stillwater Level - The elevation that a water surface would assume if all wave action were absent.

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

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

2-3 Determination of Inflow Design Flood

The Commission's Order No. 122, issued January 21, 1981, states that the adequacy of a spillway must be evaluated by considering the hazard potential which would result from failure of the project works during flood flows. If failure of the project works would present a hazard to human life or would cause significant property damage, the project works must be designed to either withstand the loading or overtopping which may occur during a flood up to the probable maximum flood, or to the point where a failure would no longer constitute a hazard to downstream life and property, or the capacity of the spillway must be adequate to prevent the reservoir from rising to an elevation that would endanger the project works. If it is determined that failure of project works would not present a hazard to human life or cause significant property damage, this determination must be supported by a detailed analysis and an explanation of the basis for the finding.

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

Analyses of dam failures are complex with many historical dam failures not completely understood. The principal uncertainties in determining outflow from a dam failure involve the mode and degree of failure. These uncertainties can be circumvented in situations where it can be shown that the complete and sudden removal of the dam would not endanger human life or cause extensive property damage. Otherwise, reasonable failure postulations and sensitivity analyses such as those suggested in Appendix IIA should be used. Although a study using the modes of failure suggested in Appendix IIA of these guidelines may indicate that a hazard does not exist, a hazard could exist for a more extensive mode of failure. In such cases, the more extensive mode of failure should be investigated to determine whether a need for remedial action is required.
2-3.1 **Hazard Evaluation**

2-3.1.1 **General**

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

Once a dam is constructed, the downstream hydrologic regime is permanently changed. The change in hydrologic regime could alter land use patterns to encroach on the flood plain that would otherwise not be developed without the dam. Consequently, for conditions with the dam in place, evaluation of the impacts of dam failure should be based upon existing development and known and prospective future development. Comparisons between existing downstream conditions with and without the dam are not relevant.

The hazard classification assigned to a dam should be based on the worst-case failure condition. In most cases, the hazard classification can be determined by field investigations and a review of topographic maps. However, when the hazard classification is not apparent from a field reconnaissance, it will be necessary to perform dambreak studies to clearly identify the consequences of a dam failure. The dambreak studies must consider the incremental effects of failure under both normal and flood flow conditions up to the point where a dam failure would not significantly increase the hazard to life or amount of property damage. This may require evaluation of failure during floods up to the probable maximum flood. The flood wave should be routed downstream to the point where a failure will no longer constitute a hazard. The results of the downstream routing should be clearly shown on inundation maps with the breach wave travel time indicated at critical downstream locations. The inundation maps should be developed at a scale sufficient to identify downstream habitable structures within the impacted area. Dambreak studies should be performed in accordance with one or more of the techniques presented in Appendix IIA.
2-3.1.2 Determining Area Affected

The area affected is the area inundated by the incremental increase in flooding levels caused by dam failure over natural flooding with the dam in place. Key elements in determining the area affected by a flood wave, resulting from a theoretical dam breach, are the height of the flood wave and the length and width of river reach over which it will persist. An associated item of concern is the flood wave travel time. These elements are primarily a function of the rate and extent of dam failure, but also are functions of channel and flood plain geometry, hydraulic head at the dam, volume of stored water and reservoir inflow, and the size, shape, and rate of the dam breach. Special cases where dam failure could cause domino-like failure of downstream dams resulting in a cumulative flood wave large enough to cause a hazard should also be considered. All of these factors should be considered when delineating the area impacted by dam failure. The area affected by potential flooding is defined from downstream flood profiles determined by hydraulic computational methods. Additional guidance on inundation mapping may be found in the Commission's Revised Emergency Action Plan Guidelines issued April 5, 1985 (Attached as Appendix IIIB). 1/

2-3.1.3 Evaluating Impacts of Dam Failure

The hazards created by dam failure include loss of human life, damage to national security installations, and social, environmental, and economic impacts.

(1) Loss of human life - The following factors should be evaluated when estimating the potential for fatalities resulting from dam failure.

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

1/ (Note: Page 21 was revised on March 18, 1987 and is included in Appendix IIIB).
- Type of flow conditions based on water depths, temperatures and velocities, rate of rise of the flood wave, duration of floodflow, and special hazardous conditions such as the presence of surface waves, debris flow or terrain conditions which may increase potential for loss of lives.

(2) Social impacts - In addition to the loss of life, social impacts which are important but cannot easily be evaluated in economic terms include: impacts on real income; impacts on health and well being of individuals including physiological and physical injury; impacts on communities (both family and larger community) including destruction of educational, historic, and cultural facilities and values; and general disruption of a way of life.

(3) Environmental impacts - Environmental impacts which are also important but cannot be easily evaluated in economic terms include: damage to wildlife and habitat; reduced visual qualities; damage to recreation, fishery, and riparian habitat; and extreme changes in the channel regime.

(4) Economic impacts - Evaluate damages to: residences; commercial property; industrial property; public utilities and facilities including transmission lines and substations; transportation systems; agricultural buildings, lands, and equipment; dams; and loss of production and other benefits from project operation.

2-3.1.4 Defining the Hazards

The degree of study required to sufficiently define the impacts of dam failure for selecting an appropriate IDF will vary with the extent of existing and potential downstream development, the size of reservoir (depth and storage volume), and type of dam. Evaluation of the river reach and areas impacted by a dam failure should proceed only until sufficient information is generated to reach a sound decision or until a good understanding of the consequences of failure is reached. In some cases, it may be apparent from a field inspection or an inspection of aerial photographs and topographic maps that loss of life and extensive economic impacts attributable to dam failure would occur and be unacceptable.

The hazard created by a dam is determined by comparing the impacts with the dam failing to those from the same flood without dam failure and the dam remaining in place.
The failure of a dam during a particular flood may increase the area flooded and also alter the flow velocity and depth of flow as well as the rate of rise of flood flows. These changes in flood flows could also affect the amount of damage. To fully evaluate the hazard created by a dam, a range of flood magnitudes needs to be examined. The upper limit for this range is the PMF.

Water surface profiles, flood wave travel times, and rates of rise should be determined for each condition. Because computations of dambreaks and flood routings are not precise, a conservative evaluation of the hazards should be made. As noted previously, computations should include sensitivity analyses and be carried only as far as necessary to reach a decision.

2-3.1.5 Selecting the Inflow Design Flood

Selecting an IDF for the design of a dam requires consideration of the consequences of dam failure. The consequences of failure may include loss of life, damage to national security installations, and social, environmental, and economic impacts.

The PMF should be adopted as the IDF in those situations where consequences attributable to dam failure for flood conditions less than the PMF are unacceptable. The determination of unacceptability exists when the area affected is evaluated and factors in section 2-3.1.3 indicate loss of human life, damage to known national security installations, extensive property and environmental damage, or serious social impact which may be expected as a result of dam failure (see qualification in following subparagraphs).

A flood less than the PMF may be adopted as the IDF in those situations where the consequences of dam failure are acceptable. In other words, where detailed studies conclude that the risk is only to the dam owners' facilities and no increased damage to downstream areas is created by failure, a risk-based approach is acceptable. Acceptable consequences exist when evaluation of the area affected and factors in section 2-3.1.3 show one of the following conditions:

- There are no permanent human habitations, or known national security installations, commercial or industrial development, nor are such habitations, or commercial or industrial developments projected to occur within the potential hazard area in the foreseeable future and the transient population is not expected to be endangered.
There are only a few permanent human habitations within the potential hazard area that would be affected by failure of the dam and there would be no significant incremental increase in the hazard resulting from the occurrence of floods larger than the proposed IDF up to the PMF. For example, where the impoundment storage is small and failure would not add appreciable volume to the outflow hydrograph, and, consequently, the downstream inundation would be essentially the same with or without failure of the dam. The consequences of dam failure may not be acceptable if the hazard to these habitations is increased appreciably by the failure flood wave or level of inundation. When a dambreak analysis shows downstream incremental effects of approximately two feet or more, engineering judgment and further analysis will be necessary to finally evaluate the need for modification to the dam. The two foot increment is not an absolute decision-making point. Sensitivity analyses are also a tool that should be used in making such judgments.

In addition to the conditions listed in section 2-3.1.3, the selected magnitude of the IDF should be based on the following special considerations:

- Dams which provide vital community services such as municipal water supply or energy may require a high degree of protection against failure to ensure those services are continued during and following extreme flood conditions when alternate services are unavailable. If the economic risk of losing such services is acceptable, the IDF can be less conservative. However, loss of water supply for domestic purposes may not be an acceptable public health risk.

- Dams identified as having a low hazard potential should be designed to not less than some minimum standard to protect against the risk of loss of benefits during the life of the project; to hold O&M costs to a reasonable level; to maintain public confidence in owners and agencies responsible for dam safety; and to be in compliance with local, State, or other regulations applicable to the facility. Flood frequency and risk base analyses may be used for this analysis. Generally, it would not be an appropriate risk to design a dam having a low hazard potential for a flood frequency of less than 100 years.
2-3.2 Probable Maximum Floods for Dam Safety

2-3.2.1 General

A deterministic approach should be used to determine the PMF. The deterministic approach may be said to be a process by which a flood hydrograph is generated by modeling the physical atmospheric and drainage basin hydraulic and hydrologic processes. The method attempts to represent the most severe combination of meteorologic and hydrologic conditions considered reasonably possible for a given drainage basin. In other words, the PMF represents an estimate of the upper limit of run-off that is capable of being produced on the watershed.

In modeling the PMF phenomena, the salient features of the rainfall-runoff process are modeled separately and then combined to form the flood hydrograph. These features include the probable maximum precipitation, antecedent precipitation, snowmelt, infiltration and surface detention losses, base flow, interflow, and the conversion of rainfall available for surface runoff into a flood hydrograph.

A PMF is developed from various combinations or sequences of events (rainstorms and snowmelts), depending upon the climatology of the watershed. The appropriate sequence of meteorologic events for defining the appropriate PMF will vary with climatic region and season. In those watersheds where snowmelt is not a consideration, the principal storm produces the PMP on the watershed with lesser storms occurring antecedent and/or subsequent to it. The occurrence of this sequence of events is called the PMF series and is normally assumed because of the persistent nature of severe meteorologic events in most regions. The antecedent storm depths and time intervals between storms will vary with season, type of storm, and region. In those watersheds where snowmelt contributes significantly in producing flood-flows, the appropriate PMF may result from the PMP occurring on an appropriate snowpack or precipitation falling on a probable maximum snowpack.

Any combination of events considered should be meteorologically compatible with the occurrence of the probable maximum event, rainfall or snowmelt. The magnitude and timing of the antecedent event should be consistent with the definition of conditions that could occur.
Discharges from upstream reservoirs should be included as part of the inflow hydrographs. When storms serve as a basis for determining inflows, the storm centering pattern that is applicable to the site being evaluated should be adopted.

2-3.2.2 Probable Maximum Precipitation (PMP)

The concept that the PMP represents an upper limit to the level of precipitation the atmosphere can produce has been stated in many hydrometeorological documents. At times, it is necessary to revise PMP estimates because of new or additional storm data, increased understanding of severe storm meteorology, or developments in techniques.

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

In orographic regions, where local influences affect the delineation of meteorological homogeneity, transposition is generally not permitted. Alternative procedures are offered for these regions that are less reliant on the adequacy of the storm sample. Most of these procedures involve development of both non-orographic and orographic components (sometimes an orographic intensification factor is used) of PMP. Orographic and non-orographic PMP's are then combined to obtain total PMP estimates for an orographic basin. To date, no single orographic procedure has been developed that offers universal applicability. These techniques have been discussed at length in various National Weather Service (NWS) reports (see Fig. 2.1) and in the Manual for Estimation of PMP (WMO, 1973). Currently, PMP estimates are available for the entire conterminous United States, as well as Alaska, Hawaii, and Puerto Rico (see Figure 2.1).

See Appendix IIC for guidelines adopted by FERC staff for proper use of Hydrometeorological Report (HMR) Nos. 51 and 52 vs. HMR No. 33.
Figure 2-1. —Regions covered by generalized PMP reports (HMR = Hydrometeorological Report, TP = Technical Report,) see references for complete listing.
Almost all discussions of PMP lead to the conclusion that the PMP estimate represents an upper bound to precipitation potential. In practical applications, the PMP estimate that is determined for a particular basin is recommended by the NWS as the best estimate of this theoretical upper bound, considering the available data and techniques. It is expected, nevertheless, that as our understanding and data increase, these "particular" PMP estimates may require adjustment in order to better represent the conceptual PMP.

2-3.2.3 Probable Maximum Precipitation on Snow

For drainage areas where snowmelt contributes significantly to the controlling flood situation, two conditions shall be considered for antecedent snowpack accumulation. They are probable maximum precipitation on snow and probable maximum snowpack with rain.

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. 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.

The time distribution of temperatures shall be fixed by the adopted time distribution of the rain; i.e., the highest temperature is made to coincide with the highest period of rainfall.

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 containing the maximum 100-year snowpack to assure that the controlling combination will be determined.

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 maximimation, an envelope of the maximum free air wind with adjustments during major storm periods shall be used. Adjustments shall include

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2/ The substance of this section is taken from ANSI/ANS-2.8-1981, "Determining Design Basis Flooding at Power Reactor Sites." Permission to reprint portions of this publication was granted by the American Nuclear Society by letter dated February 25, 1987.
seasonal, latitudinal variations, and reductions to velocities which would be expected at the surface of the snowpack. The time distribution of windspeed should follow the same pattern as the temperature and the probable maximum precipitation.

For the case of PMP 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 PMP are temperature and wind.

2.3.2.4 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.

The physical limit of snowpack water equivalent will usually occur during the spring snowmelt season. This can be estimated by detailed studies of the total winter season PMP, with assumed conservative percentages of moisture 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. The snow wedge so derived represents the probable maximum flood producing snowpack.

The time distribution of coincident 100-year rainfall on probable maximum snowpack shall include the 100-year occurrence for all time periods. The duration of the rainfall should be based on the regional characteristics of the basin. The maximum rainfall rates of the 100-year rainstorm shall be timed so that the rain begins at the peak of the snowmelt flood.

The effects of frozen ground on run-off should be considered since this condition can greatly increase the runoff potential and the resulting flood flows. Air temperatures coincident with snowmelt periods should be based on an envelope of the highest recorded temperature of the region during snowmelt periods. Dewpoint temperatures are used to represent vapor pressures to compute the convection-condensation component of snowmelt equations. In the absence of recorded dewpoint temperatures, the dewpoint may be assumed to equal air temperature during rain periods. In rainless periods, minimum air temperatures may be considered equal to dewpoint temperatures.

3/ The substance of this section is taken from ANSI/ANS-2.8-1981, "Determining Design Basis Flooding at Power Reactor Sites." Permission to reprint portions of this publication was granted by the American Nuclear Society by letter dated February 25, 1987.
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.

Reflectivity, or albedo, values vary from 80 percent for new fallen snow to 40 percent for melting, late-season snow. The time distribution may be assumed linear and progressively decreasing during snowmelt. The wind values typical of the region should be used; adjustments for topographic features of the watershed may be necessary.

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

2-3.2.5 Precipitation Losses and Precipitation Excess

When computing the PMF, it is the general practice in most cases to assume that an antecedent storm of sufficient magnitude has reduced water losses due to abstractions such as interception, evaporation, surface depression storage, and interflow to negligible levels. They can then be eliminated from further consideration. However, in areas that include geologic features such as glacial potholes, lava bases and karst topography, antecedent events may not be of sufficient magnitude to satisfy this loss. Similarly, in cases where geologic phenomena, such as underlying impervious strata overlain by relatively coarse superficial material occur, the precipitation may infiltrate rapidly to the impervious strata and then flow horizontally as interflow to a defined watercourse. If the drainage network is sufficiently dense (a short subsurface flow distance to a defined watercourse) or the formation is extremely permeable (e.g., limestone), the contribution of interflow to the flood may be of significant proportions. Interflow must be considered on a site-specific basis.

For the purpose of determining the appropriate infiltration rates for use in PMF determinations, it is the practice to assume that antecedent storm events of sufficient magnitude have occurred to satisfy losses associated with the higher and steeper part of the infiltration curve and to reduce infiltration rates to their ultimate long-term values. This antecedent storm establishes the base flow for routing the PMF.

The estimated losses should be appropriate for the season. Anticipated land use and consideration of events should collectively be considered. Soil group members derived from agricultural soil maps can be incorporated.
Once the basin-averaged net or excess precipitation has been determined, it must be converted into a flood hydrograph representing surface runoff at the catchment concentration point. Although there are a number of techniques or models available for accomplishing this, the unit hydrograph technique is the basic tool employed by the Federal construction and regulatory agencies.

The unit hydrograph is defined as the hydrograph of one inch of direct runoff that results from one inch of precipitation excess distributed uniformly over the basin and occurring at a constant rate over a specified unit duration. The response to a storm of unit duration, but non-unit depth, is the product of the unit hydrograph and the storm's precipitation excess. A complex storm can be represented as a sequence of unit-duration storm segments. The response to this storm is the sum of the sequence of corresponding individual storm-segment responses. Flood studies submitted to the Commission should include the flood hydrograph, as illustrated in Figure 2-2.

In some cases, a stream gage may be located at the basin's concentration point. Perhaps a significant flood event has been recorded at the gage and associated storm rainfall data collected. If such is the case, an appropriate unit hydrograph can be developed by reconstituting the observed flood or flood events. The PMF may then be determined by application of the unit hydrograph (based on observed data) to the precipitation excess, resulting from occurrence of the PMP over the basin.

In many cases, however, no flood records exist for the drainage basin being studied and what is loosely termed a "synthetic unit hydrograph" must be developed. Fortunately, a large number of reconstitution studies of actual floods has been completed. These studies have allowed the compilation of unit hydrographs for a wide variety of basins representing diverse climatic, topographic, vegetal, and developmental conditions. For a given condition, or combination of conditions, the salient features of the available unit hydrographs can be correlated with measurable physical parameters, e.g., drainage area, slope length of watercourse, etc.

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 sub-areas on the basis of size, drainage pattern, installed and proposed regulation facilities, vegetation, soil and cover type, and precipitation characteristics. These sub-area runoff models are connected and combined by streamcourse modeling and routed through the project reservoir. In general, sub-areas should not exceed 1000 square miles in area.
Figure 2-2. — Demonstration of principle of superposition for a typical basin.
Deterministic simulation models including unit hydrographs should be verified or calibrated by comparing results of the simulation with historic floods of record for which suitable precipitation data are available. If available, it is recommended that at least three of the largest floods of record be used for calibration.

For ungaged basins, the 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.

There is no literal way to verify a stream course model during the probable maximum flood conditions. However, a model derived from valley dimensions and plausible elevation-flow relationships, which has successfully reproduced historical events, stands on its own. Discharges from upstream reservoirs must be included in the stream course model. The lag times and times of concentration should also be identified and evaluated for reasonableness.

2-3.3 Flooding to Protect Against Loss of Benefits During the Life of the Project - Applicable Only to Low Hazard Dams

The inflow design flood for low hazard dams may be based on a percentage of the probable maximum flood or be determined by a flood frequency analysis. The magnitude of the IDF should provide adequate protection against the loss of benefits during the life of the project and be consistent with acceptable economic risks to the owner of the dam only. Generally, it would not be an appropriate risk to design a dam having a low hazard potential for a flood frequency of less than 100 years.

2-4 Accommodating Inflow Design Floods

2-4.1 Flood Routing Guidelines

2-4.1.1 General

Site-specific considerations should be used to establish flood routing criteria for each dam and reservoir. The criteria for routing the IDF should be consistent with the reservoir regulation procedure that is to be followed in actual operation. General guidelines to be used in establishing criteria follow:
2-4.1.2 Guidelines for Initial Elevations Based on Storage Allocation

- If there is no allocated or planned flood control storage (e.g. run-of-river), the flood routing usually begins with the reservoir at the normal maximum pool elevation. If regulation studies show that pool levels would either be higher or lower than the normal maximum pool elevation during the critical IDF season, then the results of those specific regulation studies should be analyzed to determine the appropriate initial pool level for routing the IDF.

- If a project has flood storage space allocated, the flood routing usually begins with the reservoir at the normal maximum pool elevation and an antecedent storm is included in the routing procedure. If the antecedent flood the IDF is not included in the routing, a conservative initial pool level should be selected based on hydrologic considerations. In special cases, pool levels higher or lower than the normal maximum pool level may be selected when results of regulation studies clearly show such conditions would prevail during the critical IDF season.

2-4.1.3 Initial Reservoir Elevation

Initial pool levels for routing runoff through reservoirs should be based on hydrologic characteristics of the basin and project operational considerations. The assumed initial levels should be reasonable for the season when the IDF may occur and generally should be based on reservoir system regulation studies which reflect basin runoff conditions and releases for specific project purposes. In some cases, it may be necessary to examine seasonal variations in pool levels and the corresponding seasonal IDF to determine maximum reservoir levels. Regulation rule curves and forecasting procedures established for the forecasted components of the IDF (usually only the snowmelt portion) may be considered when establishing initial reservoir elevations. Hydrologic characteristics and reservoir regulation objectives will vary significantly from one reservoir or reservoir system to the next.

2-4.1.4 Reservoir constraints

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

- Topographic limitations on reservoir stage which exceed the economic limits of dike construction.
- Public works around the reservoir rim which are not to be relocated, such as water supply facilities and sewage treatment plants.
- Dwellings, factories, and other developments around the reservoir rim which are not to be relocated.
- If there is a loss of unusual storage capacity caused by sediment accumulation in the portions of the reservoir, then this factor should be accounted for in routing the IDF. Deposition in reservoir headwater areas may build up a delta which can increase flooding in that area, as well as reduce flood storage capacity, thereby having an effect on routings.
- Geologic features such as terrain that may become unstable when inundated and result in landslides which would threaten the safety of the dam, domestic and/or other developments, or displace needed storage capacity.

2-4.1.5 Reservoir Regulation Requirements

Considerations to be evaluated when establishing flood routing criteria for a project include (1) regulation requirements to meet project purposes; (2) the need to impose a maximum regulated release rate to prevent flooding or erosion of downstream areas and control rate of drawdown; (3) the need to provide a minimum regulated release capacity to recover flood control storage for use in regulating subsequent floods; and (4) the practicability of evacuating the reservoir for emergencies and for performing inspection, maintenance, and repair.

Spillways, outlet works, and penstocks for powerplants are sized to satisfy project requirements and must be operated in accordance with specified regulating instructions when the reservoir is within the allocated storage levels. These facilities should be relied upon to make flood releases subject to the following limitations:
Only those release facilities which can be expected to operate reliably under the assumed flood condition should be assumed to be operational for flood routing. Reliability depends upon structural competence and availability for use. Availability of a source of auxiliary power for gate operation, accessibility of controls, design limits on operating head, reliability of access roads, and availability of operating personnel at the site during flood events are factors to be considered in determining whether to assume release facilities are operational.

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

A specific evaluation of expected flood conditions should be made before penstocks for powerplants are considered available for making flood releases. Grounded transmission lines, loss of substations, and powerload interruptions could prevent operation of turbines. All units can be considered available for operation but if there is only one unit, it should be considered unavailable, unless equipped with a bypass.

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

In flood routing, assumed releases are generally limited to maximum values determined from project uses, by availability of outlet works, tailwater conditions including effects of downstream tributary inflows and wind tides, and downstream nondamaging discharge capacities until allocated storage elevations are exceeded. When a reservoir's capacity in regulating flows is exceeded, then other factors, particularly dam safety, will govern releases.

During normal flood routing, the rate of outflow from the reservoir should not exceed the rate of inflow until the maximum inflow occurs, nor should the maximum rate of increase of outflow exceed the maximum rate of increase of inflow. This is to prevent outflow conditions from being more severe than pre-dam conditions. An exception to the preceding would be the case where streamflow forecasts are available and high early flood releases could serve to reduce maximum flood releases.
2-4.2 **Spillway and Flood Outlet Selection and Design**

**2-4.2.1 General**

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

**2-4.2.2 Gated or Ungated Spillways**

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

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

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

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

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

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

3. **Operation and maintenance.** - Gated spillways may experience more operational problems and are more expensive to maintain than ungated spillways. Constant attendance or several inspections per day by an operator during high water levels is highly desirable for reservoirs.
with gated spillways, even when automatic or remote controls are provided, and should be required during periods of major flood inflows where automatic or remote controls are not provided. Gated spillways are more subject to clogging from debris and jamming from ice, whereas, properly designed ungated spillways are basically free from these problem. Gated spillways require regular maintenance, and, as a minimum, an annual operation test for safety purposes.

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

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

(6) Emergency evacuation. - Ungated spillways cannot be used to evacuate a reservoir during emergencies. The capability of gated spillways to draw down pools from the top of the gates to the spillway crest can be an advantage when emergency evacuation to reduce head on the dam is a concern.

(7) Economics and selection. - Designs to be evaluated should be technically adequate alternatives. Economic considerations often indicate whether to select either gated or ungated spillways. The possibility of selecting a combination of more than one type of spillway is also a consideration. Final selection of the type of crest control should be based on a comprehensive analysis of all pertinent factors, including advantages, disadvantages, limitations, and feasibility of options.

2-4.2.3 Design Considerations

Dams and their appurtenant structures should be designed to give satisfactory performance and to practically eliminate the probability of failure. These guidelines identify three specific classifications of spillways (service, auxiliary, and emergency) and outlet works that are used to pass floodwaters, each serving a particular function. The following paragraphs discuss functional requirements.
Service spillways should be designed for frequent use and should safely convey releases from a reservoir to the natural watercourse downstream from the dam. Considerations must be given to waterway freeboard, length of stilling basins, if needed, and amount of turbulence and other performance characteristics. It is acceptable for the crest structure, discharge channel (e.g., chute, conduit, tunnel), and energy dissipator to exhibit marginally safe performance characteristics for the IDF. However, they should exhibit excellent performance characteristics for frequent and sustained flows such as up to the 1 percent chance flood event. Other physical limitations may also exist which have an effect on spillway sizing.

It is acceptable for auxiliary spillways that are designed for infrequent use to sustain limited damage during passage of the IDF. The design of auxiliary spillways should be based on economic considerations and be subject to the following requirements:

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

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

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

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

- The structural integrity of the dam should not be jeopardized by spillway operation.
- Large conservation storage volumes should not be lost as a result of degradation of the crest during operation.

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

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

2-4.3 Freeboard Allowances

2-4.3.1 General

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

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

2-4.3.2 Freeboard Guidelines

Following are guidelines for determining appropriate freeboard allowances.

- Freeboard allowances should be based on site-specific conditions and type of dam (concrete or embankment).
- Both normal and minimum freeboard requirements should be evaluated in determining the elevation of the top of dam. The resulting higher top of dam elevation should be adopted for design.

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

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

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

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

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

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

- Freeboard allowance for wave and volume displacement due to potential landslides which cannot be economically removed or stabilized should be considered if a reservoir is located in a topographic setting where the wave or higher water resulting from displacement may be destructive to the dam or may cause serious downstream damage.
Total freeboard allowances should include only those components of freeboard which can reasonably occur simultaneously for a particular water surface elevation. Components of freeboard and combinations of those components which have a reasonable probability of simultaneous occurrence are listed in the following paragraphs for estimating minimum, normal, and intermediate freeboards. The top of the dam should be established to accommodate the most critical combination of water surface and freeboard components from the following combinations.

