United States Department of Agriculture

Soil Conservation Service

P.O. Box 2890
Washington, D.C.
20013

January 30, 1991

TECHNICAL RELEASE 60, REVISED OCTOBER 1985
210-VI
AMENDMENT 1

SUBJECT: ENG - EARTH DAMS AND RESERVOIRS

Purpose. To eliminate the parameter H and redefine the parameters L, T, and H_w in the peak breach discharge criteria; to define the hydrologic boundaries for areas 1, 2, and 3 on page 2-10; and to update the requirements for the design of diaphragms used in piping and seepage control.

Effective Date. Effective when received.

Explanation of Changes. The equations for the theoretical breach width, T, and the related peak discharge, Q_{max}, in category 3, page 1-2 of TR-60 are applicable only when the water depth, H_w, is equal to the height of the dam. The enclosed replacement page contains the following revisions, which are applicable to a full range of H_w values:

1. The definition for the parameters L, T, and H_w are changed, and

2. The parameter H is eliminated and is replaced by the parameter H_w in two equations.

The 100-year, 10-day runoff map on page 2-10 should be divided into three areas, i.e., areas 1, 2, and 3; but the hydrologic boundaries and the area numbers were not shown. The revised page shows distinct boundary lines and area numbers to delineate and identify the areas.

The design of the drainage diaphragm used in piping and seepage control needs to meet only the requirements of Soil Mechanic Note No. 1; thus, the exception is eliminated.

Filing Instructions. Remove pages 1-1, 1-2, 2-9, 2-10, 6-7 and 6-8 of TR-60 (revised October 1985) and replace with the enclosed pages.

DIST: TR-60

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Distribution. Make this Amendment available to all offices having a copy of TR-60. Additional copies may be obtained from the Consolidated Forms and Publication Distribution Center by ordering TR-60A.

ROBERT R. SHAW
Deputy Chief for Technology

Enclosures

General

Dam Classification

In determining dam classification, a number of factors are to be considered. Consideration is to be given to the damage that might occur to existing and future developments should the dam suddenly release large quantities of water downstream due to a breach, failure or landslide into the reservoir. The effect of failure on public confidence is an important factor. State and local regulations and the responsibility of the involved public agencies are to be recognized. The stability of the spillway materials, the physical characteristics of the site and the valley downstream, and the relationship of the site to industrial and residential areas including controls of future development all have a bearing on the amount of potential damage in the event of a failure.

Dam classification is determined by the above conditions. It is not determined by the criteria selected for design. The policy on classification is in 210-V, Part 520, Subpart C DAMS (National Engineering Manual).

Classes of Dams

The following classes of dams have been established by policy and repeated here for convenience of the user.

Class (a). --Dams located in rural or agricultural areas where failure may damage farm buildings, agricultural land, or township and country roads.

Class (b). --Dams located in predominantly rural or agricultural areas where failure may damage isolated homes, main highways or minor railroads or cause interruption of use or service of relatively important public utilities.

Class (c). --Dams located where failure may cause loss of life, serious damage to homes, industrial and commercial buildings, important public utilities, main highways, or railroads.

Peak Breach Discharge Criteria

Breach routings are used to help delineate the area potentially impacted by inundation should a dam fail and can be used to aid dam classification.

Stream routings made of the breach hydrograph are to be based upon topographic data and hydraulic methodologies mutually consistent in their accuracy and commensurate with the risk being evaluated.

The minimum peak discharge of the breach hydrograph 1/ regardless of the technique used to analyze the downstream inundation area, is as follows:

1/ The breach hydrograph is the outflow hydrograph attributed to the sudden release of water in reservoir storage due to a dam breach.

1. For depth of water at the dam at the time of failure $\geq$ 103 feet.

$$Q_{max} = 65\ H_w^{1.85}$$

2. For depth of water at the dam at the time of failure < 103 feet.

$$Q_{max} = 1100\ B_r^{1.35}, \qquad \text{where } B_r = \frac{V_s\ H_w}{A}$$

but is not to be less than

$$Q_{max} = 3.2\ H_w^{2.5}$$

and need not exceed

$$Q_{max} = 65\ H_w^{1.85}$$

3. When the width of the valley, L, at the water surface elevation corresponding to the depth, H_w, is less than

$$T = \frac{65\ H_w^{0.35}}{0.416},$$

replace the equation

$$Q_{max} = 65\ H_w^{1.85}$$

in categories 1 and 2 above with the equation

$$Q_{max} = 0.416\ L\ H_w^{1.5}$$

Where,

Q_{max} = the peak breach discharge, cfs.