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

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

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

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

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

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

(1) Combination 1
   (a) Wind-generated wave runup and setup for maximum wind, and
   (b) Settlement of embankment and foundation not included in camber.

(2) Combination 2
   (a) Landslide-generated water waves and/or displacement of reservoir volume;
   (b) Settlement of embankment and foundation not included in camber; and
(c) Settlement of embankment and foundation or seiches as a result of the occurrence of the maximum credible earthquake.

For intermediate freeboard combinations, in special cases, a combination of intermediate winds and water surface between normal and maximum levels should be evaluated to determine whether this condition is critical. This may apply where there are exclusive flood control storage allocations.
2-5. References


APPENDICES
APPENDIX IIA

Dambreak Flood Studies

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

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

When routing a dambreak flood through the downstream reaches appropriate local inflows should be included in the routing, consistent with the assumed storm centerings.

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

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

I. REFERENCES

Suggested acceptable references regarding dam failure studies include the following:

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

C. Fread, D. L. "DAMBRK - The NWS Dam-Break Flood Forecasting Model," National Weather Service, Silver Spring, Maryland, July 18, 1984 (Reprinted February 1987). (This is the preferred method for performing dambreak studies).


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


II. CRITERIA

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

A. If the dambreak analysis has been performed by an acceptable method (Reference C is the preferred method), then generally only the breach parameter, peak discharge, and flood wave travel time should be verified as an indicator of the licensee's correct application of the method selected. Downstream routing parameters (i.e., Manning's "n") should be reviewed for acceptability and inundation maps should be reviewed for clarity and completeness of information (i.e., travel times). The following criteria are considered to be adequate and appropriate for verifying the selected breach parameters and peak discharge:
1. Breach Parameter - Most serious dam failures result in a situation resembling weir conditions. Breach width selection is judgmental and should be made based on the channel or valley width with failure occurring at the deepest section. The bottom of the breach should generally be assumed to be at the foundation elevation of the dam. Pages 8, 8A, 8B, and 9 of this appendix contain suggested breach parameters and should be used when verifying the selected breach parameter. For worst case scenarios, the breach width should be in the upper range while the time of failure should be in the lower range. However a sensitivity analysis is recommended to determine the reasonableness of the assumptions.

2. Peak Discharge - The peak discharge may be verified by use of equations (11) and (13) of Reference No. 1. Although the equations assume a rectangular-shaped breach, a trapezoidal breach may be analyzed by specifying a rectangular breach width that is equal to the average width of the trapezoidal breach.

**Equation 11:**

\[ C = \frac{23.4 \, A_s}{\overline{B_r}} \]

Where \( C \) = constant

\( A_s \) = reservoir surface area, in acres

\( \overline{B_r} \) = average breach width, in feet

**Equation 13:**

\[ Q_{b\text{max}} = 3.1 \, \overline{B_r} \left( tf + \frac{C}{C \sqrt{H}} \right)^3 \]

Where \( Q_{b\text{max}} \) = maximum breach outflow, in cfs

\( tf \) = time of failure, in hours

\( H \) = maximum head over the weir, in feet

This equation for \( Q_{b\text{max}} \) has been found to give results within +5% of the \( Q_{\text{peak}} \) from the full DAMBRK model.

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

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

\[ Q_{b\text{max}} = Q_0 + 3.1 \text{ Br} \left( \frac{C}{t^f + \frac{C}{v^2}} \right)^3 \]

Where \( Q_0 \) = Additional (non-breach) outflow (cfs) at time \( t_f \) (i.e., spillway flow and/or crest overflow) (optional data value, may be set to 0).

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

3. Flood Wave Travel Time

Reasonableness of the flood wave travel time may be determined by use of the following "rule-of-thumb" approximation for average wave speed:

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

(b) Assume a constant average channel slope

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

\[ d = \left( \frac{Q_n}{1.49 B S^{0.5}} \right)^{0.6} \]

Where: \( d \) = depth of flow for assumed rectangular section, ft.

\( Q \) = peak discharge, cfs

\( B \) = average width (rectangular), ft.

\( S \) = average slope, ft/ft

\( n \) = Manning's roughness coefficient

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

\[ v = \frac{1.49}{n} (S)^{0.5} (d)^{0.67} \]
Where: $V =$ average velocity, fps

(e) Compute wave speed, $C$ (kinematic velocity):

$$C = \frac{5}{3} V (0.68)$$

Where: $C =$ wave speed (mph)

(f) Determination travel time, $TT$

$$TT = \frac{x}{C}$$

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

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

$$0.5 C = 2(S)$$

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

In addition, as a "rule-of-thumb", the dynamic routing (NWS) method should be used whenever severe backwater conditions at downstream areas occur and/or the slope is less than 20 ft/mi. Otherwise, normal hydrologic routing (HEC-1) is usually acceptable. It is recommended that HEC-2 be used to determine the resulting water surface elevations when HEC-1 is used for the dambreak study.

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

1. Unsteady Flow Method (Recommended)

"The NWS Simplified Dam-Break Flood Forecasting Model" (Reference A) and "DAMBRK" (Reference C) are the recommended methods. As the flood wave travels downstream, the peak discharge generally, but not always, decreases along with the wave velocity. This attenuation in the flood wave is primarily due to energy dissipation when it is near the dam and to valley storage as it progresses in an unsteady flow downstream. For all parameters calculated, this method should usually give results within $\pm 10\%$ of the parameters determined from using the full DAMBRK model.
2. **Steady Flow Method (Provides a rough estimate)**
   
   If this method is selected, the breach parameter and peak discharge shall be verified as in part "A" above.

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

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

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

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

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

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

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

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

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

      (3) Assume Q max remains constant throughout the entire stream length under consideration.

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

      (5) Using the continuity equation to determine the velocity, estimate the travel time between each cross-section.
C. The above criteria for breach parameter, peak discharge, depth of flow, and travel time should provide the necessary "ballpark figures" needed for comparison with licensee's estimates. When large discrepancies in compared values exist, or questions arise about assumptions to be made, or it appears that an extensive review will be necessary, the Regional Director should contact the Washington Office, DINS for guidance. It should be emphasized that the intent is to review and not to make an independent analysis and that normally the studies submitted by the dam owner need not be verified if the methodology used appears to be satisfactory. The methodology is construed to be a part of the study and should be requested if not included.
### Table 1

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

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Type of Dam</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Average width of Breach (BR)</strong></td>
<td><strong>HD ≤ BR ≤ 5HD</strong> (usually between 2HD &amp; 4HD)</td>
<td>Earthen, Rockfill</td>
</tr>
<tr>
<td><em>(See Comment No. 1)</em></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>BR ≥ 0.8 x Crest Length</strong></td>
<td></td>
<td>Slag, Refuse</td>
</tr>
<tr>
<td><strong>BR = Crest Length</strong></td>
<td></td>
<td>Concrete, Arch, Timber Crib</td>
</tr>
<tr>
<td><strong>BR = Width of 1 or More Monoliths, usually BR ≤ 0.5 W</strong></td>
<td></td>
<td>Masonry, Gravity</td>
</tr>
<tr>
<td><strong>Horizontal Component of Side Slope of Breach (Z)</strong></td>
<td>0 ≤ Z ≤ 2</td>
<td>All</td>
</tr>
<tr>
<td><em>(See Comment No. 2)</em></td>
<td></td>
<td>Masonry, Concrete Timber Crib</td>
</tr>
<tr>
<td><strong>Z = 0</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>1/4 ≤ Z ≤ 1</strong></td>
<td></td>
<td>Earthen (Engineered, Compacted)</td>
</tr>
<tr>
<td><strong>1 ≤ Z ≤ 2</strong></td>
<td></td>
<td>Slag, Refuse (Non-Engineered)</td>
</tr>
<tr>
<td><strong>0 &lt; Z &lt; slope of valley walls</strong></td>
<td></td>
<td>Arch</td>
</tr>
<tr>
<td><strong>Time to Failure (TFH)</strong></td>
<td>0.1 ≤ TFH ≤ 1</td>
<td>All</td>
</tr>
<tr>
<td><em>(in hours)</em></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>TFH ≤ 0.1</strong></td>
<td></td>
<td>Arch, Timber Crib</td>
</tr>
<tr>
<td><strong>0.1 ≤ TFH ≤ 0.3</strong></td>
<td></td>
<td>Masonry, Concrete</td>
</tr>
<tr>
<td><strong>0.1 ≤ TFH ≤ 1.0</strong></td>
<td></td>
<td>Earthen (Engineered, Compacted)</td>
</tr>
<tr>
<td><strong>0.1 ≤ TFH ≤ 0.5</strong></td>
<td></td>
<td>Earthen (Non Engineered, Poor Construction)</td>
</tr>
<tr>
<td><strong>0.1 ≤ TFH ≤ 0.3</strong></td>
<td></td>
<td>Slag, Refuse</td>
</tr>
</tbody>
</table>

**Definition:**
- **HD** - Height of Dam
- **Z** - Horizontal Component of Side Slope of Breach
- **BR** - Average Width of Breach
- **TFH** - Time to Fully Form the Breach
- **W** - Crest Length

**Comments:** See page 3A-88
Comments:

1. BR is the average breach width, which is not necessarily the bottom width. BR is the bottom width for a rectangle, but BR is not the bottom width for a trapezoid.

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

3. Time to failure is a function of height of dam and location of breach. Therefore, the longer the time to failure, the wider the breach should be. Also, the greater the height of the dam and the storage volume, the greater the time to failure and average breach width will probably be.

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

5. Breach width assumptions should be based on the height of the dam, the volume of the reservoir, and the type of failure (e.g. piping, sustained overtopping, etc.).

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

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

      1. Assume a probable (reasonable) maximum breach width, a probable minimum time to failure, and a probable maximum manning's "n" value. Mannings's "n" values in the vicinity of the dam (up to several thousand feet or more downstream) should be assumed to be larger than the maximum value suggested by field investigations in order to account for uncertainties of high energy losses, velocities, turbulence, etc., resulting from the initial failure.
Comments:

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

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

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

1. Use criteria in a. 1.

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

Plot the results of both runs on the same graph showing the changes in travel time with respect to distance downstream from the dam.

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

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

2. Use criteria in a. 1.

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

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

REVISED EMERGENCY ACTION PLAN GUIDELINES
Pursuant to the authority in Section 12.22(a)(1) of the Commission's Regulations, the Director, Office of Hydropower Licensing, has revised the guidelines for the preparation of emergency action plans (EAP). The guidelines have been revised to facilitate the preparation, annual review and updating of EAP's to ensure their effectiveness and workability. The guidelines should be used in conjunction with the instructions contained in Part 12, Subpart C of the Commission's Regulations.

Owners/developers (herein referred to as owners) of all dams under Commission jurisdiction must develop and file an EAP with the Regional Engineer unless an exemption is obtained pursuant to Section 12.21 of the Regulations. All required EAP's developed subsequent to the date of this notice must follow the format established in the revised guidelines. Owners are not required to rewrite and resubmit existing EAP's in accordance with the established format. However, as part of the annual review and updating process, owners should determine whether their EAP's can be enhanced based on the information in the revised guidelines and, therefore, urged to consider reorganizing their EAP's in the format described therein.

Copies of the revised guidelines are available from the Director, Division of Inspections or the Regional Engineer.

Kenneth F. Plumb
Secretary

GUIDELINES FOR PREPARATION OF EMERGENCY ACTION PLANS

1. Purpose. These guidelines are for the purpose of defining and implementing the requirements of an acceptable emergency action plan (EAP) and for facilitating its preparation and annual testing and update. An emergency is defined as an impending or actual sudden release of water caused by an accident to, or failure of, project structures.

2. Background. The "Guidelines for Preparation of Emergency Action Plans" were established in November 1979. The guidelines were subsequently included as the Appendix to Order No. 122 of the Commission's Regulations, issued January 21, 1981. Section 12.22(a)(1) of that Order states that "an emergency action plan must conform with the guidelines established, and from time to time revised, ...." The demand for more specific, comprehensive guidelines resulted in this revision to the guidelines.

3. Scope. The guidelines establish a specific format (see Item 4) to assist in preparing an effective, workable EAP. The format was developed to include all pertinent information required for the EAP itself and its accompanying appendix. In providing the appropriate information, the EAP should be site specific, reflecting mode of operation, internal and external means of communication, and interaction with appropriate agencies and owners of other sites. The format should be used in conjunction with the instructions contained in Part 12, Subpart C of the Commission's Regulations.

It is not imperative that current EAP's be revised to comply with this format. However, each EAP should contain a discussion of the method for implementing the notification plan in a timely manner. It is recommended that the owners use these guidelines to facilitate the annual review and updating of the workability of an EAP. The Regional Engineer is to be advised of the results of this annual review prior to December 31 of each year.
Under the provisions of Section 12.22(c) of the Commission's Regulations, each owner of a hydroelectric project under the jurisdiction of the Commission with operating or other personnel located within a 10-mile radius of a nuclear power plant reactor must prepare a radiological emergency response plan to be implemented in the event of a severe accident or incident resulting in the release of radioactive materials from a nuclear plant. The guidelines for preparation of a radiological emergency response plan (see item 5) should be used in conjunction with the instructions contained in Section 12.22(c) of the Commission’s Regulations.

4. The Format. The following pages (page 3 thru 22) comprise the format by which all newly required EAP’s are to be developed. This format should facilitate the preparation, updating, and annual review of an EAP. It should be used in conjunction with the instructions contained in Part 12, Subpart C of the Commission’s Regulations. The format was developed to include all of the pertinent information to be included in the EAP as required by the Commission’s Regulations. It is recommended that the EAP be bound using a method, such as a three-ring binder, whereby outdated pages can be easily removed and replaced by updated information to ensure a complete, current, and workable plan.

Title Page

EMERGENCY ACTION PLAN

{Name} of Development

Project No. [FERC No.]

Name of the licensee/exemptee/applicant:

Address:

Submitted [date]
Verification:

State of [ ],
County of [ ], ss:

The undersigned, being first duly sworn, states that [he, she] has read the following document and knows the contents of it, and that all of the statements contained in that document are true and correct, to the best of [his, her] knowledge and belief.

(Name of person signing)

(Title)

Sworn to and subscribed before me this [day] of [Month], [year].

(Signature of Notary Public or other state or local official authorized by law to notarize documents).

SEAL

Contents of the Plan

I. Notification Flowchart
   A. Failure is imminent or has occurred
   B. Potentially hazardous situation is developing

II. General Responsibilities Under the Emergency Action Plan

III. Notification Procedures
   A. Failure is imminent or has occurred
   B. Potentially hazardous situation is developing

IV. Mitigation Activities
   A. General provisions for surveillance
   B. Surveillance at remotely controlled or unattended dams
   C. Response during periods of darkness
   D. Availability and use of alternate systems of communication
   E. Emergency supplies and resources
   F. Other concerns and actions

V. Appendix
   A. Description of the project
   B. Summary of study and analyses to determine extent of inundation
   C. Inundation Maps
   D. Plans for training, testing, and annual review
   E. Documentation
EMERGENCY ACTION PLAN

I. Notification Flowchart

- Provide a flowchart summarizing clearly who is to be notified and who is responsible for notifying which owner representative(s) and/or public official(s), and in what priority for the following emergency situations:

A. Failure is imminent or has occurred.

B. Potentially hazardous situation is developing. Situation where a failure may develop, but pre-planned actions taken during certain events (such as major floods, earthquakes, evidence of piping, etc.) may prevent or mitigate failure. Even if failure is inevitable, more time is generally available than in situation A above to issue warnings and/or take preventative actions.

- Include individual names and position titles, office and home telephone numbers, and alternate contacts and means of communication.

- The flowchart should be easy to follow under emergency conditions and should normally be limited to one page for each failure mode. Color coding may prove helpful. (Detailed information supplementing the flowchart should be provided in Section III of the emergency action plan).

- Additional copies of the flowchart should be readily available to each individual having responsibilities thereon, and should be kept up-to-date through tests and revisions.

- Note: A sample flowchart is included on page 7 of these guidelines.

II. General Responsibilities Under the Emergency Action Plan

- Advise the operators of importance of the emergency action plan (EAP) and why the EAP is necessary. Describe operators duties in implementing the EAP. Give pointers on how to communicate the emergency situation to those who need to be contacted. Include samples of typical communications.
III. Notification Procedures

* Include in the notification portion of the EAP all persons to be notified as soon as an emergency situation develops. Include individual names and position titles, location, office and phone numbers, and radio communication frequencies and call signals, if available, for owner personnel, public officials, and other personnel, including alternates. For each emergency situation, clearly indicate who is to make a call and to whom it is to be made, and in what priority.

* The number of persons to be notified by each responsible individual in the notification plan should be governed by what other responsibilities the person has been assigned.

* Describe in detail the notification procedures for the two emergency situations:

A. Failure is imminent or has occurred

B. Potentially hazardous situation is developing

Describe actions to be taken and contacts to be made. Priority of notification should address the actual emergency situation:

1. Detailed plan for notification

* Residents and owners of property that are located immediately downstream of the dam within the boundary of potential inundation where available warning time is very limited. Give names and telephone numbers (day/night), and alternate means of communication, if not applicable, so state.

* Licensee personnel. Give names and telephone numbers (day/night) of responsible individuals and alternate means of communication.

* Law enforcement officials. Give names and telephone numbers (day/night) and alternate means of communication.
Operators of other dams or water-retention facilities. Give names and telephone numbers (day/night) and alternate means of communication. If not applicable, so state.

Managers and operators of recreation facilities. Give names and telephone numbers (day/night) and alternate means of communication. If not applicable, so state.

Appropriate Federal, State and local agencies. Give names and telephone numbers (day/night) and alternate means of communication. If not applicable, so state.

Others, as appropriate. Give names and telephone numbers (day/night) and alternate means of communication.

2. Posting of the Notification Flowchart and distribution of EAP

Describe where the notification flowchart for each emergency situation will be posted. It should be in a prominent location readily accessible at the project site near a telephone and/or radio transmitter. A copy of the complete emergency action plan should also be available to the operators and dispatch center personnel.

Distribute the EAP to operational and supervisory employees and others who will be required to take certain actions when the plan is put into effect.

IV. Mitigation Activities

A. General provisions for surveillance

The EAP should contain a discussion of provisions for surveillance and detection of an emergency situation and should clearly indicate that it can be implemented in a timely manner. An important consideration in the effectiveness of the EAP is the prompt detection and evaluation of information obtained from instrumentation and/or physical inspection procedures.
Describe proper procedures to activate the alternative channels of communication. Remember that you direct these instructions to your employers.

Include any other special instructions.

E. Emergency supplies and resources

1. Stockpiling of materials for emergency use or repair.
   - Describe materials, their location and intended use.
   - Describe equipment to be used, its location and who will operate it.
   - Describe how the operator is contacted.
   - Include any other special instructions.
   - If not applicable, so state.

2. Coordination of flows
   - Describe the need for advanced coordination of flows based on weather and runoff forecasts. Include special instructions. Describe how the coordination is achieved and the chain of communication, including names and day/night telephone numbers of responsible personnel. The licensee is encouraged to coordinate with the National Weather Service (NWS) to monitor storms as well as the flood wave resulting from a dam break. The NWS may also be able to supplement the warnings being issued by using its own communication system.
   - Describe additional actions contemplated to respond to an emergency situation or failure at an unattended dam. Include periods of darkness, inclement weather, and non-business hours.

3. Alternative sources of power for spillway gate operation and other emergency uses.
   - Describe the alternative sources of power, their location, and mode of operation, and if portable, the means of transportation and routes to be followed. Include the name and day/night telephone number of the operator. If not applicable, so state.

4. Other actions devised to mitigate the extent of possible emergencies.
   - Describe other site specific actions or conditions which in your judgment will contribute to mitigation of emergencies.

- Describe actions to be taken to lower the reservoir water surface elevation. If not applicable, so state. Instruct operators on when and how this action should be taken.

- Describe actions to be taken to reduce inflow to the reservoir from the upstream dams or control structures. If not applicable, so state. Instruct operators or other persons responsible for contact with other owners on when and how this action should be taken.

- Describe actions to be taken to reduce downstream flows, such as increasing or decreasing outflows from downstream dams or control structures on the waterway on which the project is located or its tributaries. If not applicable, so state. Instruct operators on when and how this action should be taken.

- Describe any other actions to be taken.
V. Appendix

A. Description of the project

1. Provide a description of the project.

2. Provide a description of the upstream and downstream areas and topography.

R. Summary of study and analyses to determine extent of inundation

1. Identify and briefly describe the method selected to identify the inundated areas.

2. State all assumptions (including, but not limited to, reservoir inflow condition [normal or flood], temporal [time] and geometrical description of breach, Manning's "n" etc.) Several different assumptions could be made regarding the appropriate condition prevailing at the time of a dam failure.

However, it may be impractical to analyze each potential emergency which could be postulated. A fair weather dam break (reservoir at normal maximum full pool elevation, normal streamflow prevailing) is generally considered to have the most potential for loss of life. Therefore, the minimum condition to be considered is the fair weather dam break. However, failure during various flood flow conditions should be considered and should be used if they would identify special problems of flooding requiring changes and/or additions to the notification procedures. The Commission, or its authorized representative, retains the right to require, on a case-by-case basis, an investigation of other flood flow scenarios to ensure that all communities requiring notification by local officials have been identified.

3. The domino effect - a sequential failure of a multiple number of downstream dams as a result of failure of the dam for which the emergency action plan is being prepared - must be considered on a case-by-case basis. If the assumed failure of the licensee's dam would promote the failure of any dams downstream, the licensee has the responsibility to consider the domino effect in the routing of the floodwave downstream. The floodwave should be routed to the point where it no longer presents a hazard to downstream life or property (including downstream dams). Therefore, the licensee, after assuming a hypothetical failure of its dam, must assess the structural integrity of the downstream dam (based on engineering judgment or actual analysis) to determine whether it would be prudent to consider any of downstream dams to fail when subjected to the routed flood wave. If applicable, the details of domino effects should be described. If not applicable, so state.

4. Provide justification for each assumption. The assumptions made regarding the temporal and geometrical description of a breach is dependent on the type of dam being analyzed. Suggested breach parameters are included on page 21 and 22 of these guidelines.

5. Provide an elevation view of the dam and indicate the assumed breach.

6. Describe special considerations (such as reservoir slope stability).

7. Provide other helpful information, as appropriate.

8. If a dam-break analysis was made, provide results of the study. Include peak discharges and elevations and key locations, floodwave travel time to critical locations, velocity of flood wave. If a computer program was used, it may be appropriate to include a summary of the computer print-out.
9. Discuss where flood routing was terminated. It should usually be terminated where non-damaging flood levels are obtained. However, they may be terminated at a point where real-time flood warning information can be provided on a preplanned basis. (For example, if it is known that the time of failure was 12:05, p.m. and the floodwave can be monitored, it may be possible to determine that the flood wave will reach Town X at approximately 4:20 p.m.; hence, real-time flood warning information). If the flood routing is terminated before non-damaging levels are obtained, describe in detail the plan for establishing and implementing real-time flood warning information.

C. Inundation Maps

* Inundation maps are required for all high and significant hazard developments unless an unusual condition exists. The requirements for an inundation map are as follows:

1. Identify the antecedent flow conditions on which the maps are based.

2. Describe how the inundation boundaries were plotted. As a minimum, show on the map and/or in a table the maximum inundation elevation and the travel time of the front of the dam break flood wave to critical locations.

3. The map should be developed at a scale sufficient to be used for identifying downstream inhabited areas within the area subject to possible danger. Inundated areas should be clearly identified. It may be appropriate to supplement the inundation maps with water surface profiles at critical areas showing the water surface elevation prior to failure and the peak water surface elevation after failure.

4. The best available topographic map should be used. The expected inundation following the assumed failure should be delineated on the map. The lines delineating the inundated area should be drawn in such thickness or form (solid line, dashed line, dotted line) as to identify the inundation limits as the main features of the map but not bold enough to obliterate houses or other features which are to be shown as being inundated by the flood waters. Clarity is important. When plotting inundation limits between cross sections used for the analysis, the lines should reasonably reflect the change in water levels with consideration given to topographic patterns and both natural and manmade features. When inundation lines enter the area of an existing lake or reservoir, they should be so drawn as to represent an increase in the water level of such lake or reservoir. Should this increased water level overtop the dam, the appropriate inundation lines should be drawn downstream of such dam to represent expected inundation in the downstream channel up to a point where an increase in water level will no longer represent danger to life, health or property. The area between the inundation lines representing the water level may be shaded to distinguish the area of inundation. Care should be taken to select a shading which will not obliterate the background information shown on the map.

5. Describe the accuracy and limitation of the information supplied on the inundation maps and how best to use the maps. Since local officials are likely to use the maps for evacuation purposes, a note should be included on the map to advise that, because of the method, procedures, and assumptions used to develop the flooded areas, the limits of flooding shown and flood wave travel times are approximate and should be used only as a guideline for establishing evacuation zones. Actual areas inundated will depend on actual failure conditions and may differ from areas shown on the maps.
6. If inundation maps are to be shown on several pages, a map index should be included to orient the individual pages.

7. Include any other pertinent information.

D. Plans for training, testing and annual review

1. Annual training of project operators and other responsible personnel.
   * Describe plans to give detailed instructions to the operators, attendants and other responsible personnel on how to respond properly to a project emergency. Include plans for discussing procedures to be followed throughout the emergency and describe the chain of command during day/night or non-business hours.
   * Describe plans to hold refresher seminars with operators.

2. Annual review and test of the state of readiness:
   * Describe plans to make prompt updates of EAP from comprehensive review. Must annually supply FERC with statement that EAP has been tested, with inclusion of any needed revisions and updates or statement that no revisions and updates are needed. Provide all plan holders copies of all revisions. Mark pages "Revised M/D/YR/" and highlight revised material.
   * Describe the annual test. State who will be contacted, when and how. The annual test should involve a simulated drill for one of the emergency situations. Testing of remote sensing equipment at unmanned dams should be included. Coordination and consultation with local government, law enforcement officials and other organizations should be made in order to enhance the realism of the test.
   * Describe action to be taken.

- 19 -

* State who determines if the test was successful.

* State plans for submitting a critique of the test to Regional Engineer.

* Describe the checkpoints.

E. Documentation

1. Provide documentation of consultations with Federal, State and local agencies, including public safety and law enforcement bodies. Provide letters of acknowledgement from the contacted agencies.
   * Letters should indicate that each agency involved understands its responsibility for alerting and/or evacuating the public in those areas within its jurisdiction.

2. Provide letters or memoranda of contact
   * Coordination is essential to ensure that local officials responsible for warning and evacuation of the public comprehend and accept their individual and group responsibilities. Participation in the preparation of the plan will enhance their confidence in the plan and in the accuracy of its components. Coordination will provide opportunities for discussion and determination of the order in which public officials should be notified, backup personnel, alternate means of communication, and special procedures for periods of darkness, inclement weather, non-business hours, etc. Differences in procedures for notification for different emergency situations should be coordinated prior to finalizing the notification plan(s).
Advance preparations should include arrangements for such meeting(s) as are necessary with local and county governments, law enforcement officials, and other public officials who will be responsible for the warning and the evacuation of the occupants of the affected areas. The licensee should discuss the accuracy of the inundation maps or other means used to delineate the affected areas. Times available for response should also be discussed. Public officials to be notified and their priority of notification should be established. Special procedures should be developed for periods of darkness, inclement weather, and non-business hours.

All positions critical to the execution of the emergency action plan should be covered 24-hours a day, 7-days a week. Alternative or backup personnel should be identified for all public officials to be notified. Alternative means of communication should be identified.

Describe the coordination efforts. Include all letters directed by you to agencies or others and memoranda of meetings or conferences held.

---

**TABLE 1**

**SUGGESTED BREACH PARAMETERS**

(Definition Sketch Shown in Figure 1)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Type of Dam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average width of Breach</td>
<td>BR/2 ≤ BR ≤ 3H</td>
<td>Earthen, Rockfill</td>
</tr>
<tr>
<td>Note: BR = average width, not necessarily bottom width</td>
<td>BR ≥ 0.8 x Crest Length</td>
<td>Slag, Refuse</td>
</tr>
<tr>
<td>BR = bottom width for rectangle</td>
<td>BR = Crest Length</td>
<td>Concrete, Arch</td>
</tr>
<tr>
<td>BR ≠ bottom width for trapezoid</td>
<td>BR = Width of 1 or More Monoliths, usually BR ≤ 0.5 W</td>
<td>Masonry, Gravity</td>
</tr>
</tbody>
</table>

| Horizontal Component of Side Slope of Breach (Z) | 0 ≤ Z ≤ 2 | All |
| Note: Whether the shape is rectangular, trapezoidal, or triangular is not generally critical if the average width (BR) for each shape is the same. | 2 = 0 | Masonry, Gravity |
| What is critical is the assumed average width (BR) of the breach | 1 ≤ Z ≤ 2 | Slag, Refuse (Non-Engineered) |
| 0 ≤ Z slope of valley walls | Arch |

| Time to Failure (TPH) (in hours) | 0.3 ≤ TPH ≤ 3 | All |
| Note: TPH is a function of height of dam and location of breach | TPH ≤ 0.1 | Arch |
| By logic: | 0.1 ≤ TPH ≤ 0.2 | Masonry, Concrete |
| a) Larger the time to failure, the wider the breach should be | 0.3 ≤ TPH ≤ 3.0 | Earthen (Engineered, Compacted) |
| b) The greater HD & storage volume is, the greater TPH and BR will probably be | 0.1 ≤ TPH ≤ 0.5 | Earthen (Non Engineered, Poor Construction) |
| 0.1 ≤ TPH ≤ 0.3 | Slag, Refuse |

**Definition:**

- **HR - Height of Dam**
- **Z - Horizontal Component of Side Slope of Breach**
- **BR - Average Width of Breach**
- **TPH - Time to Fully Form the Breach**
- **H - Crest Length**

**Comments:**

1. The bottom of the breach should at least be at tailwater elevation or, if no tailwater, at the toe of the dam.
2. For a worst-case scenario, BR should be in the upper portion of recommended range and TPH should be in lower portion of recommended range.
5. Radiological Emergency Response Plan. Each owner of a hydroelectric project under jurisdiction of the Federal Energy Regulatory Commission located within a 10-mile radius of a nuclear plant licensed to operate shall prepare a radiological emergency response plan to be implemented in the event of a severe accident or incident resulting in the release of radioactive materials. A plan is required if the 10-mile radius includes any project structures such as the dam or powerhouse that are used in changing water flows, or project facilities that would be affected by radioactive materials in such a manner that would interfere with project operations. The plan will be a supplement to the Emergency Action Plan and made a part thereof. It should contain, but not necessarily be limited to:

A. Detailed procedures for: The evacuation of power plant personnel when advised or directed to do so by the appropriate State or local government official; setting of gate openings; continuation, curtailment, or cessation of generation; coordination with, and notification of, customers, power pools, and other interconnected power suppliers; advance coordination with operators of upstream and downstream reservoirs; and/or other actions as considered appropriate.

B. A list of State and/or local government officials who are responsible for notification of hydroelectric project personnel that a nuclear accident or incident is developing (or has occurred). This part of the plan should specifically identify the State or local government officials responsible for notifying individuals in the hydroelectric power plant owner's organization. It should also include provisions for keeping the owner's key personnel currently informed on the developing situation to allow timely action or response at the affected hydroelectric project. This portion of the plan should identify, if other than the officials noted above, the State or local government agency representatives authorized to direct or advise implementation of action, such as evacuation of the area, or other appropriate action.
C. Notification plans should be developed for alerting the following concerned individuals of proposed plan implementation. Reference can be made to the notification procedures contained in the main body of the emergency action plan if appropriate.

1. Local, State, and Federal government officials, including the FERC Regional Engineer or alternate.

2. Operators of water-related facilities

3. Residents and owners of properties that could be endangered by the change in project operation.

4. Supervisors and other company officials.

The radiological emergency response supplement to the emergency action plan shall be posted with the main body of the emergency action plan in a prominent location accessible to operating and supervisory personnel. Such personnel shall be familiar with their responsibilities under the plan. Training of these personnel shall be conducted to assure adequate and timely performance of their duties in the event of an emergency.

As with the other parts of the emergency action plan, all aspects of the plan are subject to continuous review and updating. At least once a year, a comprehensive review shall be made of the plan. Any revisions shall be made after consultation with Federal, State, and local agencies, and electric power producers and users, as appropriate. The need for an update shall be reported to the Regional Engineer no later than December 31 of each year.

The affected owner will be requested to file a plan no later than 3 months after the date of issuance of a license to operate a nuclear plant.

If the Regional Engineer determines that an emergency action plan is not required for the hydroelectric project, the radiological supplement shall, nevertheless, be filed. Evidence of coordination with the State or local director of civil defense, or the appropriate official responsible for emergency preparedness, should be obtained and forwarded with the plan. Three copies should be submitted to the Regional Office.
UNITED STATES OF AMERICA
FEDERAL ENERGY REGULATORY COMMISSION

NOTICE OF REVISION No. 1 TO TABLE 1
(Suggested Breach Parameters - page 21)
of the REVISED
EMERGENCY ACTION PLAN GUIDELINES
Issued April 5, 1985
(Issued March 18, 1987)

Pursuant to the authority in Section 12.22(a)(1) of the Commission's regulations, the Director, Office of Hydropower Licensing, has revised Table 1 (Suggested Breach Parameters - page 21) of the Revised Emergency Action Plan Guidelines issued April 5, 1985.

Copies of the revised Table 1 are available from the Commission's Division of Public Information, Director, Division of Inspections or the Regional Director (Atlanta, New York, Chicago, Portland, and San Francisco). The original table of suggested breach parameters was based on previous recommendations by Dr. Danny L. Fread of the National Weather Service. Table 1 has been revised to conform with current breach parameters recommended by Dr. Fread. Due to this revision, Table 1 has been expanded from one page to three pages (i.e. pages 21, 21A, and 21B).

Kenneth F. Plumb
Secretary

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### TABLE 1
SUDDIETE BREACH PARAMETERS
(Definition Sketch Shown in Figure 1)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Type of Dam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average width of Breach (BR) (See Comment No. 1)</td>
<td>HD &lt; BR &lt; 5HD (usually between 2HD &amp; 4HD)</td>
<td>Earthen, Rockfill</td>
</tr>
<tr>
<td></td>
<td>BR &gt; 0.8 x Crest Length</td>
<td>Slag, Refuse</td>
</tr>
<tr>
<td></td>
<td>BR = Crest Length</td>
<td>Concrete, Arch, Timber Crib</td>
</tr>
<tr>
<td></td>
<td>BR = Width of 1 or More Monoliths, usually BR &lt; 0.5 W</td>
<td>Masonry, Gravity</td>
</tr>
<tr>
<td>Horizontal Component of Side Slope of Breach (Z) (See Comment No. 2)</td>
<td>0 ≤ Z ≤ 2</td>
<td>All</td>
</tr>
<tr>
<td></td>
<td>Z = 0</td>
<td>Masonry, Concrete Timber Crib</td>
</tr>
<tr>
<td></td>
<td>1/4 ≤ Z ≤ 1</td>
<td>Earthen (Engineered, Compacted)</td>
</tr>
<tr>
<td></td>
<td>1 ≤ Z ≤ 2</td>
<td>Slag, Refuse (Non-Engineered)</td>
</tr>
<tr>
<td></td>
<td>0 ≤ Z ≤ slope of valley walls</td>
<td>Arch</td>
</tr>
<tr>
<td>Time to Failure (TPH) (in hours) (See Comment No. 3)</td>
<td>0.1 ≤ TPH ≤ 1</td>
<td>All</td>
</tr>
<tr>
<td></td>
<td>TPH ≤ 0.1</td>
<td>Arch, Timber Crib</td>
</tr>
<tr>
<td></td>
<td>0.1 ≤ TPH ≤ 0.3</td>
<td>Masonry, Concrete</td>
</tr>
<tr>
<td></td>
<td>0.1 ≤ TPH ≤ 1.0</td>
<td>Earthen (Engineered, Compacted)</td>
</tr>
<tr>
<td></td>
<td>0.1 ≤ TPH ≤ 0.5</td>
<td>Earthen (Non Engineered, Poor Construction)</td>
</tr>
<tr>
<td></td>
<td>0.1 ≤ TPH ≤ 0.3</td>
<td>Slag, Refuse</td>
</tr>
</tbody>
</table>

Definition: HD - Height of Dam
Z - Horizontal Component of Side Slope of Breach
BR - Average Width of Breach
TH - Time to Fully Form the Breach
W - Crest Length

Comments: See page 21A-21B
1. BR is the average breach width, which is not necessarily the bottom width. BR is the bottom width for a rectangle, but BR is not the bottom width for a trapezoid.

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

3. Time to failure is a function of height of dam and location of breach. Therefore, the longer the time to failure, the wider the breach should be. Also, the greater the height of the dam and the storage volume, the greater the time to failure and average breach width will probably be.

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

5. Breach width assumptions should be based on the height of the dam, the volume of the reservoir, and the type of failure (e.g., piping, sustained overtopping, etc.).

6. For a worst-case scenario, the average breach width should be in the upper portion of the recommended range, the time to failure should be in the lower portion of the recommended range, and the Manning's value should be in the upper portion of the recommended range. In order to fully investigate the effects of the impacts of a failure on downstream areas, a sensitivity analysis is required to estimate the confidence limits and relative differences resulting from varying failure assumptions:
   a. To compare relative differences in peak elevation based on variations in breach widths, the sensitivity analysis should be based on the following assumptions:
      1. Assume a probable (reasonable) maximum breach width, a probable minimum time to failure, and a probable maximum Manning's "n" value. Manning's "n" values in the vicinity of the dam (up to several thousand feet or more downstream) should be assumed to be larger than the maximum value suggested by field investigations in order to account for uncertainties of high energy losses, velocities, turbulence, etc., resulting from the initial failure.

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

HYDROMETEOROLOGICAL REPORT (HMR)

Nos. 51 and 52 vs HMR No. 33
APPENDIX IIC

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

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

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

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

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

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

   (a) HMR Nos. 51 and 52, or

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

3-0 Contents

<table>
<thead>
<tr>
<th>Title</th>
<th>Page</th>
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Appendix IIIA Seismic Zone Maps

Appendix IIIB Cracked Base Analysis
3-1 Purpose and Scope

3-1.1 General

The objective of this section of the Guidelines is to provide Staff engineers with recommended procedures and stability criteria for use in the analysis of unconstructed and existing concrete gravity structures and in the review of analyses conducted by licensees, or their consultants, in prelicensing applications, supporting design reports or post-licensing Part-12 consultants reports. It is not intended to generate any new philosophy or theories on the methods used to analyze these structures. Existing Bureau of Reclamation and Corps of Engineers literature as well as other readily available references have been used and are listed in section 3-8.

As mentioned in the preface to these guidelines, considerable engineering judgement must be exercised by staff when evaluating procedures or situations not specifically covered herein. Unique problems or unusual solutions may require deviations from the criteria and or procedures outlined in this chapter. In these cases, such deviations must be evaluated on an individual basis in accordance with Chapter 1, paragraph 1-4.

3-1.2 Review Procedures

Review by the staff of analyses performed by licensees, or their consultants, should concentrate on the assumptions used in the analysis. The basis for critical assumptions such as allowable stresses, drain effectiveness, and loading conditions should be carefully examined. The consultant's reports, exhibits, and supplemental information must provide justification for these assumptions by way of testing information or through records maintained during the actual construction of the project. Methods of analysis should conform to the conventional procedures used in the engineering profession.

1/ Reference: 18 CFR Part 4, Subpart E, §4.41 (g), and 18 CFR, Part 12, Subpart D.
3-2 Forces

3-2.1 General

Many of the forces which must be considered in the design of the gravity dam structure are of such a nature that an exact determination cannot be made. The intensity, direction and location of these forces must be assumed by the designer after consideration of all available facts and, to a certain extent, must be based on judgment and experience. 2/

3-2.2 Dead Loads

It is usually necessary to proceed with designs, at least in a preliminary manner, before a complete concrete analysis including data on weights, is available. In such cases, the unit weight of concrete for preliminary studies should be assumed to be 150 pounds per cubic foot, and should be determined by laboratory tests for final designs. In the determination of the dead load, relatively small voids, such as galleries, normally are not deducted except in low dams, where such voids create an appreciable effect upon the stability of the structure. The dead loads considered should be weights of concrete, superimposed backfill and appurtenances such as gates and bridges. 3/

3-2.3 External Hydrostatic Loads

3-2.3.1 Static Loads

Although the weight of water varies slightly with temperature, the weight of fresh water should be taken at 62.4 pounds per cubic foot. Triangular distribution of the static water pressure acting normal to the face of the dam should suffice for the design of most dams. In special cases the dynamic effects of the velocity of approach to an overflow spillway become significant and should be considered. The dynamic effects of

2/ Reference 3, page 3.
3/ Ibid.
headwater and tailwater are discussed in paragraph 3-2.3.2. Tailwater pressure, adjusted for retrogression, should be taken at full value for nonoverflow sections and at about 60 percent of full value for deep flow over the spillway, except that full value will be used in all cases for uplift assumptions. 4/

3-2.3.2 Dynamic and Subatmospheric Loads

The forces acting on an overflow dam are complicated by the jet of water. Normally such forces may be neglected in the stability analysis; however, conditions can exist which cause overturning forces to be exerted upon the dam that exceed those calculated using static methods. The primary areas of concern are the crest pressures on high overflow crests and bucket pressures in both high and low overflow spillways.

3-2.3.3 Crest Pressures

When the depth of water flowing over the crest of the overflow portion of a dam is greater than the head for which it was designed, the nappe will tend to separate from the face of the dam. This creates an area of subatmospheric pressure along the dam face which adds to the overturning forces on the dam. If the crest is poorly designed, there may be local areas where the intensity of the subatmospheric pressure approaches full vacuum, then a break occurs allowing aeration, and a sudden return to normal pressure. This is repeated periodically for very short periods of time and can cause strong vibrations to occur. If, however, the crest is properly designed, nappe separation will be minimized. Plate 4, reference 18 should be used to estimate the magnitude of these pressures.

4/ Ibid.
3-2.3.4 Bucket Pressures

The flow of high velocity water through a spillway bucket may result in high pressures on the bucket surface. Model and prototype results indicate that these pressures change continuously throughout the bucket and are a function of the entering velocity and depth of flow, radius of curvature of the bucket, and the total angle of deflection of the flow. Further discussion concerning this phenomenon can be found in reference 18. Plate 21 of this reference presents procedures for calculating bucket pressures in higher overflow structures, and Plate 36 can be used to determine toe curve pressures in low ogee structures.

3-2.4 Internal Hydrostatic Loads (uplift)

3-2.4.1 General

Uplift exists within the body of a dam, at the contact plane between the dam and its foundation, and within the foundation below the contact plane. It is an active force which must be included in the design of dams to insure structural adequacy. 5/ Uplift shall be assumed to act over 100 percent of the area of any horizontal plane cut through the dam, at the plane of contact with the foundation and at any plane within the foundation.

3-2.4.2 Horizontal Planes within the Dam

For normal loading conditions, uplift along horizontal planes above the foundation and within the body of the dam shall be assumed to vary from 100% of normal headwater at the upstream face to 100% of tailwater or zero, as the case may be, at the downstream face. For flood conditions, the uplift shall be assumed to vary from 100% of normal headwater plus 50% of the difference between the maximum water surface and normal headwater, at the upstream face, to 100% of normal tailwater plus 50% of the difference between the

maximum tailwater elevation and normal
tailwater or zero, as the case may
be, at the downstream face. When a
vertical drainage system has been
provided within the dam, the drain
effectiveness and uplift assumptions
should follow the guidance provided in
paragraph 3-2.4.3 below and should be
verified by instrumentation.

3-2.4.3 Rock Foundations

Uplift distribution at the plane of
contact between the dam and its foundation,
and within the foundation depends on
depth and spacing of drains, grout
curtain, rock permeability, jointing,
faulting, and any other geologic features
which may modify the seepage or flow
of water. Effective downstream drainage,
whether natural or artificial will usually
limit the uplift pressure at the toe of the
dam to tailwater pressure.

The magnitude of uplift reduction due to
seepage control measures such as drainage
and/or grouting is a subject which
evokes considerable discussion and
differences of opinion among engineers.
However, one point upon which most
engineers will agree is that an uplift
reduction should not be assumed unless
the geologic characteristics of the foundation
have been thoroughly investigated, and the
design of the seepage control measures
has been tailored to correct the specific
deficiencies of the site. To complicate
matters is the fact that the design of a
drainage or grouting system, once the problems
have been identified, is at best empirical
and usually subjective. In addition, the
design and installation of seepage
control measures does not complete the
solution of the site problems, nor does
it insure the proper operation of the measure
throughout the life of the project. These
can only be assured by implementation of
a comprehensive monitoring and maintenance
program. For this reason structures with
closed drainage systems, which do not
allow inspection or maintenance, are
considered for analysis purposes to be subject
to full uplift loading unless a monitoring
system is installed to verify uplift pressures on a periodic basis. Of course, future increasing uplift pressures that cannot be corrected because of the closed system may dictate structural modifications.

During design, the engineer must coordinate closely with the engineering geologists in order to evaluate the character of the foundation and the effectiveness of the proposed drainage system. Any drain effectiveness assumptions made should be coupled with a post-construction testing and monitoring program aimed at verifying the assumptions. The system should include instrumentation to verify continued operation of the drains and to determine the effects of corrosion or clogging upon the original effectiveness assumption. A maintenance program for the system should be developed and implemented that is consistent with the nature of the system. In general, maintenance should include, but not be limited to: periodic testing to locate clogged and inoperative drains; redrilling or cleaning of drains which have become clogged; installation of additional drains to achieve design concept; and periodic monitoring and calibration of pressure gages.

For existing dams, foundation information may be limited, and the extensive testing and exploration required to evaluate the strength and permeability of the foundation may limit the amount of field information available to staff engineers prior to the analysis. It is, therefore, necessary to be conservative in the selection of uplift parameters regardless of past performance. The fact that dams designed on the assumptions of uplift in use at the time the dam was built have stood successfully is not positive proof that such assumptions are correct. There is present in some rock foundations, even in poorly constructed joints in the dam, a certain amount of shear and tensile strength that is frequently neglected in the overturning analysis. This neglected strength adds to the stability of the dam and may be sufficient to counter balance deficient uplift assumptions. 6/

---

Staff review of assumptions concerning uplift reduction should consider the above discussion. Applicants should be required to submit supplemental information in support of any uplift reduction assumption. The following guidance shall be applied to staff review of the design assumptions. The uplift criteria cited herein may be relaxed only when sufficient field measurements of actual uplift pressures justify any proposed deviations.

3-2.4.3.1 Uplift Assumptions

Uplift at the foundation-concrete interface for structures having no foundation drains or a closed drainage system should be assumed to vary as a straight line from 100% of the headwater pressure at the upstream face (heel) to 100% of the tailwater pressure at the downstream face (toe) applied over 100% of the base area (See Figure 1).

Uplift at foundation concrete-rock interface for structures having an open drainage system (drain holes drilled into rock with outlets accessible for inspection and maintenance) should be assumed to vary as a straight line from full headwater pressure at the heel to reduced uplift at the drain and thence to full tailwater pressure at the toe (See Figure 2). The drain effectiveness (E) may be determined by instrumentation, but an effective maintenance plan as outlined in paragraph 3-2.4.3 must be implemented. If the drains are spaced within a distance from the upstream face, that is equal to 5% of the reservoir depth at the face of the dam, uplift may be assumed to vary linearly to tailwater from the value determined as through the drains were exactly at the heel (See Figure 3).

In all cases, uplift on any portion of the base or section not in compression should be assumed to be 100% of the assumed upstream head except when
Headwater

Tailwater

Uplift Distribution-Without Foundation Drainage

FIGURE 1
Uplift Distribution - With Foundation Drainage

FIGURE 2

Where:

\[ E = \text{Drain effectiveness expressed as a decimal.} \]

\[ H_3 = K(H_1 - H_2) \frac{L - x}{L} + H_2 \]

\[ K = 1 - E \]
When $x \leq 5\%$ of Reservoir Depth

If $H_4 > H_2$

$H_3 = K(H_1 - H_4) + H_4$

If $H_4 < H_2$

$H_3 = K(H_1 - H_2) + H_2$

**UPLIFT DISTRIBUTION**

**UNCRACKED BASE WITH DRAINAGE**

**DRAINS CLOSE TO UPSTREAM FACE**

Figure 3
the non-compressive foundation
pressure is the result of earthquake
forces. If, however, instrumentation
can verify use of less than 100%, then
uplift pressure may be reduced accordingly.
Extrapolations of uplift pressure to
headwater and tailwater conditions that
exceed the conditions monitored is not
appropriate without supporting data.

Wherever possible, drainage efficiencies
for unconstructed projects should be
verified through field investigations
involving post-construction measurement
of actual uplift pressures. Any
measured drain efficiency must be
considered valid only for the reservoir
loading at which the measurement was
taken. Extrapolation to higher
reservoir levels in the absence of
supporting field data may not be valid
in all cases where the net loading from
the unusual loading condition is
significantly greater than the usual
loading condition.

3-2.4.3.2 Drainage Galleries

If a drainage gallery is provided, it
should be at an elevation at or
below tailwater level. If the
gallery is above tailwater elevation,
the pressure at the line of drains
should be determined as though the
tailwater level is equal to the gallery
elevation (See Figure 4). However, with
instrumentation, actual data would govern.

3-2.4.3.3 Grouting

Grouting alone should not be considered
sufficient justification to assume an
uplift reduction. A grout curtain may
retard flow to the drains from the up­
stream head, but the degree of
uplift relief may be lessened as the
age of the dam increases, due to
deterioration of the curtain. There­
fore, unless drains are provided to
relieve uplift pressures which would
build up over a period of time, full
uplift pressure should be assumed
downstream of the grout curtain.
where:

\[ E = \text{Drain effectiveness expressed as a decimal} \]

\[ K = 1 - E \]

When \( H_4 > H_2 \):

\[ H_3 = K(H_1 - H_4) \left( \frac{L - X}{L} \right) + H_4 \]

When \( H_4 < H_2 \):

\[ H_3 = K(H_1 - H_2) \left( \frac{L - X}{L} \right) + H_2 \]

**UPLIFT DISTRIBUTION**

**WITH DRAINAGE GALLERY**

*Figure 4*
3-2.4.3.4 Upstream Apron

A horizontal or vertical upstream apron may serve as a cut-off where the foundation, though basically impervious and not susceptible to grouting, is so jointed that considerable flow is expected. It may also serve to reduce the hydrostatic pressure which would exist at the heel from an open crack if the joint between the apron and structure is sealed. Any assumed reduction in hydrostatic pressure must be verified by instrumentation data (See Figure 5).

3-2.4.3.5 Earthquake

Uplift pressures should be assumed to be unaffected by earthquake loading, i.e., at the pre-earthquake intensity during and immediately after the earthquake.

3-2.4.3.6 PMF

Uplift reductions should not be based on the judgement that the PMF flood event is of such short duration and the permeability of the foundation so low that the elevated headwater and tailwater pressures are not transmitted under the base of the dam. This less than conservative assumption is invalid because extreme design floods and the resulting elevated water levels often last many hours, if not days, and because in a saturated rigid system, such as a rock foundation with joints, extremely small volume changes are necessary to transmit large pressure changes. In the absence of corroborative evidence (e.g., measurements of piezometer levels during prior floods) the uplift should be assumed to vary directly with changes in headwater and tailwater levels.

3-2.4.3.7 Cracked Base Analysis

If the cracked base analysis is required and the crack does not extend beyond the location of the drains, the uplift will be as shown in Figure 6.
Headwater

Apron slab anchored to rock.

Waterstops in monolith joints, spliced with apron slab waterstops.

Expansion joint and premoulded filler.

Where:

\[ H_4 = (H_1 - H_2) \frac{L}{L_1} + H_2 \]

UPLIFT DISTRIBUTION—WITH UPSTREAM CUTOFF

FIGURE 5
When \( H_4 < H_2 \),
\[
H_3 = K(H_1 - H_2) + \frac{T}{x}(1-K)(H_1 - H_2) + H_2
\]

When \( H_4 > H_2 \),
\[
H_3 = K(H_1 - H_2) + \frac{T}{x}(1-K)(H_1 - H_2) + H_4
\]

where:
- \( E \) = Drain effectiveness expressed as a decimal
- \( K = 1 - E \)
- \( T \) = Length of crack
- \( T < x \)

**Figure 6**

**UPLIFT DISTRIBUTION**

**CRACKED BASE WITH DRAINAGE**
If the crack extends beyond the location of the drains, the performance of the drainage system cannot be accurately predicted; therefore, no allowance for drain effectiveness should be allowed unless verification in accordance with paragraph 3-2.4.3.1 can be provided. The uplift diagram will then be as shown in Figure 7.

3-2.4.3.8 Foundation Reaction

The assumed unit uplift pressure should be added to the computed foundation reaction to determine the maximum possible unit foundation pressure at any point. 7/

3-2.4.4 Soil Foundations

Uplift pressures acting upon the base of a gravity structure constructed on a pervious soil foundation are related to seepage through permeable materials. Water percolating through pore spaces in the materials is retarded by frictional resistance, somewhat the same as water flowing through a pipe. The intensity of the uplift can be controlled by construction of properly placed aprons, cutoffs and other devices. 8/ One of the following methods should be used to determine the magnitude of the uplift pressure:

3-2.4.4.1 Creep Theory

Under creep theory, the uplift pressure is assumed to be the sum of two components the seepage potential and the position potential.

The seepage potential is calculated by first determining the creep distance, which a molecule of water would follow as it flows beneath the structure. The creep distance starts at a point on the ground line directly over the heel, and

7/ Reference 3, Page 6
8/ Refernce 9, Page 333.
UPLIFT DISTRIBUTION
CRACKED BASE WITH DRAINAGE

Figure 7
ends at another point on the ground line directly above the toe, following the boundary of the sides and bottom of buried concrete. The seepage potential is then calculated by dividing the distance remaining on the creep distance, from the point under consideration to the point of zero potential (usually tailwater) by the total creep distance, and multiplying the resulting ratio by the head differential between headwater and tailwater. In some cases, the groundline elevation at the toe may be used to determine the head differential in lieu of the tailwater elevation if it results in a greater head difference.