B_r = breach factor, acre.

V_s = reservoir storage at the time of failure, acre-feet.

H_w = depth of water at the dam at the time of failure; however, if the dam is overtopped, depth is set equal to the height of dam, feet.

A = cross-sectional area of embankment at the assumed location of breach, usually the template section (normal to the dam longitudinal axis) at the general flood plain location, sq. ft.

T = theoretical breach width at the water surface elevation corresponding to the depth, H_w, for the equation $Q_{max} = 65\ H_w^{1.85}$, feet.

L = width of the valley at the water surface elevation corresponding to the depth, H_w, feet.

TABLE 2-5

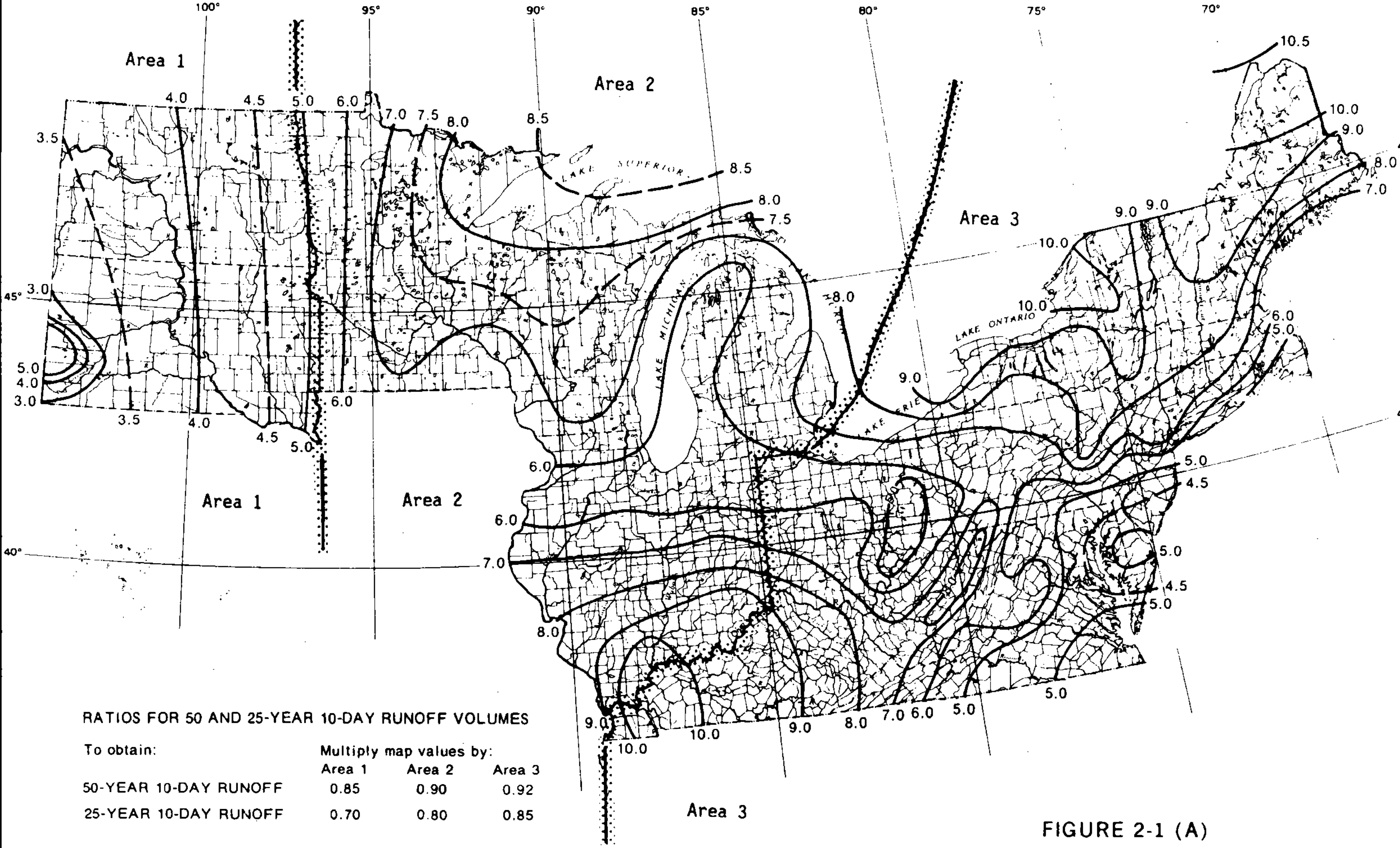
MINIMUM EMERGENCY SPILLWAY HYDROLOGIC CRITERIA

Class of Dam	Product of Storage x Effective Height	Existing or Planned Upstream Dams	Precipitation Data for 1/	
			Emergency Spillway Hydrograph	Freeboard Hydrograph
(a) 2/	less than 30,000	none	P_{100}	$P_{100} + 0.12\ (PMP - P_{100})$
	greater than 30,000	none	$P_{100} + 0.06\ (PMP - P_{100})$	$P_{100} + 0.26\ (PMP - P_{100})$
	all	any 3/	$P_{100} + 0.12\ (PMP - P_{100})$	$P_{100} + 0.40\ (PMP - P_{100})$
(b)	all	none or any	$P_{100} + 0.12\ (PMP - P_{100})$	$P_{100} + 0.40\ (PMP - P_{100})$
(c)	all	none or any	$P_{100} + 0.26\ (PMP - P_{100})$	PMP

1/ P_{100} = Precipitation for 100-year return period. PMP = Probable maximum precipitation.

2/ Dams involving industrial or municipal water are to use minimum criteria equivalent to that of class (b).

3/ Applies when the upstream dam is located so that its failure could endanger the lower dam.



RATIOS FOR 50 AND 25-YEAR 10-DAY RUNOFF VOLUMES

To obtain:	Multiply map values by: Area 1	Area 2	Area 3
50-YEAR 10-DAY RUNOFF	0.85	0.90	0.92
25-YEAR 10-DAY RUNOFF	0.70	0.80	0.85

FIGURE 2-1 (A)
100-YEAR, 10-DAY RUNOFF (INCHES)
PRINCIPAL SPILLWAY HYDROGRAPH

JANUARY 1971, REV. APRIL 1976, OCTOBER 1990

Only joints incorporating a round rubber gasket set in a positive groove which will prevent its displacement from either internal or external pressure under the required joint extensibility are to be used on precast concrete pipe conduits. Concrete pipe must have steel joint rings providing rubber to steel contact in the joint.

Articulation of the conduit (freedom for required rotation) is to be provided at each joint in the conduit, at the junction of the conduit with the riser and any outlet structure. Concrete bedding for pipe conduits need not be articulated. Cradles are to be articulated if on yielding foundations. Welded steel pipe conduits need not be articulated if the pipe and bedding rest directly on firm bedrock.

Piping and Seepage Control - Use a filter and drainage diaphragm around any structure that extends through the embankment to the downstream slope. Design the diaphragm with single or multizones to meet the requirements of Soil Mechanics Note No. 1.

Locate the diaphragm aligned approximately parallel to the centerline of the dam or approximately perpendicular to the direction of seepage flow. Extend the diaphragm horizontally and vertically into the adjacent embankment and foundation to intercept potential cracks, poorly compacted soil zones or other discontinuities associated with the structure or its installation.

Design the diaphragms to extend the following minimum distances from the surface of rigid conduits:

1. Horizontally and vertically upward 3 times the outside diameter of circular conduits or the vertical dimension of rectangular box conduits except that:

   a. the vertical extension need be no higher than the maximum potential reservoir water level, and

   b. the horizontal extension need be no further than 5 feet beyond the sides and slopes of any excavation made to install the conduit.