The position potential at any point on the path is the difference in elevation between the point under consideration and ground line, or tailwater, whichever is greater.

The effective uplift pressure at a point is then calculated by multiplying the sum of the seepage and position potentials of the points by the unit weight of water.

There are certain limitations on the use of the creep method. A "weighted creep ratio", defined as the creep distance divided by the head differential, should always be calculated. It is necessary to limit the creep ratio as shown in Table 1 in order to avoid seepage potential conditions which could lead to boils and/or blowouts.

This weighted creep ratio recognizes the differences in vertical and horizontal permeability of most soil foundations. Generally, the vertical lines of creep are more efficient at reducing head than horizontal lines because vertical permeability is usually less than horizontal permeability.
The creep distance used in the weighted creep ratio (Cw) should be as shown below.

\[ L_w = \left( \frac{K_v}{K_h} \right) L_h + L_v \]

Where:  
- \( K_h \) = horizontal permeability coefficient  
- \( K_v \) = vertical permeability coefficient  
- \( L_h \) = total horizontal length of creep path.  
- \( L_v \) = total vertical length of creep path.  
- \( L_c \) = effective creep distance

<table>
<thead>
<tr>
<th>Material</th>
<th>Cw</th>
</tr>
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<tbody>
<tr>
<td>Very fine sand or silt</td>
<td>8.5</td>
</tr>
<tr>
<td>Fine sand</td>
<td>7.0</td>
</tr>
<tr>
<td>Medium sand</td>
<td>6.0</td>
</tr>
<tr>
<td>Coarse sand</td>
<td>5.0</td>
</tr>
<tr>
<td>Fine gravel</td>
<td>4.0</td>
</tr>
<tr>
<td>Medium gravel</td>
<td>3.5</td>
</tr>
<tr>
<td>Coarse gravel including cobbles</td>
<td>3.0</td>
</tr>
<tr>
<td>Boulders with some cobbles and gravel</td>
<td>2.5</td>
</tr>
</tbody>
</table>

* Reference 35 page 617.  
** Foundation type should be determined upon consultation with Staff soil Engineers.

3-2.4.4.2 Flow Net Method

This method is a graphical procedure which involves the construction of paths of percolation (flow lines) and lines of equal potential (lines drawn through points of equal total head) in subsurface flow. Flow lines and equipotential lines are superimposed upon a cross section of the soil through which the flow is taking place. Reference for the procedure is made to any standard text book on soil mechanics. 9/

9/ See references 30, 31 and 32 for detailed guidance concerning flow nets.
The flow net method should be used whenever the weighted creep ratio for the structure under consideration is less than that shown in Table 1. As with the weighted creep method, the position potential should be added to the seepage potential (determined from the flow net), and the result multiplied by the unit weight of water to calculate the uplift pressure.

3-2.5 Earth and Silt Pressures

3-2.5.1 Earth Pressures

Earth pressures exerted on dams or other gravity structures by soil backfills should be calculated as outlined in reference 19. 10/ Gravity dams should be designed for active earth pressures if built on soil foundations, and for at-rest earth pressures if a rock foundation is anticipated. The unit weight of the soil to be used in the analysis should be supported by site investigations. If the backfill is submerged, the unit weight of the soil (bouyant weight) should be reduced by the unit weight of water.

3-2.5.2 Silt Pressures

The probable depth of silt deposits within the reservoir during the expected useful life of the dam should be given careful study. Not all dams will be subjected to silt pressure, and the designer should consider all available hydrologic data before deciding whether an allowance for silt pressure is necessary. For existing structures the silt load should be determined by hydrographic surveys. Vertical pressure exerted by saturated silt is determined as if silt were a saturated soil, the magnitude of pressure varying directly with depth.

Horizontal pressure exerted by the silt load is calculated in the same manner as submerged earth backfill.

10/ Reference 19, Pages 1 thru 5.
3-2.6 Earthquake Forces

3-2.6.1 General

Earthquake loadings should be selected after consideration of the accelerations which may be expected at each project site as determined by the geology of the site, proximity to major faults, and earthquake history of the region as indicated by available seismic records. Seismic risk maps can be used to establish the probability zone for projects which do not have detailed seismicity studies. An earthquake analysis is not required for structures in zones 0 and 1 unless site studies indicate the presence of capable faults, or recent earthquake epicenters are discovered near enough to the dam to cause structural damage in the event of an earthquake. The following methods can be used to calculate the magnitude of the earthquake forces.

3-2.6.2 Seismic Coefficient Method

This is a static procedure which utilizes ratios of assumed accelerations of the structure to the acceleration of gravity to define the horizontal inertial force acting on the dam. The hydrodynamic forces (i.e., the reaction of the reservoir water on the surface of the dam) are determined using the Westergaard formula. This method can be used to screen zone 2 projects to determine if a more rigorous analysis is required. For zone 2 projects which are found to be stable using the seismic coefficient method, no further analysis is required. However, if a zone 2 project is found to be unstable using the method, a pseudo dynamic analysis should be performed. References 3 and 4 should be used to calculate the magnitude of the earthquake force using seismic coefficient methods. Seismic zone maps are provided in Appendix IIIA.

3-2.6.3 Pseudo Dynamic Method

This procedure was developed as a hand calculated alternative to the more general analytical procedures which require computer programs. It is a simplified response spectrum analysis which determines the structural response, in the fundamental mode of vibration, to only the horizontal component of ground motion. Reference 20 should be used to calculate inertial and hydrodynamic earthquake forces for the
pseudo dynamic method. This method can be used to evaluate the compressive and tensile stresses only at locations above the base of the dam. Because of the oscillatory nature of earthquakes, and the subsequent structural responses, conventional overturning and sliding stability criteria are not valid when dynamic and pseudo dynamic methods are used.

3-2.6.4 Dynamic Methods

Dynamic response analyses are based upon the structure's dynamic characteristics and the expected ground motions for the particular location. Ground motions, in the form of time histories from a design earthquake, can be input to a finite element model of the structure and foundation to determine the dynamic stress distributions as a function of time. This procedure is generally referred to as the Time History Method. Another approach, called the Response Spectrum Method, is to calculate the approximate maximum dynamic deformation, due to the design response spectrum, of an equivalent system. Dynamic stresses can then be computed by a static stress analysis.

3-2.6.5 Use of Dynamic Analysis

A pseudo dynamic analysis is recommended for all dams in zones 3 and 4, and for all zone 2 dams for which the screening process of paragraph 3-2.6.2 above indicates possible problems. Structures which exhibit potential overstressing when subjected to pseudo dynamic loading, should be evaluated using dynamic methods. This would also involve a geological and seismological evaluation to establish the design earthquakes or response spectra. A geological and seismological review of all high hazard dams in seismic zones 2 through 4 should be conducted to locate faults and ascertain the seismic history of the region around the dam. Where capable faults or recent earthquake epicenters are discovered near enough to the dam to cause structural damage (in the event of an earthquake), a dynamic analysis should be performed. These studies and analyses should be conducted by the Licensee, or Applicant, as required by license articles on specific requests by Staff and then reviewed by Staff. Guidance concerning the review and analysis of seismic studies is provided in 3-4.5 of this Chapter.
3-2.7 Ice Pressure

Ice pressure is normally not of great importance in the analysis of gravity dams. Instances of ice damage to the gates are quite common while no instance is known of any serious ice damage occurring to the dam. Ice pressure is created by thermal expansion of the ice and by wind drag. Pressures caused by thermal expansion are dependent on the temperature rise of the ice, the thickness of the ice sheet, the coefficient of expansion, the elastic modulus and the strength of the ice. Wind drag is dependent on the size and shape of the exposed area, the roughness of the surface, and the direction and velocity of the wind. Ice loads are usually transitory. Not all dams will be subject to ice pressure, and the engineer should decide whether an ice load is appropriate after consideration of the above factors. An example of the conditions conducive to the development of potentially high ice pressure would be at a reservoir with hard rock reservoir walls which totally restrain the ice sheet. In addition, the site meteorological conditions would have to be such that an extremely rigid ice sheet develops. For the purpose of the design of structures for which an ice load is expected it is recommended that a pressure of 5000 pounds per square foot be applied to the contact surface of the structure, based upon the expected ice thickness. For dams in this country the ice thickness normally will not exceed 2 feet. The existence of a formal system for the prevention of ice formation, such as an air bubble system, may reduce or eliminate ice loadings. Information showing the design and maintenance of such a system must be provided in support of this assumption. Ice pressure should be applied at the normal pool elevation. Further information concerning ice loadings can be found in references 3 (Pages 325, 326, & 327), 14, 15, & 16.

3-2.8 Wind Pressures

Wind pressure of 30 pounds per square foot in any direction should be used in the stability investigation of dam superstructures carrying large crest gates.

11/ Reference 3, Page 5.
12/ Reference 1, Page 29.
Temperature

The effects of temperature change in gravity dams are not as important in the design as those in arch dams. However, during construction, the temperature change of the concrete in the dam should be controlled to avoid undesirable cracking. 14/

Natural shrinking, artificial cooling, and temperature changes all produce secondary stresses. These stresses in themselves will not cause failure in a dam but may modify the ability of the dam to resist applied loads. While these stresses may be kept under control by proper construction methods, they may require investigation. 15/

Volumetric increases caused by temperature rise will transfer load across transverse contraction joints if the joints are grouted. Horizontal thrusts which are caused by volumetric changes as temperature increases will result in a transfer of load across grouted contraction joints, increasing twist effects and the loading of the abutments. Ungrouted contraction joints are assumed to offer no restraint on volumetric increase caused by temperature rise and no associated transfer of load, providing the mean concrete temperatures remain below the closure temperature.

The effects of temperature change should be investigated if joints are to be grouted and when the operating temperatures are above the closure temperature if joints are to be ungrouted. 16/

References 17 and 21 provide detailed guidance concerning the control of temperatures in mass concrete during both design and construction.

14/ Reference 7, Page 20.
15/ Reference 3, Page 6.
16/ Reference 7, Page 23.
3-3 Loading Combinations

3-3.1 General

The following loading conditions and requirements are suitable in general for gravity dams of moderate height. Loads which are not indicated, such as wave action, or any unusual loadings should be considered where applicable. Power intake sections should be investigated with emergency bulkheads closed and all water passages empty.

3-3.2 Case I Usual Loading Combination - Normal Operating Condition

The reservoir elevation is at the normal power pool, as governed by the crest elevation of an overflow structure or the top of the closed spillway gates whichever is greater. Normal tailwater is used. Horizontal silt pressure should also be considered, if applicable.

3-3.3 Case II Unusual Loading Combination - Flood Discharge

The project inflow design flood up to and including the Probable Maximum Flood, if appropriate, that results in reservoir and tailwater elevations that exert the greatest head differential and uplift pressure upon the structure should be used. However, unusual conditions such as high tailwater should be examined on a case by case basis since it is possible that the worst case loading condition exists under other than extreme floods. For further discussion on this latter consideration, refer to paragraph 3-5.4

3-3.4 Case IIA Unusual Loading Combination - Ice

Case I loading plus ice pressure, if applicable. Generally, ice pressure will not be a factor in the stability analysis, but may affect the operation, or structural integrity of flashboards and spillway gates.

17/ Reference 3, Page 11.
3-3.5 Case III Extreme Loading Combination - Normal Operating with Earthquake

The same loading as in Loading Condition I is used except that the inertial force due to the earthquake acceleration of the dam, and the increased hydrostatic forces due to the reservoir reaction on the dam are added.

Three methods can be used to evaluate the earthquake forces; they are: use of static seismic coefficients; pseudo dynamic procedures; and dynamic analysis using either response spectra or time histories to define the earthquake ground motion. Recommended procedures for each of these methods are presented in paragraph 2.6.

3-3.6 Case IIIA Extreme Loading Combination - (Unconstructed-Dams Only) - Construction Condition with Earthquake

Construction of the dam completed, no water in reservoir, no tailwater but with earthquake forces as determined in Case III applied.

3-4 Methods of Analysis

3-4.1 General

Selection of the method of analysis should be governed by the design stage, the type and configuration of the structure being considered. The gravity method will generally be sufficient for the analysis of most structures, however, more sophisticated methods may be required for curved in plan structures, structures with unusual configurations or to conduct detailed stress analyses of final designs.

3-4.2 The Gravity Method

The gravity method of Stress and Stability Analysis is used a great deal for preliminary studies of gravity dams, depending on the phase of design and the information required. The gravity method is also used for final designs of straight gravity dams in which the transverse contraction joints are neither keyed nor grouted.
The gravity method provides an approximate means for determination of stresses in a cross section of a gravity dam. It is applicable to the general case of a gravity section with a vertical upstream face and with a constant downstream slope and to situations where there is a variable slope on either or both faces. Uplift pressures on a horizontal section are usually not included with the contact pressures in the computation of stresses, and are considered separately in the computation of stability factors. 18/

Use of the gravity method requires that several simplifying assumptions concerning the application of loads on the dam and the structural behavior of the dam be made. These assumptions are: The concrete in the dam is a homogeneous, isotropic and uniformly elastic material; all loads are transmitted to the foundation through cantilever action of the dam without support from adjacent monoliths; and normal stresses are distributed linearly on horizontal planes.

Examples of the use of the Gravity Method are presented in reference 1, pages 28 through 43.

When using the Gravity Method, stability factors for shear-friction and overturning, and foundation stresses are determined. The stability factor for sliding should be determined in accordance with reference 5. Uplift should be included in the shear stress calculation required for equations (1) and (2) of that reference. It is important to note that the factors of safety adopted for sliding in reference 5 are based upon the premise that extensive foundation exploration has been conducted using sophisticated, state-of-the-art techniques. Staff geologists should, therefore, be consulted concerning the adequacy of the foundation exploration and testing program prior to acceptance of any sliding analysis based on reference 5. In general, the factors of safety in Table 2 will govern.

18/ Reference 1, Page 37.
Stability against overturning is usually determined by calculation of the location of the resultant of all forces acting on the dam, and by determination of interface and foundation stresses. Foundation stresses should be determined without uplift included in order to check for the cracked base condition, as outlined in Section 3-4.6. Stability criteria and required factors of safety for overturning and sliding are presented in Section 3-5.0.

3-4.3 Trial-Load Methods

A gravity dam may be considered to be made up of a series of vertical cantilever elements from abutment to abutment. If the cross-canyon profile is narrow with steep sloping walls, each cantilever from the center of the dam toward the abutments will be shorter than the preceding one. Consequently, each cantilever will be deflected less by the waterload than the preceding one and more than the succeeding one. If the transverse contraction joints in the dam are keyed, regardless of whether grouted or ungrouted, the movements of each cantilever will be restrained by the adjacent ones. The longer cantilever will try to deflect the adjacent shorter cantilever forward and the shorter cantilever will tend to restrain the longer cantilevers. This interaction between adjacent cantilever elements causes torsional moments, or twists, which materially affect the manner in which the waterload is distributed between the cantilever elements in the dam. This changes the stress distribution from that of ordinary gravity analysis in which the effects of twist, as well as deformation of the foundation rock, are neglected. All straight gravity dams having keyed transverse contraction joints should, therefore, be treated as three-dimensional structures and designed on that basis.

If the canyon is wide and flat, the cantilevers in the central portion of the dam are of about the same length and the effects of twist are usually negligible. However, twist effects may be important in the abutment regions where the length of the cantilevers changes rapidly. This action tends to twist the cantilevers from their seats on the sloping canyon walls, thus tending to develop cracks in the dam in these regions.
A sharp break in the cross-canyon profile will result in an abrupt change in the length of cantilevers in that region. This has the effect of introducing an irregular wedge of rock in the dam and causes a very marked change in stresses and stability factors. Such conditions should be eliminated, if possible, by providing additional rock excavation so that a smooth profile is obtained. 19/

A detailed description of the theory and procedures used in Trial-Load methods is beyond the scope of these guidelines. Reference 1, pages 43 through 68 should be used as guidance for this method.

3-4.4 Finite Element Methods

3-4.4.1 General

Dams of moderate height may be analyzed using the Gravity and Trial-Load methods of analysis; however, the stress changes which occur near the base of a dam, due to foundation yielding, may be of importance in high dams. The Finite Element Method (FEM) permits the engineer to model closely the actual geometry of the structure and its interaction with the foundation. References 22 and 23 provide the theory and formulation of the method in detail. Because of the complexity of problems handled by FEM, computerized analysis is a must. Preview of FEM studies should concentrate upon the modeling assumptions made and the actual computer input to ensure that the computer model accurately predicts the structural behavior of the dam. In some cases, two dimensional (2-D) FEM may sufficiently model the behavior of the dam. However, three-dimensional (3-D) FEM should be used when the structure or loading is such that plane stress or strain conditions may not be assumed.

3-4.4.2 Two-Dimensional Finite Element Analysis

The two-dimensional finite element method is capable of analyzing the majority of problems associated with variations in the geometry of sections of the dam. Three-dimensional effects can be approximated

19/ Reference 1, Page 43.
by making a two-dimensional analysis in more than one plane. The two-dimensional finite element method is capable of solving for stresses economically even when great detail is necessary to attain sufficient accuracy. 20/

Two-dimensional finite element analysis is adaptable to gravity dam analysis when the assumption of planarity is used. The stress results for loading of typical transverse sections (perpendicular to the axis) are directly applicable. Sections including auxiliary works can be analyzed to determine their stress distribution.

The two-dimensional finite element analysis allows the foundation, with its possible wide variation in material properties, to be included with the dam in the analysis. Zones of tension cracks and weak seams of material can be included in the foundation. 21/

3-4.4.3 Three-Dimensional Finite Element Analysis

Three-dimensional FEM may be required when the geometry of the problem is such that the stability of the dam depends upon stress distribution parallel to its axis, as in a curved-in plan structure, or when the cross section of the dam or its loading is not uniform. Three-dimensional states of stress can be approximated, using 2-D FEM, by combining the results of 2-D FEM studies done on transverse and longitudinal sections of the structure. This method may be preferable due to its simplicity.

3-4.4.4 Uplift Loads for Finite Element Studies

Uplift pressures must be included as point loads acting at each nodal point on the dam-foundation interface. Pressures are usually calculated using conventional straight line distributions (as outlined in Section 3-2.4) or flow net methods, and then averaged over the distance between nodes.

20/ Reference 1, Page 72.
21/ Ibid, Page 74.
The load on each node is then determined by multiplying this average pressure by the distance between nodes. The resulting product is then applied to the node in an upward direction. If the program has the capability to use uniformly varying edge loads on the elements, then the uplift distribution can be input directly. Care must be taken to apply the same loads, in a downward direction, to the row of nodes directly below the interface in order to obtain accurate stresses in the foundation.

3-4.4.5 Analysis of Results of Finite Element Method Studies

It is recommended that an experienced staff structural engineer review any FEM study submitted in support of FERC's dam safety program. The validity of any computer analysis is determined by the input data and the program used. Additional factors combine to make this even more important in a FEM analysis. For instance, an FEM analysis is an elastic analysis as opposed to the rigid body analysis used in the gravity method, and the results can be extremely difficult to verify by hand calculations. Another factor is that subtle changes in material properties, boundary conditions, element type, and model geometry can have a significant effect upon the results. The engineer must be able to recognize the influence of these factors upon displacements and stresses in order to judge the reasonableness of the analysis and interpret the results.

Staff review of the results of FEM study should include an assessment of the validity of the stress distributions on any planes of importance, such as along the dam-foundation interface. For 2-D FEM studies, this can be done by plotting the stresses on the plane and determining the area under the curve. At the interface, the area should be equal to the dam weight minus any uplift, and should equal the dam weight at the row of nodes immediately below the interface. If these conditions are not satisfied, the computer input should be re-examined to determine if the computer input is accurately modeling the prototype.
Whenever possible, stress contour plots should be provided. Study of these plots can reveal much about the behavior of the structure that may not be readily apparent by examining tables of stresses and displacements. Zones of high tensile stress concentration at the interface can indicate the need for a cracked base analysis. Contours should be fairly smooth, if not, the finite element mesh used in the model may be too coarse and a finer mesh should be used. High stress concentrations around galleries or at changes in geometry may indicate the need for reinforcement.

3-4.5 Dynamic Methods

Earthquake loadings in areas of high seismicity may justify, or require, the use of dynamic analysis to properly assess a structure's response to ground accelerations.

Detailed discussions of the theory and practical development of these methods are presented in references 24 and 25. These methods require the use of complex matrix mathematics and computer programs.

Staff review of dynamic analyses should concentrate upon the computer model, the basis for selecting the design earthquake and the material properties used in the analysis. The computer model should accurately model the structure's geometry and, in particular, should include voids and any lumped masses which might affect the vibration mode shapes of the structure. Foundation interaction can significantly influence the dynamic response of the dam and should be included in the model. The model should also consider the hydrodynamic effects of reservoir-structure interaction. The variables which must be considered in the selection of these modeling parameters coupled with the uncertainties involved in the selection of material properties, can cause wide ranging differences in stress distributions within the dam. Therefore, several analyses may be required in order to test the sensitivity of the dynamic behavior to various combinations of properties and assumptions. The ultimate strengths of the materials may be used in earthquake analyses using dynamic methods.

Guidance concerning the review of design earthquakes is presented in references 4 and 34.

3-4.6 Cracked Base Analysis

A dam must be safe either with or without uplift; therefore, the concrete stresses and foundation reactions should be computed with and without uplift to determine the maximum conditions. Base pressures should be calculated excluding uplift. These pressures should then be compared to the uplift pressures at each point across the base to identify
any portion of the base not in compression (i.e., where the foundation reaction is less than the uplift pressure a "cracked" section is assumed and, thereby, subject to full headwater pressure). This condition requires that the uplift diagram be modified and a cracked base analysis be performed. See Appendix IIIB for further explanation of this procedure.

3-4.6.1 For Unconstructed Gravity Dams

This condition shall not be allowed for load Cases I, II or IIA but is permitted for load Cases III and IIIA. Cracked base analysis for load Cases III and IIIA shall be conducted as follows:

a. Base pressures shall be calculated without uplift, and compared to the uplift pressure at the heel of the dam. If the uplift pressure exceeds the base pressure, the base shall be considered cracked.

b. If the base is cracked, the eccentricity of the resultant of all forces, including uplift, shall be determined using the following expression:

\[ e' = \frac{\sum M + Mu}{\sum W - U} \]

where: \( \sum M \) = summation of moments of all external forces about centerline of the base, including earthquake but excluding uplift

\( Mu = \) moment of uplift force about centerline of the base

\( \sum W \) = summation of vertical forces without uplift

\( U = \) uplift force which existed immediately prior to the earthquake. \(^{22/}\)

c. Determine the uncracked portion of the base using the following expression:

\[ T_1 = \frac{3(L - e')}{2} \]

where \( T_1 = \) uncracked base length

\( L = \) total base length.

---

\(^{22/}\) Note: Due to the oscillatory nature of earthquake loading, uplift is assumed to be unaffected by seismic event.
d. Determine the toe pressure \((B_5)\) using the following expression:

\[
B_5 = \frac{2(W-U)}{T_1} + A_2
\]

where \(A_2 = \) tailwater pressure.

e. Only the uncracked portion of the base, \(T_1\), shall be used to determine the sliding stability.

f. Load Cases I, II and IIA shall be reevaluated for the post earthquake condition using the uncracked length determined in step C above. Sliding factors should be recalculated using only friction on the cracked portion of the base, and friction plus cohesion on the uncracked portion.

3-4.6.2 For Existing Gravity Dams.

References 1 and 7 provide detailed analysis procedures for cracked base analysis for load cases I, II and IIA. Paragraph 3-4.6.1 should be used for analysis of load cases III and IIIA.

3-4.7 Computer Programs - Review of Computer Studies

Computer studies submitted by applicants or licensees should include the name of the computer program used, and sample input data and output data. Program documentation should be requested for any program which has not been thoroughly tested and proven to be accurate.

Documentation should include samples of verification runs and a description of the analysis procedures used in the program. Well known programs such as: SAP4, STRUDL, ANSYS, etc., may be used without supporting documentation.

Input data should be checked in order to determine if the computer model will accurately predict the structural behavior of the prototype and the loads to which it will be subjected.

Output data should be spot checked and compared to hand calculated solutions wherever possible, in order to assure that the basic laws of statics have been satisfied, i.e., summation of forces and moments equals zero.
Stability Criteria

3-5.1 General

Specific stability criteria for a particular loading combination is dependent upon the type of analysis being done (i.e., foundation or concrete analysis), the degree of understanding of the foundation-structure interaction and site geology, and, to some extent, on the method of analysis. For unconstructed projects, preliminary analyses are generally based upon more conservative criteria than final designs. As the design process progresses the designer has available more sophisticated, and detailed, foundation information and material testing results. Therefore, the unknowns associated with the preliminary designs are reduced by the final design stage and somewhat lower safety factors may be acceptable.

For constructed projects, assumptions used in the analysis should be based upon construction records and the performance of the structures under historical flood loadings. In the absence of available design data and records, site investigations must be conducted to verify all assumptions.

3-5.2 Unconstructed Projects - Static Methods of Analysis

3-5.2.1 Case I, II or Case IIA Loading

The basic requirements for stability of a gravity dam, for these loading conditions are:

a. That it be safe against overturning at any horizontal plane within the dam, at the base or at any plane below the base. This requires that the allowable unit stresses established for the concrete and foundation materials not be exceeded. The allowable stresses should be determined by dividing the ultimate strengths of the materials by the appropriate safety factors in Table 2.

b. Tensile stresses at the rock/concrete interface and cracked base analyses shall not be allowed for these loading conditions.
# TABLE 2

**Recommended Factors of Safety**

**Dams having a high or significant hazard potential.**

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>Usual</td>
<td>3.0</td>
</tr>
<tr>
<td>Unusual</td>
<td>2.0</td>
</tr>
<tr>
<td>Extreme</td>
<td>Greater than 1.0</td>
</tr>
</tbody>
</table>

**Dams having a low hazard potential.**

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>Usual</td>
<td>2.0</td>
</tr>
<tr>
<td>Unusual</td>
<td>1.25</td>
</tr>
<tr>
<td>Extreme</td>
<td>Greater than 1.0</td>
</tr>
</tbody>
</table>

1/ Safety factors apply to the calculation of stress and the Shear Friction Factor of Safety within the structure, at the rock/concrete interface and foundation.

2/ Loading conditions as defined in paragraph 3-3.0.

3/ Safety factors are based upon the use of the gravity method of analysis. Safety factors should not be calculated for overturning, i.e., Mr/Mo.
c. That it be safe against sliding on any horizontal plane within the dam, on the foundation, or on any horizontal seam in the foundation. Reference 5, should be used to evaluate sliding stability.

3-5.2.2 Case III or Case IIIA Loading

When the extreme loading combination consists of an earthquake loading using the seismic coefficient method, the basic requirements for stability under Case I or Case II loading shall apply except that tensile stresses at the interface and the associated cracked base analysis will be allowed if the structure stabilizes when the cracked base analysis is conducted in accordance with Section 3-4.6 of this chapter. An analysis must be conducted of the post earthquake condition using the cracked base and modified material parameters to ensure stability under Cases I, II and IIA. The eccentricity of the resultant of all forces shall fall within the base.