2. Vertically downward:

   a. for conduit settlement ratios (δ) of 0.7 and greater (reference SCS Technical Release No. 5), the greater of (1) 2 feet or (2) 1 foot beyond the bottom of the trench excavation made to install the conduit. Terminate the diaphragm at the surface of bedrock when it occurs within this distance. Additional control of general seepage through an upper zone of weathered bedrock may be needed.

   b. 1.5 times the outside diameter of circular conduits or the outside vertical dimension of box conduits for conduit settlement ratios (δ) less than 0.7.

Design the diaphragms to extend in all directions a minimum of 2 times the outside diameter from the surface of flexible conduits, except that the diaphragm need not extend beyond the limits in 1a and 1b above nor beyond a bedrock surface beneath the conduit.

Provide minimum diaphragm thickness of 3 feet and minimum thickness of 1 foot for any zone of a multizone system. Use larger thickness when needed for (1) capacity, (2) tieing into embankment or foundation drainage systems, (3) accommodating construction methods, or (4) other reasons.

For homogeneous dams, locate the diaphragm in the downstream section of the dam such that it is:

1. Downstream of the cutoff trench,

2. Downstream of the centerline of the dam when no cutoff trench is used, and

3. Upstream of a point where the embankment cover (upstream face of the diaphragm to the downstream face of the dam) is at least one-half of the difference in elevation between the top of the diaphragm and the maximum potential reservoir water level.

For zoned embankments, locate the diaphragm downstream of the core zone and/or cutoff trench, maintaining the minimum cover as indicated for homogeneous dams. When the downstream shell is more pervious than the diaphragm material, locate the diaphragm at the downstream face of the core zone.

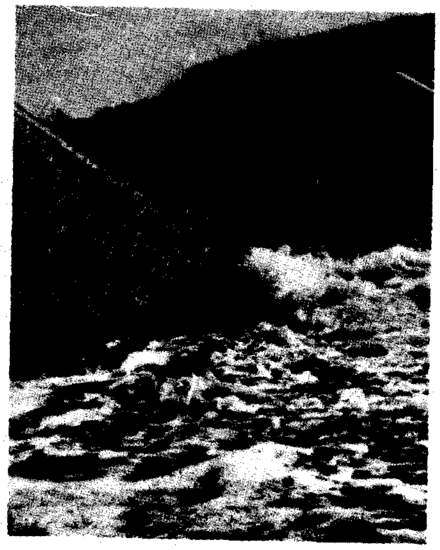
It is good practice to tie these diaphragms into the other drainage systems in the embankment or foundation. Foundation trench drains and/or embankment chimney drains that meet the minimum size and location limits are sufficient and no separate diaphragm is needed.

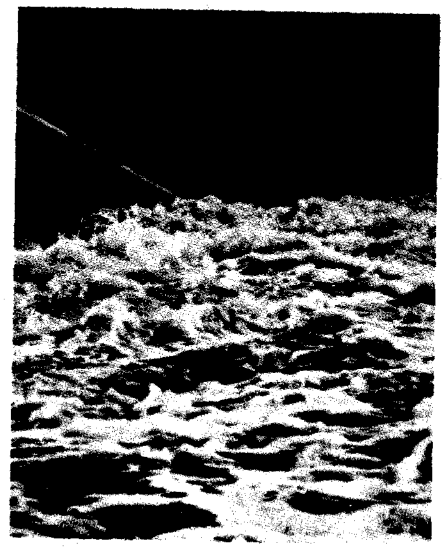
Design the minimum capacity of outlets for diaphragms not connected to other drains by assuming the coefficient of permeability (k) in the zone upstream of the diaphragm is 100 times the coefficient of permeability in the compacted embankment material. Assume this zone has a cross-sectional area equal to the diaphragm area and the seepage path distance equal to that from the embankment upstream toe to the diaphragm. This higher permeability simulates a sealed filter face at the diaphragm with partially filled cracks and openings in the upstream zone.