3-5.3 Constructed project - Static Methods of Analysis

The requirements of section 3-5.2 shall apply except as modified below:

3-5.3.1 Tensile Stresses

Tensile stresses within the body of the dam should not exceed 4 to 6 percent of the compressive strength of the concrete. The tensile strength of horizontal lift joints within the dam may be less and testing may be required to establish allowable stresses when the analysis indicates zones of high stress.

The tensile strength of the rock-concrete interface should be assumed to be zero. The rock foundations may consist of adversely-oriented joints or fractures such that even if the interface could resist tension, the rock formation immediately below may not be able to develop the same strength. Therefore, since stability would not be enhanced by an interface with tensile strength when a joint, seam or fracture in the rock only a few inches or feet below the interface has zero tensile strength, no tension will be allowed at the interface.
3-5.3.2 Cracked Base Analysis

Cracked base will be allowed for all loading conditions, provided that the crack stabilizes within the base of the dam, and adequate sliding safety factors are obtained using only the uncracked portion of the base.

3-5.4 Safety Factor Evaluation

The safety factors determined in sections 3-5.2 and 3-5.3 shall be evaluated on a case-by-case basis in order to assess the overall safety of a particular project. On some projects, the recommended safety factors presented in Table 2 may not be attained for each and every load case. In these cases, engineering judgement must be applied to determine the acceptability of the calculated safety factor. Consideration must be given to the adequacy of the data presented in support of the analyses and the load case for which the safety factor does not meet the criteria. If the loading condition in question results in the worst loads on the structure, then a lower factor of safety may be acceptable, provided that an adequate analysis is supported by field data verifying critical material strengths and uplift assumptions.

For example, it may be acceptable to apply the factor of safety criteria for the unusual loading combination from Table 2 to the normal loading combination if the normal headwater and tailwater elevations result in the maximum net forces tending to indicate overturning of the structure. However, use of an overall lower safety factor for the normal loading condition must be supported by adequate field data and an acceptable analysis of the structure.

3-5.5 Alternate Criteria - Dynamic and FEM Analyses

3-5.5.1 Case I, Case II or Case IIA Loading

When an FEM analysis is conducted using equivalent static loads, the basic criteria of Section 3-5.2 shall apply. Conventional factors of safety for sliding and overturning should be determined by integrating the stress distributions at the structure-foundation interface to calculate resultant locations.
3-5.5.2 Case III or Case IIIA Loading

When the earthquake loading is calculated using dynamic or pseudo dynamic methods, the following criteria shall apply:

a. The dam shall be capable of surviving a Maximum Credible Earthquake (MCE) without a failure of a type that would result in loss of life or significant damage to downstream property. Inelastic behavior with associated damage is permissible under the maximum credible earthquake for the site.

The Maximum Credible Earthquake is defined as the severest earthquake that is believed to be possible at the site on the basis of geological and seismological evidence. It is determined by regional and local studies which include a complete review of all historic earthquake data of events sufficiently nearby to influence the project, all faults in the area, attenuations between causative faults and the site.

b. The dam shall be capable of resisting an Operating Base Earthquake (OBE) within the elastic range of the materials. The OBE is generally more moderate than the maximum credible earthquake and is selected on a probabilistic basis from regional and local geology and seismology studies as being likely to occur during the life of the project. It is generally as large as earthquakes that have occurred in the seismotectonic province in which the site is located.

See References 4 and 34 for detailed information concerning the above criteria.

3-5.5.3 Evaluation of Dynamic Analysis Results

Since tensile cracking is acceptable for the MCE loading, the significance of the tensile stresses which exceed the tensile strength of the concrete shall be evaluated as follows:

a. Sliding Stability Analysis. To evaluate the significance of cracking, perform a sliding stability analysis for the portion of the dam above the plane where the greatest cracking is expected. The earthquake loading should be the same as that used for the
stress analysis, zero cohesion should be assumed along the cracked portion of the plane, and the uplift loading should be the same as for the uncracked condition. The uplift is not increased during the earthquake-induced excitation due to the very short duration of this excitation.

b. Evaluation of Sliding Stability Results. If the sliding safety factor is equal to, or greater than, 1.0, adequate sliding resistance can be assumed. If the sliding safety factor is less than 1.0, compute an upper bound for the permanent displacement along the plane of investigation by assuming a horizontal crack the entire length of the sliding plane, and perform a partial nonlinear finite element analysis to compute the permanent displacement.

c. Partial Nonlinear Analysis. For an upperbound solution, perform a linear finite element acceleration-time history method of analysis with a special slip element to model the discontinuity along the crack. The slip element should have normal and tangential strength but no tensile strength.

The element should limit the magnitude of the shear stresses to the value allowed by the frictional force if the element is in compression. More than one cracked plane can be modeled at one time. Some general purpose finite element computer programs which include the slip element are SAP 80, ANSYS, ADINA, and ABAQUS.

d. Overturning Stability. During the earthquake-induced excitation, the overturning stability for a gravity dam can be assumed to be adequate if the overturning criteria for the static loading conditions are met. This mode of failure of a gravity dam due to an earthquake-induced excitation is not likely because of the large base width to height ratios of gravity dam cross-sections and the nature of the excitation—peak oscillatory motion of very short duration.
e. Stability After the MCE. The dam, in its post-earthquake condition, should be capable of containing the reservoir for a sufficient period of time to allow for strengthening of the dam, if necessary. For the normal static loading condition, the overturning and sliding stability should be checked assuming appropriate uplift in any cracked portion of the base as determined by the dynamic analysis. The resultant should be sufficiently within the base so that 50 percent of the unconfined compressive strength of the concrete is not exceeded and the sliding safety factor is 1.3 or greater.

f. Sequence of Analysis for the Operating Basis Earthquake (OBE). If no cracking is estimated to occur for the MCE loading, a stress analysis for the OBE is not required. However, effects of the OBE on the continued operation of essential equipment should be determined. If the level of the OBE is sufficiently high to require a dynamic analysis, the simplest linear elastic analysis appropriate for the monolith should be performed for the OBE loading using a five percent damping ratio. The combined maximum static and dynamic tensile stress should be no greater than 15 percent of the unconfined compressive strength of the concrete to assume no cracking occurs, provided the lift joints are sound.

3-5.6 Foundation Stability

3-5.6.1 Rock Foundations

The foundation or portions of it must be analyzed for stability whenever the structural configuration of the rock is such that direct shear failure is possible, or whenever sliding failure is possible along faults, shears and/or joints. Associated with stability are problems of local overstressing in the dam due to foundation deficiencies. The presence of such weak zones can cause problems under either of two conditions: (1) when differential displacement of rock blocks occurs on either side of weak zones, and (2) when the width of a weak zone represents an excessive span for the dam to bridge over. To prevent local overstressing, the zones of weakness in the foundation must be strengthened.
so that the applied forces can be distributed without causing excessive differential displacements, and so that the dam is not overstressed due to bridging over the zone. 23/

Sliding failure may result when the rock foundation contains nearly horizontal seams close to the surface. Such seams are particularly dangerous when they contain clay, bentonite, or other similar substances. Rock that is otherwise satisfactory may have to be removed in order to eliminate an objectionable seam below it. 24/

3-5.6.2 Soil Foundations

Gravity dams constructed on soil foundations are usually relatively small structures which exert low bearing pressures upon the foundation. Large structures on soil foundations are usually supported by bearing or friction piles and are beyond the scope of these guidelines. When the foundation consists of pervious sands and gravels, such as alluvial deposits, two possible problems exist; one pertains to the amount of underseepage, and the other is concerned with the forces exerted by the seepage. Loss of water through underseepage may be of economic concern for a storage or hydroelectric dam but may not adversely affect the safety of the dam. However, adequate measures must be taken to ensure the safety of the dam against failure due to piping, regardless of the economic value of the seepage. 25/

The forces exerted by the water as it flows through the foundation can cause an effective reduction in the weight of the soil at the toe of a dam and result in a lifting of the soil. If uncontrolled, these seepage forces can cause a progressive erosion of the foundation, often referred to as "piping" and allow a sudden collapse of the structure. The design of the erosion, seepage and uplift control measures requires extensive knowledge of type, stratification,

23/ Reference 1, Page 76.
24/ Reference 3, Page 8.
permeability, homogeneity, and other properties of the foundation materials. Some of the control measures which may be required may include some, all or various combinations of the following devices:

a. Upstream apron, usually with cutoffs at the upstream end.

b. Downstream apron, with scour cutoffs at the downstream end, and with or without filters and drains under the apron.

c. Cutoffs at the upstream or downstream end or at both ends of the overflow section, with or without filters or drains under the section.

A detailed discussion of these measures and their usages is given in reference 9. 26/

An indication of the susceptibility of a foundation to piping can be determined by calculating the hydraulic gradient at the toe of the dam. The critical hydraulic gradient, that which will cause piping of sand is approximately 1.0. The hydraulic gradients should be calculated for all dams with weighted creep ratios (as defined in Section 3-2.4.4.1) less than that required by Table 1.

This will require the construction of a flow net and the calculation of the factor of safety against piping at critical points in the foundation. If a factor of safety of less than 2.0 is obtained, the condition should be corrected by adjusting the base configuration or dimensions or by adding some means of cutoff which extends the seepage path. The factor of safety against piping shall be calculated by the following formula:

\[
\text{Safety Factor (SF)} = \frac{1}{i_s}
\]

where \( i_s \) = hydraulic gradient at the point under consideration

26/ See Reference 9, Pages 340 & 341.
3-6 Construction Materials

3-6.1 General

The material strength assumptions made during the design process must be verified by the site investigations and attained in the field during the construction process. Specific details concerning geological and foundation investigations and construction supervision will be presented in future guidelines. In the interim, the following guidance shall apply.

3-6.2 Site Investigations

The collection, study and evaluation of foundation data is a continuing program from the time of site selection to the completion of construction. Data collection starts with the assembly of general information concerning the site. This information is usually collected from field examinations of natural outcrops, road cuts and other prominent surface features. The amount of investigation necessary for appraisal will vary with the anticipated difficulty of the foundation. In general the investigation should be sufficient to define the major geologic conditions with emphasis on those which will affect design. 27/ Core drilling during this phase of the investigation is generally not necessary, but may be required for foundations with known problems, and for larger dams.

The following data should be obtained during the preliminary investigations and continually refined until the completion of construction: 28/

a. Dip, strike, thickness, composition, and extent of faults and shears.

b. Depth of overburden

c. Depth of weathering

d. Joint orientation and continuity

e. Lithology throughout the foundation

27/ Reference 1, page 13.
28/ Data collection recommendations from Reference 1, pg. 13.
f. Physical properties tests of the foundation rock. Tests performed on similar foundation materials may be used for estimating the properties in the preliminary phase.

During staff review of foundation investigation reports, staff geologists and soils engineers should be consulted concerning the adequacy of the data submitted with respect to defining the structural and geological capability of the foundation. Once construction has commenced, the geology as encountered in the excavation should be defined and compared with the pre-excavation geology. 29/ Any geologic changes should be carefully examined and consideration given to the possible impacts upon the design of the structure.

3-6.3 Concrete Properties

3-6.3.1 General

Many factors affect the strength and durability of mass concrete. The concrete must be of sufficient strength to safely resist the design loads throughout the life of the structure. Durability of the concrete is required to withstand the effects of weathering (freeze & thaw), chemical action and erosion. This strength and durability of the concrete must be uniform throughout the structure, because the weakest part will govern its structural adequacy.

3-6.3.2 Structural Properties

Stresses in a gravity dam are usually low; therefore, concrete of moderate strength is generally sufficient to withstand design loads. Laboratory tests should be performed on representative samples of actual production concrete to determine the following parameters:

a. Compressive strength at 7 days and 28 days. 30/
b. Tensile strength
c. Shear strength (cohesion and internal friction angle.)
d. Modulus of elasticity (sustained and instantaneous)
e. Poisson's ratio
f. Unit weight

Depending upon the stress conditions to which the concrete will be subjected, tests appropriate for the required material strengths should be performed at regular intervals throughout the dam construction in order to insure that uniform quality has been maintained.

For existing structures, drilling and testing of concrete should be performed wherever possible to verify analysis assumptions, especially when the analysis indicates high stresses or low safety factors.

Staff review of these tests should compare the laboratory results to the original design assumptions, and should examine the testing procedures to determine if the tests were conducted in conformance with recommended ASTM and ACI procedures as listed below:

a. Compressive tests; ASTM-C39
b. Tensile tests; ASTM C78
c. Shear tests; RTH 203-80 31/

30/ For the determination of allowable stresses within the body of a dam, the maximum ultimate compressive strength shall be limited to 4500 psi for use with Table 2.

31/ Reference 27, Page 173.
e. Poisson's ratio, ASTM C469

f. Collection of test samples:
   ASTM-C 31, C172, and C192

g. Evaluation of tests results:
   ACI 214

Additional guidance concerning the design of mass concrete mixes and the determination of the cured properties of the concrete are presented in reference 17.

3-6.3.3 Durability

The durability of concrete is influenced by the physical nature of the component parts, and although performance is largely influenced by mix proportions and degree of compaction, the aggregates constitute nearly 85 percent of the constituents in a mass concrete and good aggregates are essential for durable concrete. 32/ The environment in which the structure will exist must be considered in the mix design and in the evaluation of the suitability of aggregate sources proposed for use in the mix. Generally, the environmental considerations which must be examined are: weathering due to freezing and thawing cycles; chemical attack from reactions between the elements in the concrete, exposure to acid waters, exposure to sulfates in water and leaching by mineral-free water; and erosion due to cavitation or the movement of abrasive material in flowing water. Mix designs and material considerations, which will insure a durable concrete are presented in references 1 and 17. 33/

32/ Reference 33; applies only to the testing of rock core to concrete bond specimens.

33/ Reference 1, Pages 281 thru 285; Reference 17, paragraph 3.9.
3-6.3.4 Dynamic Properties

The apparent compressive and tensile strengths of concrete varies with the speed of testing. As the rate of loading increases compressive and tensile strengths also increase, therefore, the properties of concrete under dynamic loadings such as during an earthquake are greater than under static conditions.

a. For two and three dimensional finite element analyses using response spectra with a damping ratio of five percent, the combined maximum static and dynamic tensile stresses should be less than 15 percent of the unconfined compressive strength of the concrete. Some cracking may occur under tensile stresses of this magnitude, however, the dam can be considered to have adequate earthquake strength for MCE loadings.

b. If the combined maximum static and dynamic tensile stress using a five percent damping ratio exceeds 15 percent, but is less than or equal to 20 percent of the unconfined compressive strength, the analysis should be repeated using a seven percent damping ratio. Estimate the extent of cracking by assuming a crack to occur whenever the extreme fiber tensile stress exceeds 10 percent of the unconfined compressive strength and to propagate to the zero stress level along the plane of investigation.

c. If the combined maximum static and dynamic tensile stress using a five percent damping ratio exceeds 20 percent of the unconfined compressive strength of the concrete, follow the same procedure as described above, using a 10 percent damping ratio to estimate the extent of cracking.

3-6.4 Foundation Properties

In most instances, a gravity dam is keyed into the foundation so that the foundation will normally be adequate if it has enough bearing capacity to resist the loads from the dam. 34/ If, however, weak planes or zones of inferior rock are present within the foundation, the stability of the dam will be governed by the sliding resistance of the foundation. The foundation investigations should establish the following strength parameters:

34/ Reference 1, Page 15.
a. Shear and sliding strengths ($\phi$ and C) of the discontinuities and the rock.

b. Bearing capacity (compressive strength)

c. Elastic Modulus

d. Poisson's ratio

These parameters are established by laboratory tests on samples obtained at the site. In some instances, in situ testing may be justified. In either instance, it is important that samples and testing methods be representative of the site conditions. The results of these tests will, generally, yield ultimate strength or peak values and must, therefore, be divided by the appropriate factors of safety in order to obtain the allowable working stresses. Recommended factors of safety are presented in Table 2.

Foundation permeability tests should be conducted in conjunction with the drilling program, or as a separate study, in order to establish uplift parameters and to design an appropriate drainage system. Permeability testing programs should be designed to establish the permeability of the rock mass and not an isolated sample of the rock material. The mass permeability will usually be higher, due to jointing and faulting, than an individual sample. Guidance for staff review of permeability tests can be found in reference 26. [35]

Prior to the selection of allowable foundation strengths, for existing dams, all available geologic and foundation information should be reviewed for descriptions of the type of material and structural formation on which the dam is constructed. A general description of the foundation material can be used as a basis for choosing a range of allowable strengths from published data, if testing data is not available. Staff geologists should be consulted if the available information refers to material parameters or structural features which are suspected to be indications of poor foundation conditions. Some terms which should alert the engineer to possible problem areas are listed below:

a. Low RQD ratio (RQD = Rock Quality Designation).

b. Solution features such as caves, sinkholes and fissures.

c. Columnar jointing.
d. Closely spaced horizontal seams or bedding planes.
e. Highly weathered or fractured material.
f. Shear zones or faults and adversely oriented joints.
g. Joints or bedding planes described as slickensided, or filled with gouge materials such as bentonite or other swelling clays.
h. Foliation surfaces.

Compressive - In general, the compressive strength of a rock foundation will be greater than the compressive strength of the concrete within the dam. Therefore, crushing (or compressive failure) of the concrete will usually occur prior to compression failure of the foundation material. When testing information is not available this can be assumed, and the allowable compressive strength of the rock may be taken as equal to that of the concrete. However, if testing data is available, the safety factors from Table 2 should applied to the ultimate compressive strength to determine the allowable stress. Where the foundation rock is nonhomogeneous, tests should be performed on each type of rock in the foundation.

Tensile - A determination of tensile strength of the rock is seldom required because unhealed joints, shears, etc., cannot transmit tensile stress within the foundation. Therefore, the allowable tensile strength for the foundation should be assumed to be zero.

Shear - Resistance to shear within the foundation and between the dam and its foundation depends upon the cohesion and internal friction inherent in the foundation materials, and in the bond between concrete and rock at the contract surface. Ideally, these properties are determined in the laboratory by triaxial and direct shear tests and in the field through in situ testing. The possible sliding surface may consist of several different materials, some intact some fractured. Intact rock reaches its maximum break bond resistance with less deformation than is necessary for fractured materials to develop their maximum frictional resistances. Therefore, the shear resistance developed by each fractured material

depends upon the displacement of the intact rock part of the surface. If the intact rock shears, the shear resistance of the entire plane is equal to the combined sliding frictional resistance for all materials along the plane. 37/ The shear resistance versus normal load relationship for each material along the potential sliding plane should be determined by testing wherever possible.

In the absence of test results conservative values of the shear strength parameters should be assumed based upon the type and condition of the foundation material. If the analyses based on the conservative values indicate marginal safety factors, testing may be dictated to verify the appropriate cohesion values.

In the interim, staff geologists and soils engineers should be consulted concerning the adequacy of any foundation evaluation program.

References


8. American Concrete Institute, "Building Code Requirements for Reinforced Concrete", ACI-318-77.


3-8 Appendices
SEISMIC ZONE MAPS

<table>
<thead>
<tr>
<th>Map</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Contiguous States</td>
<td>A-2</td>
</tr>
<tr>
<td>California, Nevada and Arizona</td>
<td>A-3</td>
</tr>
<tr>
<td>Alaska</td>
<td>A-4</td>
</tr>
<tr>
<td>Hawaii</td>
<td>A-5</td>
</tr>
</tbody>
</table>

NOTE: Seismic zones for areas not shown in this appendix can be found in the seismic zone tabulation in section 1 of reference 20.
SEISMIC ZONE MAP OF THE CONTIGUOUS STATES AND PUERTO RICO

SEISMIC PROBABILITY

<table>
<thead>
<tr>
<th>ZONE</th>
<th>DAMAGE</th>
<th>COEFF.</th>
</tr>
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<tr>
<td>0</td>
<td>NONE</td>
<td>0.0</td>
</tr>
<tr>
<td>1</td>
<td>MINOR</td>
<td>0.05</td>
</tr>
<tr>
<td>2</td>
<td>MODERATE</td>
<td>0.10</td>
</tr>
<tr>
<td>3</td>
<td>MAJOR</td>
<td>0.15</td>
</tr>
<tr>
<td>4</td>
<td>GREAT</td>
<td>0.20</td>
</tr>
</tbody>
</table>

Figure A-1
SEISMIC ZONE MAP
CALIFORNIA, NEVADA & ARIZONA
Appendix IIIB

Cracked Base Analysis
Appendix IIIB

Cracked Base Analysis

1. General - The procedures for analyzing gravity dams recommended by these guidelines is basically the gravity method used by the Bureau of Reclamation (USBR). This method differs from that used by the Corps of Engineers primarily in procedures for the consideration of uplift forces. The USBR procedure separates uplift from the base pressure calculations, and compares the base pressure to the uplift pressure at the upstream face (minus the tensile strength of the lift joint) to determine whether a crack has been initiated. The Corps method includes uplift as a load on the base of the structure, and then determines "non-compressive" zones of stress to be cracked or tensile zones. This discussion is intended to clarify the differences in the two methods and to further explain the method recommended by these guidelines.

2. USBR Procedures - "Design of Gravity Dams", reference 1, recommends procedures identical to the Corps procedures for determining base pressures without uplift, i.e. basic static analysis, and therefore is consistent with the Corp's "Gravity Dams Design" manual reference 2. However, the USBR method assumes that uplift pressures below the dam are not redistributed along the dam foundation interface by the geometric properties of the interface, rather, the uplift should be added point-by-point to the base pressure diagram. The combined pressure diagram is thus determined by summing the base pressure and uplift diagrams. This diagram represents the total vertical force on the bottom of the dam.

The USBR uses the following equation to determine whether cracking of a lift joint has been initiated.

\[ Sc = \frac{pwh - ft}{SF} \]

Where: 
- \( Sc \) = minimum allowable compressive stress at upstream face
- \( p \) = reduction factor to account for drainage
- \( w \) = unit weight of water
- \( h \) = depth below water
- \( ft \) = tensile strength of concrete at lift surfaces
- \( SF \) = Safety Factor
This is a general equation meant to be applicable for use both at the interface and at lift joints within the body of the dam. The USBR recommends that p values be 1.0 if drains are not used and 0.4 if drains are used, and the values for SF are as shown in Table 2 of this chapter.

Equations 5, 6, and 7 on page 32 of reference 1 are then used to determine crack length and toe pressures. Attachment 1 shows the derivation of these equations.

3. Corps of Engineers Procedures - The Corps' EM-1110-1-2200, "Gravity Dam Design" (reference 2) also recommends that basic static procedures be used to analyze gravity dams. In paragraph 2-11, page 6 of this manual the following two statements are made:

"The assumed uplift pressure should be added to the computed unit foundation reaction to determine the maximum possible unit foundation pressure at any point."; and "Internal stresses and foundation pressures should be computed both with and without uplift to determine the maximum condition."

These two statements are consistent with the USBR recommended procedure of calculating base pressure without uplift and then adding the uplift diagram point-by-point.

In practice, the Corps applies uplift as a load in the structure and determines cracking potential by examining the resulting foundation pressure diagram to locate areas which are not in compression. An iterative process is then used which involves changing the uplift diagram to show 100% of headwater in non-compression zones and recalculating the foundation pressure. This procedure is continued until the remaining base is found to be in compression.

4. Differences in Results - The procedures recommended by the USBR and the Corps are basically the same and usually yield the same results. For all dams without drains and for some structures with drains the results will be identical. However, structures with high drain efficiencies and taller dams may show differing results when analyzed by the two methods. Attachment 2 illustrates such a case. The reason for the differences is that the Corps' procedure, by including uplift as a load, linearizes the uplift distribution and thereby distributes the headwater uplift pressure over the entire base. This causes some analyses to not indicate cracking using the Corps method while cracking is indicated using the USBR procedures.
5. **FERC Procedures** - These guidelines recommend that the cracking potential of the interface be determined using the USBR procedures. However, certain limitations contained in this chapter must be noted. The equation in paragraph 2 above, while general in nature, must be applied with care at the interface between the dam and foundation.

When using this equation to determine cracking potential at the interface, \( t \) is chosen as zero, and \( p \) will equal 1.0 unless the drains are located within 5% of the reservoir depth from the upstream face (See paragraph 3-2.4.3.1). Therefore, cracking will be predicted whenever the uplift pressure due to headwater exceeds the foundation pressure at the heel of the dam.

This procedure is more conservative than the Corps' method and is a special application of the USBR method in that the reduction for drainage is not always allowed at the upstream heel.
Attachment 1

Example Problem.
\[ \gamma_{mc} = 150 \text{pcf} \\
\gamma_{water} = 62.4 \text{pcf} \]

<table>
<thead>
<tr>
<th>Item</th>
<th>Weight (kips)</th>
<th>Moment Arm (ft)</th>
<th>Moment Act. &amp; (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Driving Force ( \frac{100(62.4)100(1/2)}{1000} )</td>
<td>312.0</td>
<td>33.33</td>
<td>10,400.0</td>
</tr>
<tr>
<td>Driv. = ( \frac{100(80)1/2}{1000} )</td>
<td>600.0</td>
<td>13.33</td>
<td>8000.0</td>
</tr>
<tr>
<td>( \pm W = 600.0 \text{kip} )</td>
<td>( \pm H = 312.0 \text{kip-ft} )</td>
<td>( \pm M = 2400.0 \text{kip-ft} )</td>
<td></td>
</tr>
<tr>
<td>Uplift ( \frac{3120(70)1/2}{1000} )</td>
<td>109.2 &amp; 6.67</td>
<td>728.0</td>
<td>( \checkmark )</td>
</tr>
<tr>
<td>3120(10) ( \frac{1/2}{1000} )</td>
<td>15.64</td>
<td>36.67</td>
<td>5720</td>
</tr>
<tr>
<td>3120(10)</td>
<td>31.24</td>
<td>35.0</td>
<td>10920</td>
</tr>
<tr>
<td>( \pm U = 158.0 \text{kip} )</td>
<td>( \pm M_u = 2392.0 \text{kip-ft} )</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\( \text{pcf} \) = pounds per cubic foot
**Corps Method**

\[ \Delta M = 2400.0 + 2392.0 = 4792.0 \text{ k}\cdot\text{ft} \]

\[ \Delta V = 600 - 156 = 444 \text{ k} \]

\[ \Delta M/\Delta V = 10.79 \quad L/6 = 13.33 \]

**Base Stresses**

\[ \text{heel} = \frac{444}{80} - \frac{4792(6)}{80} = 5.55 - 4.49 = 1.06 \text{ ksf} \]

Compression

\[ \text{toe} = 5.55 + 4.49 = 10.04 \text{ ksf compression} \]

**Entire Base in Compression**

**Bureau Method**

\[ \Delta M = 2400.0 \text{ k}\cdot\text{ft} \]

\[ \Delta W = 600 \text{ k} \]

\[ c = 4.0 \text{ to right of } \Delta \]

Heel pressure = \[ \frac{600}{80} (1 - 6(4)) = 5.25 \text{ ksf} \]

Uplift pressure = 6.24 ksf > 5.25 ksf

Therefore base is Cracked

\[ c' = \frac{2400}{600 - 6.25(80)} = 24.0 \]

\[ T_1 = 3(40 - 24) = 48 \text{ \& base uncracked} \]

\[ B_5 = \frac{2(600 - 6.25(80)) + 6.24}{48} = 10.41 \text{ ksf} \]

toe pressure

**Corps Method yields no cracking and Bureau method yields a 32' (40% of base) crack.**
IF uplift is not reduced by drainage

\[ U = \frac{6240(80)^{1/2}}{1000} = 249.6 \text{ k} \]

\[ M_u = 249.6(80) = 3328.0 \text{ k} \]

**Corps' Method**

\[ \Sigma M = 2400 + 3328 = 5728.0 \text{ k} \]

\[ \Sigma V = 600 - 249.6 = 350.4 \text{ k} \]

\[ \Sigma M/\Sigma V = 16.35 \text{ k} / \text{ k} = 13.33 \]

**Base Stresses**

\[ \text{heel} = \frac{350.4 - 5728.0}{80} = 4.38 - 5.37 = -0.99 \text{ ksf tension} \]

\[ \text{face} = 4.38 + 5.37 = 9.75 \text{ ksf compression} \]

**Bureau Method**

By inspection, base is cracked. Since 5.25 < 6.24 in previous example.

Results will be the same as before, i.e.

\[ c' = 24.0' \]

\[ T_f = 48.0' \]

\[ 85 = 10.41 \text{ ksf} \]

The two methods yield the same results when uplift is not reduced by drainage.
Attachment 2

Derivation of USBR Equations
Derivation of Bureau equations.

given:

\[
\begin{align*}
A & \quad e \\
B & \quad e \\
\end{align*}
\]

Summation of forces above & of base = \( \Sigma M \)

\( \Sigma M = \Sigma We \)

Base pressures without uplift

Pressures = \( \frac{\Sigma W}{T} (1 \pm \frac{6e}{T}) \)

\[
\begin{align*}
A & = \frac{\Sigma W}{T} (1 - \frac{6e}{T}) \\
B & = \frac{\Sigma W}{T} (1 + \frac{6e}{T})
\end{align*}
\]

Consider uplift

assume \( A3 > A1 \); therefore base is cracked.
combined (cracked base) pressure diagram

\[ \text{cracked base uplift} \rightarrow T/2 \rightarrow \text{foundation reaction} \]

Figure 1

Draw free body diagram, the resultant must move to new location in order to resist increased load.

Figure 2

and applied moments

The weight is resisted by a combination of uplift and foundation reaction, however, for purposes of calculating the unknowns \( e' \) and \( T_i \), consider only the geometry of the combined diagram. The diagram can be separated into a rectangle and a triangle as follows:

Figure 3
From the free body diagram of statics
from \( \Sigma V = 0 \):  
\[
\Sigma W = \text{area of combined diagram} \\
\text{or } \Sigma W = A_3 \cdot T + (B_5 - A_3) \frac{T_1}{2}
\]

Solve for \( B_5 \)

\[
\Sigma W - A_3 \cdot T = (B_5 - A_3) \frac{T_1}{2} \\
2(\Sigma W - A_3 \cdot T) = B_5 - A_3 \\
\frac{2(\Sigma W - A_3 \cdot T)}{T_1} + A_3 = B_5 \text{ - equation (7) page 32}
\]

from \( \Sigma M_\psi = 0 \)

sum moments about \( \psi \) of base.

Use Figure 3.

\[
\Sigma M_\psi = A_3(T) \cdot \phi + (B_5 - A_3) \frac{T_1}{2} \left( \frac{T}{2} - \frac{T_1}{3} \right)
\]

set \( e' = \frac{T}{2} - \frac{T_1}{3} \) - equation 4

then \( \Sigma M_\psi = (B_5 - A_3) \frac{T_1}{2} (e') \)

substitute for \( B_5 \) and solve for \( e' \)

\[
\Sigma M_\psi = \left( \frac{8(\Sigma W - A_3 \cdot T) + A_3}{T_x} - A_3 \right) \frac{T_1}{2} (e')
\]

\[
\Sigma M_\psi = \left( \frac{8\Sigma W - A_3 \cdot T} {T_x} \right) e' \text{ - equation 5 page 32}
\]

or \( e' = \frac{\Sigma M_\psi}{\Sigma W - A_3 \cdot T} \)
Now, knowing \( e' \), Equation 4 can be solved for \( T_1 \):

\[
e' = \frac{T_2 - T_3}{2}
\]

\[
T_1 = 3 \left( \frac{T_2 - e'}{2} \right)
\]

Equation 6, page 32

Now compare results from equations to iterative method of analysis - use simplified structure:

- \( \delta_{concr} = 150 \text{pcf} \)
- \( \delta_{wate} = 62.4 \text{pcf} \)

- \( W_T = 80(100) + 50(\frac{1}{2}) = 600 \text{ k} \)
- \( H = 100^2 (62.4) = \frac{312 \text{ k}}{2} \)

\[\begin{align*}
T_1\sum L &= 312 (100) - 600 (40 - 80\%) = 10400 - 8000 = 2400 \text{ k'f} \\
\delta &= 2400/600 = 4.0' \\
A_1 &= \frac{600}{80} (1 - 6(4)) = 5.25 \text{ ksf} \\
\text{uplift pressure} &= \frac{100 (62.4)}{1000} = 6.24 \text{ ksf} > 5.25 \text{ ksf} \\
\text{equations} \\
e' &= \frac{2400}{600 - 6.24(80)} = 23.81' \\
T_1 &= 3 (40 - 23.81) = 48.57' \\
BS &= \frac{2 (600 - 6.24(80))}{48.57} + 6.24 = 10.39 \text{ ksf}
\end{align*}\]
**Iteration**

**Uplift Force (before crack)** = 6.24(80) = 249.6 k

**Uplift Moment** = 249.6(40 - 80/3) = 3328 k⋅f

ΣM = 2400 + 3328 = 5728 k⋅f

ΣV = 600 - 249.6 = 350.4 k

ε = 16.35'

A1 = \[\frac{350.4}{80} \left(1 - \frac{6(16.35)}{80}\right)\] = -1.10 ksf (Tension)

ε2 = 9.75 ksf

**Stress Diagram**

\[
x = \frac{1}{10.75} = 7.44'
\]

1st estimate of crack length = 10'

**Uplift Diagram**

\[
\text{Force} = 6.24(10) + 6.24(70) = 280.8 \text{ k}
\]

\[
\text{Moment} = 62.4(85) + 18.4(6.67) = 3640.7 \text{ k⋅f}
\]

ΣM = 2400 + 3640.7 = 6040.7 k⋅f

ΣV = 600 - 280.8 = 319.2 k; ε = 18.92'

A1 = \[\frac{319.2}{70} \left(1 - \frac{6(18.92)}{70}\right)\] = -0.88 ksf (Stress at end),

Crack must be longer.
2nd estimate of crack length = 25' (55' uncracked)

\[ \text{uplift force} = 6.24(25) + 6.24(55) = 327.6 \text{ k} \]
\[ \text{uplift moment} = 156(27.5) + 17.16(-3.33) = 3717.6 \text{ k}\text{l} \]
\[ 2M = 2400 + 3718.6 = 6118.6 \text{ k}\text{l} \]
\[ EV = 600 - 327.6 = 272.4 \text{ k} \]
\[ e = 22.46', e' = 22.46 - 12.5 \]
\[ A1 = \frac{272.4}{55}(1 - \frac{6(7.96)}{55}) = -0.43, k\leq F \leq \phi \]

\[ \therefore \text{Crack must be longer} \]

3rd estimate of crack length = 40' (40' uncracked)

\[ \text{uplift force} = 6.24(40) + 6.24(40) = 374.4 \text{ k} \]
\[ \text{uplift moment} = 249.6(20) + 124.8(-13.33) = 3328.0 \text{ k}\text{l} \]
\[ 2M = 2400 + 3328 = 5728 \text{ k}\text{l} \]
\[ EV = 600 - 374.4 = 225.6 ; e = 25.39', e' = 25.39 - 20 \]
\[ A1 = \frac{225.6}{40}(1 - \frac{6(5.39)}{40}) = 1.03, k\leq F > \phi \]

\[ \therefore \text{Crack length is less than 40'} \]

From curve drawn through 3 points, it appears that crack is about 31' long.
Try 31'
4\textsuperscript{th} quarter crack length = 31', uncracked = 49' 

\begin{align*}
\text{uplift} &= 31(6.24) + \frac{6.24(49)}{2} = 346.3' \\
\text{moment} &= 193.44(24.5) + 152.88(-7.33) = 3618.2' \\
\Sigma M &= 3618.2 + 2400 = 6018.2' \\
\Sigma V &= 600 - 346.3 = 253.7' \\
L &= 253.7 \left(1 - \frac{6(8.2)}{49}\right) = 0.03 \text{ ksf} < 0.5' \\
\text{crack length is about 31'} \\
\text{From equation crack is } 80 - 48.57 = 31.43'
\end{align*}
DRAFT

ENGINEERING GUIDELINES

FOR

THE EVALUATION OF

HYDROPOWER PROJECTS

CHAPTER IV

ISSUED APRIL 1987

FEDERAL ENERGY REGULATORY COMMISSION

OFFICE OF HYDROPOWER LICENSING
ABOUT THIS PUBLICATION

This is the fourth chapter of a FERC publication entitled Engineering Guidelines for the Evaluation of Hydropower Projects. Chapters I, II and II, issued in October 1986, can be purchased at $9.50 per copy from the Public Reference Section, Room 1000, at the address shown below. Additional chapters will be issued as they are completed.

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MAIL TO: Engineering Guidelines for Evaluation of Hydropower Projects - IV
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NOTE: The following chapter of Engineering Guidelines for the Evaluation of Hydropower Projects is undergoing peer review by independent consultants and is therefore subject to revision and clarification.
These engineering guidelines have been prepared by the Office of Hydropower Licensing (OHL) to provide technical guidance to the Staff in the processing of applications for license and in the evaluation of proposed changes or additions to existing projects under the jurisdiction of the Federal Energy Regulatory Commission (Commission).

The guidelines are intended to provide personnel of the Office of Hydropower Licensing, including the Regional Office and Washington Office technical personnel, with procedures and criteria to be used in engineering review and analyses of projects over which the Commission has jurisdiction. In addition, these guidelines should be used in the evaluation of consultant or licensee conducted studies. The guidance is intended to cover a majority of studies usually encountered by the Staff. However, special cases may require deviations from, or modification of, the guidelines. When such cases arise, Staff members must determine the applicability of alternate criteria or procedures based upon their experience and must exercise sound engineering judgment when considering situations not covered by the guidelines. The alternate procedures, or criteria, used in these situations should be justified and accompanied by any suggested changes in the guidelines that may be necessary to incorporate such procedures in future revisions. Staff technical personnel consist of the professional disciplines (e.g. professional engineers and geologists) required to perform such studies. Since every dam site and hydropower related structure is unique, individual design consideration and construction treatment will be required. Technical judgment is therefore required in most analytical studies.

These guidelines are primarily intended for internal use by OHL Staff, but also provide the criteria that licensee and applicants should use in any studies presented to the Commission under Parts 4 and 12 of the Regulations (18 CFR, Parts 4 and 12). When any portion becomes outdated, obsolete, or needs revision for any reason, they will be revised and supplemented as necessary. New pages will be prepared and issued with instructions for page replacements.
Chapter IV
Embankment Dams

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Appendices

Appendix 4-A Engineering Data
Appendix 4-B Procedures For Evaluating Embankment Dams
for Seismic Stability, Liquefaction Potential, and Deformation
1.0 Purpose and Scope

1.1 General

The guidelines presented in this chapter provide staff engineers with recommended procedures and criteria to be used in reviewing and evaluating the safety of existing and proposed earth and rockfill (embankment) dams. The review performed by staff engineers will be conducted to ensure that all decisions, methods, and procedures performed by licensees, or their consultants, are sound regarding dam safety, and to ensure that accepted, up-to-date state-of-the-art procedures are consistent with the Commission's Dam Safety Program objectives (the term licensees also refers to applicants for license where appropriate).

The evaluation of the safety of new and existing embankment dams present special and unique problems. Existing dams may prove difficult to analyze especially where the dam was designed before the development of modern design and construction technology was developed or where adequate records are not available. Even for a relatively new dam, where records are extensive, evaluation can be cumbersome for the following reasons: (a) various levels of completeness of records, (b) different site conditions, (c) varying degrees of quality in design and construction, and (d) differing depth of evaluation required for each dam. The objective set forth in this chapter is to provide systematic procedures performing staff evaluations.

1.2 Depth of Review

The review of existing dams will not be as detailed as the procedures involved in the design of new dams. Instead, the review is intended to evaluate procedures and methodology of design and analysis to ensure that safe and adequate embankment dams have been constructed. The licensee's or its consultant's investigations and evaluations should
be examined to determine if all areas of importance were considered and appropriate design criteria have been used.

For proposed dams, the licensee will be required to submit a design report in accordance with the Commission's Regulations. This report should be critically examined to determine if all appropriate design criteria have been met.

During the investigation and evaluation for both proposed and existing dams, important areas to consider are as follows:

- The embankment must be safe against overtopping by wave action and during inflow design flood conditions.

- The slopes must be stable during all conditions of reservoir operations, including rapid drawdown.

- Seepage flow through the embankment, foundation, and abutments must be controlled so that no internal erosion (piping) takes place and there is no sloughing in areas where seepage emerges.

- The embankment must not overstress the foundation.

- Embankment slopes must be protected against erosion.

- The embankment and foundation must be stable under the most severe earthquake conditions that can reasonably be expected. 1/

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1/ Reference 1, pg. 210, 211
For existing dams, an independent analysis of the embankment stability or adequacy need not necessarily be performed by staff. The data presented by the licensee should be reviewed to determine if they are correct and if the latest information has been considered. The criteria used by the licensee or its consultant should be consistent with any changed conditions discovered during onsite examinations such as loadings, seepage, increased pore pressures in the dam or the foundation, erosion, etc.

For proposed dams, an analysis of the stability and adequacy is required unless specifically exempted by the Commission. The methods and procedures used in the evaluation of any embankment should be consistent with the latest, accepted state-of-the-art methods and criteria, and with guidance contained in this chapter of the Guidelines.

1.3 References

Any criteria and methods of evaluation and analysis used in reviewing licensee's reports should be based on criteria and procedures established in literature published by the Corps of Engineers or Bureau of Reclamation, or in other recognized engineering references. Selected references are listed in Section 7.0.

2.0 Sources of Data and Information

To properly evaluate all information and data presented in the licensee's design report, various available FERC reports should also be reviewed. Available reports include:

- Prelicense Inspection Reports of existing dams and/or Site Inspection Reports of proposed damsites
- Operation Reports
- Construction Reports
- Independent Consultant's Safety Inspection Reports
One or more of the above listed reports should be available for licensed projects. If a license has not previously been issued, the staff engineer performing the review should refer to the Prelicense Inspection Report prepared by the staff engineer responsible for the project in the Regional Office.

For existing dams, additional data may be available from the facility owner, previous owners, state or local agency if the facility is a publicly owned project and from the state agency responsible for dam safety, such as Department of Water Resources, Department of Environmental Resources, Division of Dam Safety or Department of Natural Resources. Also, technical information may be available from Corps of Engineers Phase I Inspection Reports of public or private entities having impounding structures upstream or downstream of the facility.

For proposed dams, the source of information will generally be the licensee and/or its consultants and engineers. For all proposed dams, the licensee will be required to provide staff with all data necessary to evaluate whether the design of the structure is safe and adequate.

Data that may be available from the sources referenced should include:

- Logs of drill holes, test pits, and exploratory trenches
- Site geologic reports
- Site seismicity reports
- Materials exploration and testing reports
3.0 Review of Existing Data

Appendix 4-A is a listing of various engineering data related to the design, construction, and operation of an embankment dam. Prior to review and analysis of existing data, this appendix may be useful in organizing the data as discussed in the Safety Evaluation of Existing Dams (SEED) Manual.

The engineer performing the review should examine all data to determine if potential problem areas have been recognized and methods are proposed for correction. Additionally, the data should be examined to determine if the source of any current conditions or problems, such as seepage, settlement, cracking, etc., are evident from existing data. The methodologies and criteria used in the design should be examined and compared to the present state-of-the-art procedures and criteria. Advances in state-of-the-art methodologies may require a reevaluation of the original design. The SEED Manual discusses in greater detail specific information to look for in the reports and data that may be available.

Reference 2
4.0 Need for Supplemental Information

The objective of reviewing existing data is to be in a position to use as much information as is available to evaluate the structural adequacy of existing or proposed embankment dams. Data should be the prevalent basis for judgments on dam safety. If potentially hazardous conditions are believed or determined to exist and the existing data are insufficient to resolve the problem, it may be necessary to request supplemental investigations, analyses, or information to complete the evaluation. The supplemental information could involve additional visual inspections, measurements, foundation exploration and testing, and materials testing. Conditions that may require supplemental information are as follows:

- Significant cracking, settlement or sloughing of an existing embankment and the potential for such in any proposed structure.

- Uncontrolled seepage conditions through the embankment, the abutments, or at the toe area, and the potential for such in any proposed structure.

- The use of available data with state-of-the-art analytical methods.

5.0 Evaluation of Embankments

The safety of an embankment dam is dependent primarily on two items, the absence of excessive deformations under all conditions of environment and operation, and the control of seepage to prevent migration of materials and thus preclude adverse effects on stability.
To properly evaluate the stability of an embankment dam, the following characteristics of the structure should be reviewed. For existing dams, the review should include all records of its past behavior in addition to these characteristics:

- Embankment zoning
- Seepage control measures and records
- Deformation, predicted or recorded
- Erosion control measures
- Structural stability analyses
- Liquefaction potential
- Soil properties
- Overtopping potential

5.1 Embankment Zoning

There are two types of embankment dams, homogeneous and zoned. Zoned dams are generally preferred since zoning permits the use in the embankment of several different material types that may be available from borrow areas or required excavations. Homogeneous embankments are usually not recommended except when free-draining materials are not readily available.

For zoned embankments, the zoning geometry and properties of the materials placed in the zones should be reviewed to determine: (1) the structural design, and (2) the types of internal features such as chimney drains, blanket drains, toe drains, etc., that are proposed or were used to provide for and maintain embankment stability. One should keep in mind that embankment zoning is also established for economic

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3/ Reference 1, pg. 209, 210
reasons according to the availability of materials. 4/ The embankment zoning should provide an adequate impervious zone, transition zones between the core and the shells, and seepage control zones. Desirable characteristics that these zones should have or provide are as follows:

a. For the impervious zone (core), it is desirable that the width of the core at any elevation be equal to or greater than one-third the height of the dam above that elevation. The minimum top width of the core should not be less than 10 feet. 5/ For dams less than 50 feet high, the width to height ratio of the impervious core at any elevation in the dam should not be less than 1.0. 6/ The coefficient of permeability of the core material should be $10^{-4}$ cm/sec or less.

b. Transition zones must meet filter criteria to protect the adjacent zones from piping. The transition zones should be sufficiently wide to ensure that they are continuous and constructable without contamination. 7/ 8/

c. Seepage control features within the embankment should be sized adequately to contain all seepage flows. The features should also be sufficiently pervious to ensure that all seepage will be intercepted and controlled. 9/

4/ Reference 1
5/ Reference 3, pg. 5-3
6/ Reference 1, Chapter 6
7/ Reference 4, pg. 57, 606
8/ Reference 1, Chapter 6
9/ Reference 3, pg. 5-3
d. Zoning of an embankment that places the more pervious material to each side of the core zone is preferable. This placement improves the stability of the embankment during rapid drawdown conditions and keeps the downstream slope drained for greater effective weight. 10/

Homogeneous dams may also have seepage control features including a transition zone between the main embankment material and the drain. Desirable characteristics listed in paragraph 5.1 b and c also apply to the features of this type of structure. The homogeneous structure is usually more massive and usually has flatter slopes than a zoned embankment of the same height. These characteristics compensate for a less effective weight of soil due to a higher phreatic line in the homogeneous embankment. They also provide greater slope stability during rapid drawdown. 11/

5.2 Seepage Control Measures

All embankment dams are subject to some seepage through, under and around them. 12/ If uncontrolled, seepage may be detrimental to the stability of the structure as a result of excessive internal pore water pressures or by piping. 13/ For existing dams, records or evidence that seepage flows have removed any significant degree of fine grained material must be evaluated. Any such record requires further field investigation. The predicted and/or recorded loss of water due to seepage may be of economic significance for

10/ Reference 5, pg. 7
11/ Reference 1, pg. 209
12/ Reference 5, pg. 1
13/ Reference 3, pg. 1-6
hydroelectric projects if quantities are relatively large. Otherwise seepage should be effectively controlled to preclude structural damage or interference with normal operations.

In the evaluation of seepage reduction or seepage control measures as they pertain to dam safety, one should review and evaluate the following:

a. Protective control measures such as relief wells, weighted graded filters, or horizontal drains which prevent seepage forces from endangering the stability of the downstream slope. 14/

b. Filters and transition zones designed to prevent movement of soil particles that might clog drains or result in piping. 15/ 16/

c. Drainage blankets, chimney drains, and toe drains designed to ensure that they control and safely discharge seepage for all conditions. 17/

d. Contacts of seepage control features with the foundation, abutments, embedded structures, etc., designed to prevent the occurrence of hydrofracturing of embankment and/or foundation materials. 18/

14/ Ibid., pg. 1-6, b
15/ Reference 4, pg. 57
16/ Reference 1, pg. 235
17/ Reference 3, pg. 1-6
18/ Reference 1, Appendix E
e. Measures such as compaction requirements, seepage collars, placement of special materials, or other similar features to prevent internal erosion from seepage at the interface with concrete structures. 19/ 20/ If seepage collars are present, special attention should be given to compaction requirements around them. The use of seepage collars is not recommended for incorporation into the design of new structures.

f. For existing embankments, all seepage records compiled during the existence of the structure should be reviewed for significant trends or abnormal changes. The causes of these abnormalities should be determined as accurately as possible.

5.3 Deformation, Predicted or Recorded

The type, amount, and rate of deformation of an embankment, either vertical or horizontal movement, must be estimated during the design stage and should be recorded during the operation of the structure. For proposed embankments, the structure must be cambered to allow for the estimated settlement during the life of the structure. For existing embankments, any evidence or records of unusual settlement, cracking, or movement should be reviewed to determine whether these conditions are detrimental to the continued safe operation of the structure. Field investigations may be required to determine the causes of these abnormalities. These investigations may involve such items as surveying the structure, installing movement detecting instruments, or excavating test pits for examination, etc. 21/

19/ Ibid.
20/ Reference 3, Chapter 2
21/ Reference 4, Chapter 12
As a result of deformation, cracking can develop through the impervious core section below the line of saturation which may result in piping. Adequately sized and graded filter zones located downstream from the impervious core can prevent piping. 22/ Corrective measures or instrumentation may be needed if adequate filter zones do not exist.

5.4 Erosion Control Measures

Upstream and downstream slopes, the toe area, groin areas at the abutments, approach and discharge channels, and areas adjacent to concrete structures should be protected against excessive erosion from wave action, surface runoff, and impinging currents. Inadequate erosion protection can result in slope instability. 23/ Some common types of protection used are riprap, gabions, paving (concrete or asphalt), and appropriate vegetative cover.

The slope and toe protection of all embankment dams should be reviewed to determine if the dam is adequately protected against erosive forces. If the slope protection is being continually displaced, heavier protection is required. Additionally, if embankment materials, consisting of silty and sandy soils, are being moved into the slope protection, measures must be taken to correct this condition before erosion becomes detrimental to the embankment. A bedding layer must be designed according to established filter criteria and placed under the riprap protection. 24/

22/ Ibid., Chapter II
23/ Reference 3, Chapter 5
24/ Reference 1, Chapter 6
5.5 **Structural Stability Analyses**

The evaluation of the structural stability of embankment dams shall be based on all available design information for proposed structures and on design and construction information and records of performance for existing embankments. The Corps of Engineers Guidelines for Safety Inspection of Dams 25/ can be used as a guide in performing the review.

Stability studies and analyses for proposed embankments will be conducted during design in accordance with methods discussed in Section 6.0. For existing embankments, the initial stability studies and analyses will normally be acceptable if they were performed by these methods. Additional stability analyses should be performed if initial design analyses do not exist or are incomplete, if existing conditions and hazard potential of the project have changed since construction occurred, or if the embankment has been subjected to loading conditions more severe than originally envisioned during the design. Satisfactory behavior of the embankment under loading conditions not expected to be exceeded during the life of the structure should generally be indicative of satisfactory stability, provided adverse changes in the physical condition of the embankment has not occurred. 26/

Evidence of any adverse changes which may affect the stability of an embankment can be obtained from visual inspection and observation of available instrumentation data covering such items as changes in pore water pressures, displacements, changes in loading conditions, seepage, etc. Review of

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25/ Reference 6
26/ Reference 6, pg. 10
maintenance records and related information may also provide a reference to structural behavior data for a particular structure. Should a review of project records indicate possible deficiencies in the stability of an embankment, additional information may be required regarding the foundation and the embankment materials. The Corps of Engineers Guidelines for Safety Inspection of Dams 27/ and other available literature 28/ 29/ 30/ 31/ 32/ 33/ 34/ can be referred to in establishing the information necessary to determine the condition and material properties of the foundation and embankment.

5.6 Potential for Liquefaction

The phenomenon of liquefaction of loose saturated sands having a contractive structure may occur when such materials are subjected to shear deformation with high pore water pressures developing, resulting in a loss of resistance to deformation.

The potential for liquefaction in an embankment or its foundation must be evaluated on the basis of empirical knowledge and engineering judgment supplemented by special laboratory tests when necessary. Simplified methods for evaluating soil liquefaction potential are used by Seed

27/ Reference 6
28/ Reference 3
29/ Reference 5
30/ Reference 7
31/ Reference 8
32/ Reference 9
33/ Reference 10
34/ Reference 15
and Idriss 35/ and Castro 36/ to relate blow count values from standard penetration tests to safe, unsafe, and marginal conditions. For the evaluation of embankment dams, these methods can be employed in conjunction with guidelines given in Section 6.0 for stability analyses.

5.7 Soil Properties

Soil properties including strength and seepage parameters to be used as input data for stability analyses should be realistically representative of the range and variation that exist in the foundation, abutment, and embankment materials. 37/ For information concerning the foundation and embankment soils, refer to the procedures established in the Corps of Engineers and Bureau of Reclamation Guidelines 38/ 39/ 40/ 41/, and other literature 42/ 43/ 44/. The selection of the proper input parameters and their correct use in a stability analysis are more important than the mechanics of the stability analyses.

35/ Reference 12
36/ Reference 13
37/ Reference 14
38/ Reference 8
39/ Reference 9
40/ Reference 2
41/ Reference 15
42/ Reference 16
43/ Reference 4
44/ Reference 17
5.8 Embankment Overtopping Potential

All embankment dams, either proposed or existing, should be evaluated for overtopping potential under the most extreme conditions expected. Chapter II of these Guidelines discusses the Spillway Design Flood and provides freeboard criteria. The maximum reservoir elevation determined for the design flood and expected wave runup are conditions that should be considered. However, a less severe storm with lower reservoir elevation but greater wave propagation may result in conditions that are more critical than those produced by the design flood. There should be no overtopping of the embankment.

6.0 Stability Analysis

6.1 General

As discussed in paragraph 1.2, a new, independent stability analysis by staff is not necessarily required for a proposed or existing embankment. Spot checks of analyses may be required to verify that application of the specific analytical approach is correct. The analysis and evaluation of the structural adequacy of an embankment dam by the licensee and/or its consultant should be reviewed based on information formulated by the licensee and information developed by the Regional Office staff from various project inspections and data requests resulting from the licensing or inspection program. For embankment dams, stability analyses should be examined to determine if the criteria used and loading conditions analyzed are appropriate based on review of the above information and if methods of analyses used are acceptable based on the present state-of-the-art, and if proper types of failure surfaces have been analyzed (i.e., wedge and/or circular).
An independent stability analysis should be performed by staff if physical conditions differ from those assumed in the licensee's analysis, if soil parameters are inconsistent with material types, if soil parameters and pore water pressures are inconsistent with the method being used, or if the critical failure surface does not appear to have been determined.

Staff presently has available a Corps of Engineers stability computer program prepared by the Waterways Experiment Station based on the Modified Swedish Circle Method of Analysis. This program can be used by staff in reviewing the results of the licensee's analysis. It should, however, be understood that the results obtained by this method of analysis may not necessarily agree exactly with the licensee's results based on other methods; however, it will provide an indication as to the adequacy of the analysis being reviewed. Staff is not limited to the use of the computer program based on the Modified Swedish Circle Method of Analysis and, therefore, may also use other available computer programs.

A brief discussion is included in this section of the Engineering Guidelines concerning some methods of stability analysis and why the results obtained from each method may vary to some degree. Additionally, references are listed in Section 7.0 that analyze the various methods of stability analyses in detail. A historical development of methods of stability analyses is presented in Reference 16.

45/ Reference 16
46/ Reference 16, pp. 323-326
6.2 **Review Approach**

Stability analyses should be reviewed to determine if input data appear appropriate based on a knowledge of the embankment and foundation materials, on pore pressures in the embankment and its foundation, or if the method of analysis chosen by the licensee is being used correctly. The literature provides several publications, textbooks, and other sources of information that discuss in detail the various methods of analyses available. Refer to Section 7.0 for references that can be used in obtaining information for use in reviewing a particular method of stability analysis. 47/

For small embankment dams on stable foundations where it may not be necessary to perform detailed independent stability analyses, the reviewer should use information assembled by the Bureau of Reclamation 48/ in determining whether embankment slopes are adequate. However, if seepage, slope erosion, sloughing, piping, high pore water pressures, or other deficiencies exist and are not addressed, the licensee should be requested to analyze the conditions.

A review of the stability analysis presented by the licensee shall include an evaluation and summary of the data used in the analysis and an evaluation to determine if the critical conditions have been investigated. The items to be evaluated include:

47/ References 20 and 26
48/ Reference 1, pg. 265, 267
The soil densities and shear strengths to be used for the various loading conditions investigated can be evaluated by studying available laboratory test data and/or comparing data presented to that known for similar materials based on past experiences and on data available from other dams consisting of similar materials.

Pore water pressures used in the analysis of the various loading conditions investigated should be reviewed to determine if they are realistic based on available instrumentation data or estimates based on such methods as those proposed by Casagrande 49 and Carstens and May 50.

When field explorations and laboratory testing are required to provide additional information concerning the strength characteristics of the embankment materials, the sampling and laboratory testing procedures should be reviewed to determine if they were adequately accomplished and are representative of the conditions analyzed. Corps of Engineers and Bureau of Reclamation technical guidelines concerning sampling and laboratory testing procedures can be used to complete this review. 51 52 53

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49/ Reference 18
50/ Reference 19
51/ Reference 9
52/ Reference 10
53/ Reference 15
6.3 Conditions to be Investigated

An embankment and its foundation are subject to shear stresses imposed by the weight of the embankment and by pool fluctuations, seepage, or earthquake forces. Loading conditions vary from the commencement of construction of the embankment until the time when the embankment has been completed and has a full reservoir pool behind it. The range of loading conditions encompasses the following conditions at various stages from construction through the operational stage of the completed embankment:

- Construction
- Sudden drawdown
- Partial pool with steady seepage
- Steady seepage
- Earthquake

In all loading cases, the shear strength along any potential failure surface must be defined. The shear strength available to resist failure along any particular failure surface depends on the loading conditions applied.

6.4 Shear Strength

Generally, the shear strengths of materials used in stability analyses are determined from laboratory testing procedures which attempt to duplicate the various loading conditions to which the embankment is expected to be subjected. 54/ 55/ 56/

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54/ Reference 11
55/ Reference 16
56/ Reference 20
From the time construction begins until the reservoir has been filled and a state of steady seepage has been established, three different conditions of drainage will have occurred. Shear strength values used in stability analyses for each condition of drainage are determined from laboratory tests on specimens of the material which are compacted to the density and water content that simulates the conditions anticipated in the dam. Tests corresponding to the three conditions of drainage are:

(a) \( \sigma \) or unconsolidated-undrained (UU) test in which no initial consolidation is allowed and the water content is kept constant during shear.

(b) \( R \) or \( \bar{R} \) consolidated-undrained (CU) tests in which consolidation is allowed under initial stress conditions, but in which the water content is kept constant during application of shearing stresses. The \( R \) test is identical to the \( \bar{R} \) test except that pore water pressure measurements are made during the test.

(c) \( S \) or consolidated-drained (CD) test in which consolidation is permitted under the stress conditions and also for each increment of loading during shear.

References:

57/ Reference 16
58/ Reference 11
59/ Reference 10, pp. 328-338
60/ Reference 16
61/ Reference 20
6.4.1 Laboratory Testing

Testing procedures for determining the shear strengths of soils to be used in stability analyses, as well as determining other engineering properties of soils, such as density, moisture content, consolidation, permeability, gradation, etc., can be found in Corps of Engineers and Bureau of Reclamation manuals. 62/ 63/ When reviewing analyses of existing embankments only R and S shear strength parameters are considered. For proposed dams, shear strength parameters obtained from the O test will also be used.

6.4.2 O - Unconsolidated-Undrained Shear Strength

The O test is performed on specimens of impervious materials under simulated loading conditions expected to occur during construction of embankments and results in an approximation of the end-of-construction shear strength of the material.

6.4.3 R and R̄ - Consolidated-Undrained Shear Strength

The R and R̄ tests apply to conditions in which impervious or semipervious soils that have been fully consolidated under one set of stresses are subjected to a stress change during the test without time for consolidation to take place (soil is sheared without allowing dissipation of pore pressures).

62/ Reference 10
63/ Reference 15
6.4.4 S - Consolidated-Drained Shear Strength

The shear strength resulting from an S test is obtained by fully consolidating the soil specimen under the applied confining stress and, when drainage is complete, applying shear stresses slowly enough to allow full drainage to occur during the shearing process under each loading increment.

6.5 Types of Stress Analyses

In general there are two types of stress analyses that are used in the evaluation of existing and proposed embankments. These are the total stress analysis and the effective stress analysis. The total stress analysis is used in the design of embankments for loading conditions during construction, rapid drawdown, and earthquake. The effective stress analysis is generally used to evaluate embankments during and after construction when piezometer observations are available, for partial pool and steady seepage loading conditions, and for conditions existing in the embankment and foundation after consolidations has occurred.

6.6 Loading Conditions for Analysis

As outlined in Section 6.3, an embankment may be subjected to several loading conditions during its life, ranging from construction to full pool operation. The loading conditions for which an embankment must be analyzed are five in number and are presented in detail in the following paragraphs.
6.6.1 **End of Construction Loading Condition**

At the end of construction, an embankment dam is still undergoing internal consolidation under its own weight. For homogeneous dams or for zones in dams constructed from impervious materials, pore water pressures will be built up during construction due to the inability of the impervious soil mass to drain rapidly during consolidation.

The shear strength applicable to the impervious dam or zones within the dam during the construction loading condition, is determined by the O test conducted at field moisture contents. The type of stress analysis that applies to this loading condition is the total stress analysis. Because of the difficulty in estimating pore water pressures within the embankment during this stage of loading, an effective stress analysis is not recommended. For pervious zones in the embankment where drainage can occur rapidly, S strengths should be used in the analysis.

The end of construction analysis using shear strengths obtained from the O test as representative of the strength available in the impervious zones of an embankment, represents a lower limit of stability since consolidation is progressing during the course of construction.
6.6.2 Sudden Drawdown Loading Condition

In the sudden drawdown loading condition the structure has been subjected to a prolonged high pool during which time a steady seepage condition has been established through the embankment. The soil in the embankment below the phreatic surface is in a completely saturated state and is fully consolidated under the weight of the overlying material. If subsequently the reservoir pool is drawn down faster than pore water can escape, excess pore water pressures develop. Consequently, the reduced factor of safety following a reservoir drawdown is due primarily to the existence of higher pore water pressures (drawdown pore water pressures) acting inside the upstream slope. The shear strength is governed by the state of stress developed during consolidation under buoyant weight before drawdown.

The shear strength parameters required for an analysis under this loading condition are obtained from the R test. An expression needs to be determined for relating consolidation pressure before shear to the undrained shear strength. Laboratory tests are performed under consolidated-undrained conditions, in which the samples are consolidated under stresses corresponding to the conditions immediately preceding the drawdown. If the material being investigated can drain so rapidly as to dissipate practically

64/ Reference 16, pg. 370
65/ Reference 20, pg. 26
66/ Reference 4, pg. 258
67/ Reference 20, pp. 23-27
all the excess pore water pressure as the drawdown progresses 68/, the drained or S strength is the strength used in the analysis. This type of analysis is referred to as a total stress analysis.

If an effective stress analysis is conducted, one method of measuring the effective stress parameters is to perform consolidated-undrained triaxial tests on the soil with the measurement of pore pressure. This type of test is referred to as an R test. The accuracy of this type of analysis rests in how well the pore pressures can be estimated. If R tests are run on undisturbed samples retrieved from an existing embankment, results of pore-pressure observations in the field can be used in determining pore pressure coefficients to be used in the R testing procedure. For further discussion on differences between total and effective stress analyses refer to References 16 and 4. Laboratory procedures for the R and R tests are discussed in Reference 10.

When conducting a sudden drawdown analysis the Corps of Engineers uses shear strength based on the minimum of the combined R and S envelopes (figure 1). 69/ Shear strengths of free-draining materials where dissipation of pore water pressure can proceed as the reservoir pool is drawn down will be based on the S shear strength envelope of the material.

68/ Reference 4, pg. 258
69/ Reference 11
The unit weights of the soils to be used in analyzing the "before drawdown" condition will be the moist weights above the line of saturation and submerged weights below. In analyzing the "after drawdown" condition, moist unit weights will be used for the zone above the original phreatic surface, saturated unit weights will be used within the drawdown zone, and submerged weights will be used below the level of drawdown.

\[ \Phi = \text{Angle of internal friction based on total stresses} \]

\[ \Phi' = \text{Angle of internal friction based on effective stresses} \]

Figure 1
6.6.3 Steady Seepage Loading Condition

Steady seepage develops after a reservoir pool has been maintained at a particular elevation (e.g., maximum storage pool) for a sufficient length of time to establish a steady line of saturation through the embankment. The seepage forces which develop in the steady state condition act in a downstream direction. The condition of steady seepage throughout an embankment may be critical for downstream slope stability. The seepage forces can be estimated by assuming a horizontal phreatic line through the impervious zone at the elevation of the storage pool intersected by zones of free-draining material. However, high abutment groundwater tables may cause the phreatic surface to be higher in the vicinity of the abutments. In homogeneous impervious embankments, the line of seepage can be estimated by various methods. Examples of estimating the line of seepage through an embankment are given in Reference 5. If sufficient instrumentation is available, piezometer levels in both the embankment and foundation can be reviewed and phreatic surfaces can be developed accordingly.

70/ Reference 11, pg. 19
71/ Reference 18
72/ Reference 19
The pore water pressures which exist within an embankment at any given time are generated as the result of two actions which can be considered independent for practical purposes: (1) gravity seepage flow, and (2) changes in pore volume due to changes in the total stresses. 73/ The full reservoir stability condition is nearly always analyzed using the effective stress method of analysis and the pore water pressures acting are assumed to be those governed by gravity flow through the embankment. 74/

For design purposes, the Corps of Engineers generally uses the shear strength of impervious soils corresponding to a strength envelope midway between the R and S test envelopes when the S strength is greater than the R strength and to the S envelope when the S strength is less than the R strength (figure 2). The shear strength of freely draining cohesionless soils should be represented by the S test envelopes. 75/

The unit weights to be used in the analysis will be the moist unit weight above the line of saturation and submerged weights below this line.

73/ Reference 16
74/ Ibid.
75/ Reference 11, pg. 18
In the case where a steady seepage condition exists in an embankment, an additional horizontal thrust may be imposed by a surcharge pool up to the probable maximum pool elevation, generally not for a prolonged period of time. Thus the impervious zone would not become saturated above the steady state condition established under normal reservoir conditions. The shear strengths to be used in the stability analyses should be the same as those used in the steady seepage case with maximum storage pool.

6.6.4 Partial Pool Loading Condition

The same information applies to the partial pool loading condition as to the steady seepage loading condition except that the upstream slope is also analyzed.
6.6.5 **Earthquake**

Evaluations of seismic effects for embankments located in areas of low or negligible seismicity shall be accomplished using the seismic coefficient method of analysis. Seismic coefficients at least as large as shown in figures 6, 6a, 6b, and 6c of Reference 11 shall be employed as applicable. The seismic coefficient method assumes that the earthquake causes additional horizontal forces in the direction of potential failure. This investigation need only be applied to those critical failure surfaces found in analyzing loading conditions without earthquake loading. An analysis of earthquake loading is seldom necessary in conjunction with sudden drawdown stability analysis. However, if earthquake loading is possible during reservoir drawdown associated with a pumped storage project where frequency of drawdown occurs on a daily cycle, earthquake effects during sudden drawdown should be investigated.

For embankments located in areas of strong seismicity, a dynamic response analysis should be performed based on present state-of-the-art procedures. Refer to Corps of Engineers ER 1110-2-1806, "Earthquake Design and Analysis for Corps of Engineers Dams," for the earthquake loading to be used in dynamic response analyses and for guidance in performing seismic evaluations.

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76/ Reference 11, change 1, dated 17 February 1982
In general, an embankment dam should be capable of retaining the reservoir under conditions induced by the maximum credible earthquake where failure would cause loss of life. The following investigations should be accomplished for all proposed and existing embankments. 77/

a. A seismic stability investigation using dynamic methods of all proposed and existing dams located in Seismic Zones 3 and 4.

b. An evaluation of the liquefaction potential for all dams that have or will have liquefiable materials either in the embankment or foundation.

c. A geological and seismological review of existing dams in Seismic Zones 2, 3, and 4, to locate faults and ascertain the seismic history of the region around the dam and reservoir.

d. A seismic stability investigation of existing dams by dynamic methods regardless of the seismic zone in which the dam is located where capable faults or recent earthquake epicenters are discovered within a distance where an earthquake would cause structural damage.

77/ Reference 21, Chap. IV, App. 4-B
6.7 Factors of Safety

The factor of safety includes a margin of safety to guard against ultimate failure, to avoid unacceptable deformations, and to cover uncertainties associated with the measurement of soil properties or the analysis used. In selecting a minimum acceptable factor of safety an evaluation should be made on both the degree of conservatism with which assumptions were made in choosing soil strength parameters and pore water pressures, and the influence of the method of analysis which is used. The latter concerns the method of calculation in which side earth forces are considered and how assumptions of directions of side earth forces affect stability analysis results.

The degree of safety against ultimate failure may be defined as:

\[
\text{Factor of Safety} = \frac{\text{Strength}}{\text{Stress}}
\]

or

\[
\text{F.S.} = \frac{\tau_f}{\tau}
\]

where

\[
\tau_f = \text{shear strength along the same shear surface}
\]

\[
\tau = \text{equilibrium shear stress along the same shear surface}
\]

---

78/ Reference 22, pg. 48
79/ Reference 16, pp. 368-371
A qualitative estimate of the factor of safety can be obtained by examining conditions of equilibrium when incipient failure is postulated, and comparing the strength necessary to maintain limiting equilibrium with the available strength of the soil.

Therefore, the slope stability analysis of soils requires measurements of the shear strength and computation of the shear stress. Appropriate minimum values of factors of safety to be used in the stability analysis of a slope depend primarily on the measurement of strength. Factors influencing the selection of minimum factors of safety include:

- Reliability of laboratory shear strength testing results
- Embankment height
- Thoroughness of investigations
- Construction quality, construction control of embankment fills
- Judgment based on past experience
- Design conditions being analyzed
- Predictions of pore water pressures used in effective stress analyses

Reference 23
FERC minimum factors of safety listed in Table 1. Final accepted factors of safety should depend upon the degree of confidence in the engineering data available. In the final analysis, the consequences of a failure with respect to human life, property damage, and impairment of project functions are important considerations in establishing factors of safety for specific investigations.

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Minimum Factor of Safety</th>
<th>Slope to be Analyzed</th>
</tr>
</thead>
<tbody>
<tr>
<td>End of construction condition</td>
<td>1.3</td>
<td>upstream and downstream</td>
</tr>
<tr>
<td>Sudden drawdown from maximum pool</td>
<td>&gt;1.0*</td>
<td>upstream</td>
</tr>
<tr>
<td>Sudden drawdown from spillway crest or top of gates</td>
<td>1.2*</td>
<td>upstream</td>
</tr>
<tr>
<td>Steady seepage with maximum storage pool</td>
<td>1.5</td>
<td>downstream</td>
</tr>
<tr>
<td>Steady seepage with surcharge pool</td>
<td>1.4</td>
<td>downstream</td>
</tr>
<tr>
<td>Earthquake (for steady seepage conditions with seismic loading using the seismic coefficient method)</td>
<td>&gt;1.0</td>
<td>upstream and downstream</td>
</tr>
<tr>
<td>Earthquake (for all dynamic analyses using a deformation method)</td>
<td>&lt;2 feet of Newmark-type deformation along the potential failure plane</td>
<td></td>
</tr>
</tbody>
</table>

* The factor of safety should not be less than 1.5 when drawdown rate and pore water pressures developed from flow nets are used in the stability analyses and where rapid drawdown is a normal operating condition as with pumped storage reservoir.
6.8 Static Stability Analysis

Various analytical methods for evaluating the static stability of an embankment dam exist. The method utilized in the licensee's analysis should be consistent with the anticipated mode of failure, dam cross section, and soil test data.

6.8.1 Limit Equilibrium

Many methods of stability analyses exist that use the same general approach of employing the "limit equilibrium method" of slope stability analysis. In this type of approach a quantitative estimate of factor of safety can be obtained by examining the conditions of equilibrium when incipient failure is postulated, and comparing the strength necessary to maintain limiting equilibrium with the available strength of the soil. The factor of safety (F.S.) is thus defined as the ratio of the total shear strength available (S) on the failure surface assumed to the total shear stress mobilized (τ) along the failure surface in order to maintain equilibrium. 81/

\[
F.S. = \frac{S}{\tau} \quad (1)
\]

A state of limiting equilibrium exists when the shear strength mobilized is expressed as:

\[
\tau = \frac{1}{F.S.} (S) \quad (2)
\]

81/ Reference 24
F.S. is a factor of safety with respect to shear strength and 1/F.S. is the degree of mobilization of the shear strength. It may be shown that the definition of F.S. given by equation (1) is equivalent to the one used in the Ordinary Method of Slices, where the factor of safety is defined as the ratio of the resisting moment to the overturning moment. 82/

The shear strength of a soil is expressed by the following expression:

$$s = c + \sigma \tan \theta$$

in which c and $\sigma$ equal the Mohr-Coulomb shear strength parameters, where c represents the cohesive resistance of the soil and $\sigma$ represents the normal stress on the shear surface. Thus, to determine the shear strength along a potential failure surface the normal stress on the shear surface must be known. In analyzing both force and moment conditions of equilibrium it becomes apparent that the problem of determining the distribution of the normal stress on the shear surface is statically indeterminate, that is, there are more unknowns than there are equations of equilibrium. 83/ An approach to this situation is to make assumptions to reduce the number of unknowns in order that the problem is statically determinate, such as is done in the "limit equilibrium" analysis procedures. Different procedures use different assumptions. Some methods do not satisfy all

82/ Reference 25, pg. 784
83/ Reference 25
conditions of equilibrium, such as moment equilibrium or vertical and horizontal force equilibrium. Table 2 shows equilibrium conditions satisfied by various methods of analysis.

<table>
<thead>
<tr>
<th>Procedure</th>
<th>Overall Moment</th>
<th>Individual Slice Moment</th>
<th>Vertical Force</th>
<th>Horizontal Force</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary Method of Slices</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>Bishop's Modified Method</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>Janbu's Generalized Procedure of Slices</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Spencer's Procedure</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

Studies have been performed to examine the accuracy of the equilibrium methods of slope stability analysis.  

6.9 **Seismic Stability Analysis**

Various methods of analyses are available for evaluating the seismic stability of an earth dam. These may be classified as:

(a) Pseudostatic methods

(b) Simplified procedures

(c) Dynamic response analyses

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84/ Reference 25, pp. 783-791
85/ Reference 26, pp. 475-498
Regardless of the method of analysis, the final evaluation of the seismic safety of the embankment should be based on all pertinent factors involved in the investigation and not solely on the numerical results of the analysis. References presented in the Corps of Engineers ER 1110-2-1806 can be used in determining the scope of analysis required for properly assessing the seismic stability of an embankment dam. In addition, procedural guidelines for dynamic methods of seismic stability analyses are outlined in Appendix 4-B to these guidelines.

6.9.1 General Approach

Analyses for earthquake loading should begin with simplified procedures and proceed to more rigorous methods of analysis as a particular situation may warrant. In general, a pseudostatic method of analysis shall be used with conservative estimates of data and information pertinent to the analysis when materials are known to be non-liquefiable or where structures are proposed or existing in areas of low seismicity. In areas of severe and/or frequent seismic loading such as in seismic Zones 3 and 4 or where foundation liquefaction potential exists, more rigorous dynamic methods of analyses may be necessary.

---

86/ Reference 21 and 33
87/ Reference 33
6.9.2 Pseudostatic Analysis Procedures

For several years the standard method of evaluating the safety of embankment dams against sliding during earthquakes has been the pseudostatic method of analysis. In using this approach no special consideration has been given to the nature of the slope-forming or foundation materials and if the computed factor of safety was larger than unity, it has generally been concluded that the seismic stability question has been satisfactorily resolved. In Terzaghi's opinion, depending on the slope-forming materials, a slope may remain stable if the factor of safety is less than unity and may fail if the factor of safety has been found to be greater than unity based on the pseudostatic approach.

In general, earthquake analyses using the seismic coefficient method shall be performed for all structures proposed or existing in Seismic Zones 1 and 2. Seismic coefficients at least as large as shown in the Corps of Engineer ER 1110-2-1806, should be employed in the analysis. In Zones 3 and 4 and in other zones where an initial analysis does not demonstrate the embankment to be safe, more sophisticated analyses should be performed.

88/ Reference 27, pg. 220
89/ Ibid.
90/ Reference 33
6.9.3 Simplified Analysis

Dr. H. Bolton Seed has drawn various conclusions following a close study of embankment dam performance during earthquakes. As a result of his study, Seed has determined that the seismic resistance of dams constructed of clayey soils is much higher than those constructed of saturated sands or other cohesionless soils. Due to limitation of the pseudostatic analysis approach and difficulties in evaluating the resistance of saturated cohesionless soils, methods of evaluating deformations in earth dams were developed. The computed displacements can be compared to allowable displacements to determine the adequacy of the embankment. Methods for evaluating deformations have been developed by Seed and Newmark.

When embankments and/or their foundations are composed of sandy soils, an analysis must be performed to determine if a liquefaction potential exists. There are various simplified methods available for evaluating soil liquefaction potential which have established empirical correlations between in situ behavior of sands and standard penetration resistance. In addition, methods exist to assess the liquefaction

91/ Reference 27, pg. 227
92/ Reference 28
93/ Reference 29
94/ Reference 12
95/ Reference 30
96/ Reference 13
potential of a soil by determining whether the soil is contractive or dilative. Under cyclic loading of sufficient magnitude and duration, a loose saturated sand having a contractive structure will develop high pore water pressures, lose a large portion of its resistance to deformation, and flow. See Appendix 4-B for additional information.

6.10 **Dynamic Analysis**

Dynamic response analyses using state-of-the-art methods shall be conducted for those embankments which are located in Zones 3 or 4, or are constructed of materials which could undergo significant loss of strength due to loading such as hydraulic fill dams or dumped fill dams. Earthquake parameters should be established from seismological and geological studies and dynamic properties of the soils should be determined from laboratory and field testing procedures for use in a dynamic stability analysis.

The results of dynamic analyses should be assessed on the basis of whether or not the dam would have sufficient residual integrity to retain the reservoir during and after the most adverse earthquake which may occur near the project location. Appendix 4-B should be consulted whenever results of dynamic analyses are reviewed.

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97/ Reference 31
98/ Reference 32
99/ Reference 6, pg. 19
7.0 References


APPENDIX 4-A

ENGINEERING DATA

This appendix lists engineering data which should be collected relating to the design, construction, and operation of an embankment dam to be used in establishing the adequacy of embankment structures.

1. General Project Data

a. Construction dates.

b. Design of structures.

c. As-built drawings indicating plans, elevations, and sections of embankment and appurtenant structures.

d. Information on any modifications made, if applicable, such as dam raising.

2. Geotechnical Data

a. Regional and site seismicity.

b. Foundation data and geological features including logs or borings, geological profiles and cross sections, and reports of foundation treatment.

c. Engineering properties assigned to construction materials and the foundation for design purposes including results of laboratory tests, field permeability tests, construction control tests, and assumed design properties for materials.
3. **Construction History**

   a. Construction procedures and methods used.
   
   b. Properties and characteristics of construction materials.
   
   c. How was quality control measured and maintained?
   
   d. Final foundation and embankment reports.

4. **Operation and Maintenance Records**

   a. Performance record to date based on instrumentation observations and surveillance reports.
   
   b. Comparison of conditions to which embankment has been subjected, to those assumed in the original design.
   
   c. Remedial measures undertaken during life of project.
   
   d. Known deficiencies and any work underway to correct deficiencies.

5. **Inspection History**

   a. Operation inspections reports.
   
   b. Safety inspections reports.
APPENDIX 4-B

PROCEDURES FOR EVALUATING EMBANKMENT DAMS
FOR SEISMIC STABILITY, LIQUEFACTION POTENTIAL, AND DEFORMATION
APPENDIX 4-B

PROCEDURES FOR EVALUATING EMBANKMENT DAMS
FOR SEISMIC STABILITY, LIQUEFACTION POTENTIAL, AND DEFORMATION

Introduction

The purpose of this appendix is to establish procedures for evaluating the seismic stability, liquefaction potential, and deformation potential of embankment dams. This appendix is not intended to provide an exhaustive list of procedures, but rather a guide to ensure consistency within FERC for evaluating embankment dams for seismic liquefaction and deformation.

Seismic Evaluations

The term "project" applies to all embankment dams, associated embankment structures, and their foundations. All projects must be evaluated for the effects that seismic loadings will have on them.

a. Projects with well-compacted embankments and foundations located in seismic zone 1 or 2 [ref. 1] and all confirmed low hazard potential projects may be evaluated by the pseudostatic method using the seismic coefficient assigned to the seismic zone the project is in.

b. Site-specific seismic studies will be performed for all projects not covered in Paragraph a. above. These studies will identify earthquake source areas and the maximum credible earthquake and the distance from the site for each relevant source area. Potential for fault rupture in the dam foundation and in the reservoir will be assessed.
Fig. 2, Par. 13.58

Chapter 13
Seismic Design
and Analysis

Embankment Dams

DRAFT

NO LIQUEFACTION AT SITES SUSCEPTIBLE
LiqEFACTION TO LIQUEFACTION
- THRESHOLD BASED ON CHINESE DATA
LIGEFACTION FROM JAPANESE EARTHQUAKES
- LOWER BOUND BELOW WHICH NO LIQUEFACTION SHOULD OCCUR

SEISMIC POTENTIAL AT SITE VS. EMBRICAL LIQUEFACTION OCCURRENCE

EARTHQUAKE - RICHTER MAGNITUDE

EPICENTRAL DISTANCE (km)

1 2 3 4 5 10 20 50 100 200 500 1000
Methods of Analysis

The sequence for evaluating the effects of seismic load on a project is as follows:

a. Determine the appropriate seismic coefficient or the maximum credible earthquake magnitude, distance, and site-ground motion parameters as appropriate for the project.

b. Liquefaction analyses are performed to predict embankment and foundation responses to earthquake. Prior to determining the strength of the deposits, a quick evaluation of the potential for liquefaction may be made by examining the record of liquefaction occurrence versus the seismic potential of the site [fig. 1 from ref. 5]. The lower bound of these occurrences provides a reasonable limit for the potential occurrence of liquefaction for sites with poor soil conditions. The empirical record presented on figure 1 reflects the experience in three seismically active area of the world (China, Japan, and western United States). If conditions at the site are not represented by these three area, adjustments may be required in the use of figure 1, or it may be inapplicable. If the magnitude and distance for the site fall on or above the boundary, then the potential for liquefaction exists and an evaluation of the soil deposits is required.

The procedure currently used by FERC [ref. 2] is to determine the material strength properties by conducting standard penetration tests (SPT). All hydraulic-fill projects and suspected loose-fill projects will be evaluated for liquefaction.
The resistance afforded by these strength properties is then compared to the resistive capacity required as appropriately determined from the seismic zone coefficient or from the magnitude and distance of the relevant earthquakes. Empirical correlations or analytical methods may be used to establish the required strength. Projects that are determined to be potentially susceptible to liquefaction may be further evaluated using a dynamic analysis [ref. 6, 7] and strengths of materials which can be determined by correlating SPT blow counts to soil strengths to show the limits of liquefaction, if any occurs.

Should liquefaction be determined to be a problem, corrective action must be taken to correct this problem if the project is not a confirmed low hazard potential project. If liquefaction is not found to occur or to be a problem, then further evaluation for seismic-induced deformation and a seismic stability analysis should be made.

c. Deformation analyses like liquefaction analyses are performed to predict embankment and foundation response to earthquakes. For projects not subject to liquefaction, deformation should not be a problem if the following conditions are satisfied:

(1) The dam is well-built (densely compacted) and peak accelerations are 0.2g or less [ref. 8];

(2) The slopes of the dam are 3 horizontal to 1 vertical or flatter;
(3) The static factors of safety of the critical failure surfaces involving the crest (other than infinite slope case) are greater than 1.5 under loading conditions expected prior to an earthquake;

(4) Freeboard at the time of the earthquake is a minimum of 2 to 3 percent of the embankment height (not less than 3 feet).

If these conditions are not satisfied, a deformation analysis should be made. This analysis can be made using the Newmark approach, a simplified Newmark procedure [ref. 4], or by more rigorous methods such as a finite element (strain potential) approach. The deformation calculated along the failure plane by these methods should not exceed 2 feet.

The basic steps involved in conducting a state-of-the-art type of an analysis are as follows:

(1) Determine the magnitude and source of the earthquake or earthquakes that should be considered;

(2) Determine the time-history or time-histories of the ground motion associated with the earthquake or earthquakes;

(3) Determine the properties of the embankment and foundation materials;

(4) Determine the dynamic response of embankment and foundation materials by case history comparisons, comparisons with other tests and/or dynamic testing;
(5) Predict the extent of structural deformations resulting from earthquake shaking; and

(6) If predicted deformations are not tolerable, explore design alternatives that would provide a tolerable response.

d. Seismic stability analyses are used to evaluate the ability of embankment dams to resist seismic loadings. As indicated in Seismic Evaluations, some embankments may be evaluated for seismic stability by using the seismic coefficient. The seismic coefficient is then applied in a pseudostatic analysis to determine a factor of safety. The minimum acceptable factor of safety for this type of analysis is $F.S. > 1$.

All other embankment dams not subject to the pseudostatic analysis must be analyzed by an acceptable state-of-the-art type of dynamic analysis. For an analysis of this type, the embankment should have a minimum factor of safety of $F.S. > 1$ during the earthquake and a minimum of $F.S. = 1.3$ immediately after the earthquake stops.

References


Convention, St. Louis, Missouri, October 1981.


### ROUTING AND TRANSMITTAL SLIP

**Date:** 1-13-89

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<th>(Name, office symbol, room number, building, Agency/Post)</th>
<th>Initials</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Larry Wolf</td>
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<td></td>
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</table>

**REMARKS**

Attached is a copy of change 2, dated 12/2/88 to Chapter III "Gravity Dams" of the "Engineering Guidelines for the Evaluation of Hydropower Projects".

DO NOT use this form as a RECORD of approvals, concurrences, dispositions, clearances, and similar actions.

**FROM:** (Name, org. symbol, Agency/Post)  
**Phl Mabini**

**Room No.—Bldg.**

**Phone No.**  
(503) 322-5842
Attached is Change 2, dated December 2, 1988, to Chapter III "Gravity Dams" of the "Engineering Guidelines for the Evaluation of Hydropower Projects".

Paragraph 3-5.5 of Chapter III of the Engineering Guidelines provides guidance on the evaluation of safety factors determined for gravity dams and Table 2 of Chapter III lists acceptable safety factors for various loading conditions.

At the time the guidelines were developed, a higher level of conservatism was considered appropriate. In many cases, decisions on the stability of gravity structures were based on data that may have been less extensive than one might consider desirable. Therefore, the lack of data was offset by more conservative safety factors.

Since the issuance of the current regulations in 1981 (Order No. 122), we have emphasized to dam owners the importance of gathering adequate data for evaluating the stability of dams. This emphasis on data was further reflected in the Engineering Guidelines. Over time, dam owners have begun to approach safety evaluations with better data and analysis. Simultaneously, investigatory and analytical methods and research on dam safety have improved. Given this background, we have concluded that it is appropriate to modify the safety factor criteria for gravity dams for the Probable Maximum Flood (PMF) loading condition. In reaching the decision to modify the safety factor for the PMF, the state-of-the-art in investigatory, testing, and analytical methods were taken into account along with an increased willingness on the part of dam owners to use state-of-the-art methods.

Sincerely,

Ronald A. Corso
Ronald A. Corso, Director
Division of Dam Safety and Inspections
Chapter III, Gravity Dams
Change 2, December 2, 1988

The following changes should be made to Chapter III, "Gravity Dams" of the "Engineering Guidelines for the Evaluation of Hydropower Projects". Note that the page and paragraph numbers apply to the October 1988 reprinting of the July 1987 edition of the guidelines.

1. Replace page 3-ii with new page 3-ii.

2. Replace pages 3-36 through 3-43 with new pages. The changes on these pages are as follows:
   a. The stability criteria has been revised and reorganized to distinguish between analyses conducted using static methods of analysis and those using dynamic methods.
   b. Allowable tensile stresses within the body of the dam has been increased to 10 percent of the unconfined compressive strength for static methods of analysis.
   c. A new section 3-5.4 has been added to emphasize the need for a post earthquake stability analysis.
   d. Section 3-5.5, Safety Factor Evaluation (formerly section 3-5.4) has been rewritten to cite examples of acceptable deviations from the safety factors recommended in Table 2, and gives the supporting data requirements for use of these deviations.

3. Replace pages 3-50 through 3-52 with the new pages. Section 3-6.3.4 has been revised to require rapid rate testing to establish the concrete properties to be used in a dynamic analysis of a new design. The concrete properties to be used in the dynamic analysis of an existing project may be determined in accordance with currently accepted industry practice as referenced.

4. Replace page 3-57 with the new page 3-57. Reference number 37 has been added.
# Chapter III
Gravity Dams

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c. Determine the uncracked portion of the base using the following expression:

\[ T_1 = \frac{3(L - e')}{2} \]

where: \( T_1 \) = uncracked base length

\( L \) = total base length.

d. Determine the toe pressure (\( B_5 \)) using the following expression:

\[ B_5 = \frac{2(\Sigma W - U) + A_2}{T_1} \]

where \( A_2 \) = tailwater pressure.

e. Only the uncracked portion of the base, \( T_1 \), shall be used to determine the sliding stability.

f. Load Cases I, II and IIA shall be reevaluated for the post-earthquake condition using the uncracked length determined in step C above. Sliding factors should be recalculated using only friction on the cracked portion of the base, and friction plus cohesion on the uncracked portion.

### 3-4.6.2 For Existing Gravity Dams.

References 1 and 7 provide detailed analysis procedures for cracked base analysis for load cases I, II and IIA. Paragraph 3-4.6.1 should be used for analysis of load cases III and IIIA.

### 3-4.7 Computer Programs - Review of Computer Studies

Computer studies submitted by applicants or licensees should include the name of the computer program used, and sample input data and output data. Program documentation should be requested for any program which has not been thoroughly tested and proven to be accurate.

Documentation should include samples of verification runs and a description of the analysis procedures used in the program. Well known programs such as: SAP4, STRUDL, ANSYS, etc., may be used without supporting documentation.

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Input data should be checked in order to determine if the computer model will accurately predict the structural behavior of the prototype and the loads to which it will be subjected.

Output data should be spot checked and compared to hand calculated solutions wherever possible, in order to assure that the basic laws of statics have been satisfied, i.e., summation of forces equals zero and summation of moments equals zero.

3-5 Stability Criteria

3-5.1 General

Specific stability criteria for a particular loading combination is dependent upon the type of analysis being done (i.e., foundation or concrete analysis), the degree of understanding of the foundation structure interaction and site geology, and to some extent, on the method of analysis. For unconstructed projects, preliminary analyses are generally based on more conservative criteria than final designs. As the design process progresses the designer has available more sophisticated and detailed foundation information and material testing results. Therefore, the unknowns associated with the preliminary designs are reduced by the final design stage and somewhat lower safety factors may be acceptable.

For constructed projects, assumptions used in the analysis should be based upon construction records and the performance of the structures under historical flood loadings. In the absence of available design data and records, site investigations must be conducted to verify all assumptions.

3-5.2 Static Methods of Analysis

3-5.2.1 Basic Requirements

The basic requirements for stability of a gravity dam are:

a. That it be safe against overturning at any horizontal plane within the dam, at the base or at any plane below the base. This requires that the allowable unit stresses
established for the concrete and foundation materials not be exceeded. The allowable stresses should be determined by dividing the ultimate strengths of the materials by the appropriate safety factors in Table 2.

b. That it be safe against sliding on any horizontal plane within the dam, on the foundation or on any horizontal seam in the foundation. Reference 5, should be used to evaluate sliding stability.

3-5.2.2 Tensile Stresses

Tensile stresses within the body of the dam should not exceed 10 percent of the compressive strength of the concrete. The tensile strength of horizontal lift joints within the dam may be less, and testing may be required to establish allowable stresses.

The tensile strength of the rock-concrete interface should be assumed to be zero. The rock foundations may consist of adversely-oriented joints or fractures such that even if the interface could resist tension, the rock formation immediately below may not be able to develop the same strength. Therefore, since stability would not be enhanced by an interface with tensile strength when a joint, seam or fracture in the rock only a few inches or feet below the interface has zero tensile strength, no tension will be allowed at the interface.

3-5.2.3 Cracked Base Analysis

Theoretical base cracking will be allowed for all loading conditions, provided that the crack stabilizes within the base of the dam, and adequate sliding safety factors are obtained using only the uncracked portion of the base. If the base cracking occurs during loading conditions III or IIIA, Section 3-5.2.4 shall apply.
TABLE 2

Recommended Factors of Safety 1/

Dams having a high or significant hazard potential.

<table>
<thead>
<tr>
<th>Loading Condition 2/</th>
<th>Factor of Safety 3/</th>
</tr>
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<tbody>
<tr>
<td>Usual</td>
<td>3.0</td>
</tr>
<tr>
<td>Unusual</td>
<td>2.0</td>
</tr>
<tr>
<td>Extreme</td>
<td>Greater than 1.0</td>
</tr>
</tbody>
</table>

Dams having a low hazard potential.

<table>
<thead>
<tr>
<th>Loading Condition 2/</th>
<th>Factor of Safety 3/</th>
</tr>
</thead>
<tbody>
<tr>
<td>Usual</td>
<td>2.0</td>
</tr>
<tr>
<td>Unusual</td>
<td>1.25</td>
</tr>
<tr>
<td>Extreme</td>
<td>Greater than 1.0</td>
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</table>

1/ Safety factors apply to the calculation of stress and the shear Friction Factor of Safety within the structure, at the rock/concrete interface and in the foundation. Deviations from recommended safety factors are discussed in paragraph 3-5.5.

2/ Loading conditions as defined in paragraph 3-3.0.

3/ Safety factors are based upon the use of the gravity method of analysis. Safety factors should not be calculated for overturning, i.e., Mr/Mo.

For new designs, base cracking will not be allowed for loading combinations I, II and IIA. Cracking will be allowed for Case III and IIA loadings as described in Section 3-5.2.4.

3-5.2.4 Seismic Loading

When the extreme loading combination consists of an earthquake loading using the seismic coefficient method, the basic requirements for stability under Case I or Case II loading shall apply except that a cracked base analysis will be allowed if the structure stabilizes when the analysis is conducted in accordance with Section 3-4.6 of this chapter. An analysis must be conducted of the post-

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earthquake condition in accordance with Section 3-5.4 using the cracked base and modified material parameters to ensure stability under Cases I, II and IIA.

3-5.2.5 Finite Element Analysis

When an FEM analysis is conducted using equivalent static loads, the basic criteria of this section shall apply. Conventional factors of safety for sliding and overturning should be determined by integrating the stress distributions at the structure foundation interface to calculate resultant locations.

3-5.3 Dynamic Methods of Analysis

3-5.3.1 Basic Requirements

When the earthquake loading is calculated using dynamic or pseudo dynamic methods, the following criteria shall apply:

a. The dam shall be capable of surviving a Maximum Credible Earthquake (MCE) without a failure of a type that would result in loss of life or significant damage to downstream property. Inelastic behavior with associated damage is permissible under the MCE for the site. The MCE is defined as the severest earthquake that is believed to be possible at the site on the basis of geological and seismological evidence. The MCE should be determined by regional and local studies which include a complete review of all historic earthquake data of events located sufficiently nearby to influence the project, all faults in the area, and attenuations between causative faults and the site.

b. The dam shall be capable of resisting an Operating Basis Earthquake (OBE) within the elastic range of the materials. The OBE is

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generally more moderate than the MCE and is selected on a probabilistic basis from regional and local geology and seismology studies as being likely to occur during the life of the project. It is generally as large as earthquakes that have occurred in the seismotectonic province in which the site is located. See References 4 and 34 for detailed information concerning the above criteria.

3-5.3.2 Tensile Stresses

Since tensile cracking is acceptable for the MCE loading, the significance of the tensile stresses which exceed the tensile strength of the concrete shall be evaluated as outlined in Section 3-5.4.

3-5.3.3 Evaluation of Dynamic Analysis Results

a. Sliding Stability Analysis. To evaluate the significance of cracking, perform a sliding stability analysis for the portion of the dam above the plane where the greatest cracking is expected.

b. If the sliding safety factor is equal to, or greater than 1.0, adequate sliding resistance can be assumed. If the sliding safety factor is less than 1.0, compute an upper bound for the permanent displacement along the plane of investigation by assuming a horizontal crack the entire length of the sliding plane, and perform a partial nonlinear finite element analysis to compute the permanent displacement.

c. Partial Nonlinear Analysis. For an upperbound solution, perform a linear finite element acceleration time history method of analysis with a special slip element to model the discontinuity along the crack. The
slip element should have normal and tangential strength but no tensile strength.

The element should limit the magnitude of the shear stresses to the value allowed by the frictional force if the element is in compression. More than one cracked plane can be modeled at one time. Some general purpose finite element computer programs which include the slip element are SAP 80, ANSYS, ADINA, and ABAQUS.

d. **Overturning Stability.** During the earthquake-induced excitation, the overturning stability for a gravity dam can be assumed to be adequate if the overturning criteria for the static loading conditions are met. This mode of failure of a gravity dam due to an earthquake-induced excitation is not likely because of the large base width to height ratios of gravity dam cross-sections and the nature of the excitation peak oscillatory motion of very short duration.

3-5.4 **Post-Earthquake Stability Analysis**

3-5.4.1 **Stability After the MCE**

The dam, in its post-earthquake condition, should be capable of containing the reservoir for a sufficient period of time to allow for strengthening of the dam, if necessary. For the normal static loading condition, the overturning and sliding stability should be checked assuming appropriate uplift in any cracked portion of the base as determined by the dynamic analysis. The resultant should be sufficiently within the base so that 50 percent of the unconfined compressive strength of the concrete is not exceeded and the sliding safety factor is 1.3 or greater.
3-5.4.2 **Analysis for Operating Basis Earthquake (OBE)**

If no cracking is estimated to occur for the MCE loading, a stress analysis for the OBE is not required. However, effects of the OBE on the continued operation of essential equipment should be determined. If the level of the OBE is sufficiently high to require a dynamic analysis, the simplest linear elastic analysis appropriate for the monolith should be performed for the OBE loading using a five percent damping ratio. The combined maximum static and dynamic tensile stress should be no greater than 15 percent of the static unconfined compressive strength of the concrete to assume no cracking occurs, provided the lift joints are sound.

3-5.5 **Safety Factor Evaluation**

The safety factors determined in previous sections shall be evaluated on a case-by-case basis in order to assess the overall safety of a particular project. Engineering judgment must be used to evaluate any calculated safety factor which does not conform to the recommendations of Table 2. In applying engineering judgment, consideration must be given to both the adequacy of the data presented in support of the analyses and the loading case for which the safety factor does not meet the criteria.

For example, it is acceptable to apply the factor of safety criteria for the unusual loading combination from Table 2 to the usual loading combination if the normal headwater and tailwater elevations result in the maximum net forces tending to indicate overturning of the structure.

In addition, it will also be acceptable to allow a factor of safety as low as 1.5 for the unusual loading combination on a case-by-case basis when the PMF is the Inflow Design Flood (IDF). Generally, a factor of safety of less than 2.0 should not be accepted for the unusual loading condition if the IDF is less than the PMF, or if the unusual condition criteria is used for the normal loading condition as outlined above. Before approving a safety factor of 1.5 when the PMF is the IDF, the factor of safety at lesser
flood magnitudes should be reviewed to ensure that the safety factor at 50 percent of the PMF is 2.0.

It is preferable to better define design parameters and loading conditions than to utilize higher safety factors to accommodate uncertainties in the analysis. Therefore, in order to be flexible in the evaluation of safety factors, Staff should require the licensee/exempee to better define foundation and material strength parameters, and loading conditions. Any decision to accept safety factors lower than those shown in Table 2 will be based on: (1) the degree of uncertainty in the data and analyses provided and (2) the nature of the loading condition, i.e. an infrequent versus frequent or sustained loading.

In accepting any lower safety factor as outlined in the examples described above or other situations, the stability analyses must be supported by a program that includes, but is not limited to, adequate field level investigations to define material (dam and foundation) strength parameters, installation and verification of necessary instrumentation to evaluate uplift assumptions and loading conditions, a detailed survey of the condition of the structure, and proper analysis procedures. This program should be submitted for approval by the Director, Division of Dam Safety and Inspections. Flexibility on safety factors beyond that discussed above will be infrequent and on special case specific consideration.

3-5.6 Foundation Stability

3-5.6.1 Rock Foundations

The foundation or portions of it must be analyzed for stability whenever the structural configuration of the rock is such that direct shear failure is possible, or whenever sliding failure is possible along faults, shears and/or joints. Associated with stability are problems of local overstressing in the dam due to foundation deficiencies. The presence of such weak zones can cause problems under either of two conditions: (1) when differential displacement of rock blocks occurs on either side of weak zones, and (2) when the width of a weak zone represents an excessive span for
the dam to bridge over. To prevent local overstressing, the zones of weakness in the foundation must be strengthened so that the applied forces can be distributed without causing excessive differential displacements, and so that the dam is not overstressed due to bridging over the zone. 23/

Sliding failure may result when the rock foundation contains nearly horizontal seams. Such seams are particularly dangerous when they contain clay, bentonite, or other similar substances. Rock that is otherwise satisfactory may have to be removed in order to eliminate an objectionable seam below it. 24/

3-5.6.2 Soil Foundations

Gravity dams constructed on soil foundations are usually relatively small structures which exert low bearing pressures upon the foundation. Large structures on soil foundations are usually supported by bearing or friction piles and are beyond the scope of these guidelines. When the foundation consists of pervious sands and gravels, such as alluvial deposits, two possible problems exist; one pertains to the amount of underseepage, and the other is concerned with the forces exerted by the seepage. Loss of water through underseepage may be of economic concern for a storage or hydroelectric dam but may not adversely affect the safety of the dam. However, adequate measures must be taken to ensure the safety of the dam against failure due to piping, regardless of the economic value of the seepage. 25/

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23/ Reference 1, Page 76.
24/ Reference 3, Page 8.
a. Compressive tests; ASTM C39

b. Tensile tests; ASTM C78

c. Shear tests; RTH 203-80 31/


e. Poisson's ratio, ASTM C469

f. Collection of test samples: ASTM C31, C172, and C192

g. Evaluation of tests results: ACI 214

Additional guidance concerning the design of mass concrete mixes and the determination of the cured properties of the concrete are presented in reference 17.

3-6.3.3 Durability

The durability of concrete is influenced by the physical nature of the component parts, and although performance is largely influenced by mix proportions and degree of compaction, the aggregates constitute nearly 85 percent of the constituents in a mass concrete and good aggregates are essential for durable concrete. 32/ The environment in which the structure will exist must be considered in the mix design and in the evaluation of the suitability of aggregate sources proposed for use in the mix. Generally, the environmental considerations which must be examined are: weathering due to freezing and thawing cycles; chemical attack from reactions between the elements in the concrete, exposure to acid waters, exposure to sulfates in water and leaching by mineral-free water; and

31/ Reference 33: applies only to the testing of rock core to concrete bond specimens.

32/ Reference 27; Page 173.

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erosion due to cavitation or the movement of abrasive material in flowing water. Mix designs and material considerations, which will insure a durable concrete are presented in references 1 and 17.

3-6.3.4 Dynamic Properties

The apparent compressive and tensile strengths of concrete varies with the speed of testing. As the rate of loading increases, compressive and tensile strengths also increase. Therefore, the strength properties of concrete under dynamic loadings, such as during an earthquake, are higher than under static conditions.

References 34 and 37 provide a detailed discussion of the rates and types of testing which should be conducted to determine the dynamic properties of concrete for use in linear finite element analyses. The concrete properties to be used in the dynamic analysis for new designs should be based on rapid rate testing conducted on test mixes which approximate the properties of the concrete to be used in the actual construction. The rates of testing should be coordinated with the expected stress cycles of the design seismic event.

Existing structures which require a dynamic analysis, and which have concrete coring and testing information available should use the recommendations of reference 37 to determine the dynamic tensile strength. Since dynamic compressive stresses rarely are of concern, the allowable compressive strength for static loading can be utilized in the dynamic analysis. If, however, coring is to be conducted to determine concrete properties, then rapid rate tests should be run to establish both the dynamic compressive and tensile strength.

33/ Reference 1, Pages 281 thru 285; Reference 17, Paragraph 3.9.

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3-6.4 **Foundation Properties**

In most instances, a gravity dam is keyed into the foundation so that the foundation will normally be adequate if it has enough bearing capacity to resist the loads from the dam. 34/ If, however, weak planes or zones of inferior rock are present within the foundation, the stability of the dam will be governed by the sliding resistance of the foundation. The foundation investigations should establish the following strength parameters:

- a. Shear and sliding strengths (φ and C) of the discontinuities and the rock.
- b. Bearing capacity (compressive strength)
- c. Elastic Modulus
- d. Poisson's ratio

These parameters are established by laboratory tests on samples obtained at the site. In some instances, in situ testing may be justified. In either instance, it is important that samples and testing methods be representative of the site conditions. The results of these tests will, generally, yield ultimate strength or peak values and must, therefore, be divided by the appropriate factors of safety in order to obtain the allowable working stresses. Recommended factors of safety are presented in Table 2.

Foundation permeability test should be conducted in conjunction with the drilling program, or as a separate study, in order to establish uplift parameters and to design an appropriate drainage system. Permeability testing programs should be designed to establish the permeability of the rock mass and not an isolated sample of the rock material. The mass permeability will usually be higher, due to jointing and faulting, than an individual sample. Guidance for staff review of permeability tests can be found in reference 26. 35/

34/ Reference 1, Page 15.
35/ Reference 26, Designation E-1E, Page 593

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Prior to the selection of allowable foundation strengths, for existing dams, all available geologic and foundation information should be reviewed to gain an understanding of the type of material and structural formation on which the dam is constructed. A general description of the foundation material can be used as a basis for choosing a range of allowable strengths from published data, if testing data is not available. Staff geologists should be consulted if the available information refers to material parameters or structural features which are suspected to be indications of poor foundation conditions. Some terms which should alert the engineer to possible problem areas are listed below:

a. Low RQD ratio (RQD = Rock Quality Designation).

b. Solution features such as caves, sinkholes and fissures.

c. Columnar jointing.

d. Closely spaced horizontal seams or bedding planes.

e. Highly weathered or fractured material.

f. Shear zones or faults and adversely oriented joints.

g. Joints or bedding planes described as slicken sided, or filled with gouge materials such as bentonite or other swelling clays.

h. Foliation surfaces.

Compressive - In general, the compressive strength of a rock foundation will be greater than the compressive strength of the concrete within the dam. Therefore, crushing (or compressive failure) of the concrete will usually occur prior to


37. Raphael, Jerome M., "Tensile Strength of Concrete", Journal of the American Concrete Institute, Mar/Apr 1984, p158.

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