For channels, chutes or other open structures, seepage and piping control can be accomplished in conjunction with drainage for reduction of uplift and water loads. The drain, properly designed to filter the base soils, is to intercept areas of potential cracking caused by shrinkage, differential settlement or heave and frost action. These structures

TR60

Earth Dams and Reservoirs





Technical Release No. 60
210 - VI

Revised Oct. 1985

U.S. Department of Agriculture
Soil Conservation Service
Engineering Division

TECHNICAL RELEASE

NUMBER 60

CONTENTS

PREFACE

This Technical Release (TR) describes design procedures and provides minimum requirements for planning and designing earth dams and associated spillways. This TR was developed to provide uniform criteria for earth dams and reservoirs. SCS plans, designs and constructs complex dams under widely varying conditions. It is essential that these dams be constructed with uniform criteria to assure consistent performance. As new experience, materials, and knowledge become available, this document will be revised.

This TR applies to all Class (a) dams with a product of storage times the effective height of the dam of 3,000 or more, those more than 35 feet in effective height and all Class (b) and (c) dams. Requirements are stated as maximum or minimum limits and may not be satisfactory design criteria for all sites. In some cases problems may arise where proven solutions are not available or alternative procedures may need to be evaluated before the best solutions can be developed and selected. Experience, state laws and regulations, investigations, analysis, expected maintenance, environmental considerations or safety laws may dictate more conservative criteria to insure satisfactory performance.

This edition of the TR incorporates those revisions previously issued by changes on February 10, 1982, June 2, 1982, and April 6, 1984. Also included are additions for peak breach discharge criteria, streamflow diversion analysis, conservation storage analysis, updates on hydrometeorological report references and revision in embankment slope stability criteria.

DEFINITIONS

Base flow is the sustained or fair-weather discharge which persists after storm runoff and associated quick return flow have been depleted. It is usually derived from groundwater discharge or gradual snow or ice melt over extended periods of time, but need not be continuous flow. (It can be based on annual or seasonal periods depending upon when major floods usually occur.)

Conservation storage is water impounded for consumptive uses such as municipal, industrial and irrigation and non-consumptive uses such as recreation and fish and wildlife.

The control section in an open channel spillway is that section where accelerated flow passes through critical depth.

A dam is an artificial barrier, together with any associated spillways and appurtenant works, across a watercourse or natural drainage area, which does or may impound or divert water.

Design life is a period of time during which a dam is designed to perform its assigned functions satisfactorily.

A dry dam is a dam that has an ungated outlet positioned so that essentially all stored water will be drained from the reservoir by gravity. The reservoir will normally be dry.

An earth dam is a dam in which the principal barrier is an embankment of earth or rock fill or combination of earth and rock fill.

An earth spillway is an open channel spillway in earth materials without vegetation.

Economic life is the period of time during which economic benefits accrue to a dam.

Effective height of dam is the difference in elevation in feet between the lowest open channel emergency spillway crest and the lowest point in the original cross section on the centerline of the dam. If there is no open channel emergency spillway, the top of the dam becomes the upper limit.

The emergency spillway is the spillway designed to convey excess water through, over or around a dam.

The emergency spillway hydrograph is the hydrograph used to establish the dimensions of the emergency spillway.

An emergency spillway system is a single emergency spillway or combination of emergency spillways designed to work together.

The exit channel of an open channel spillway is the portion downstream from the control section which conducts the flow to a point where it may be released without jeopardizing the dam.

The freeboard hydrograph is the hydrograph used to establish the minimum settled elevation of the top of the dam. It is also used to evaluate the structural integrity of the spillway system.

The inlet channel of an open channel spillway is the portion upstream from the control section.

Joint extensibility is the amount a pipe joint can be extended from the fully engaged position without losing strength or watertightness. In case of rubbergasket joints it is measured from the center of the gasket to the point of flare of the bell ring or collar when the joint is fully closed.

Joint gap is the longitudinal dimension between the end face of the spigot end of a pipe joint and the corresponding face of the bell end of the connecting pipe. It does not include the beveled portion designed for sealing compounds.

The joint use pool is the portion of a reservoir which serves two or more purposes; for instance, conservation storage and floodwater storage.

A primary emergency spillway is the spillway with the lowest crest elevation in an emergency spillway system.

The principal spillway is the lowest ungated spillway designed to convey water from the reservoir at predetermined release rates.

The principal spillway hydrograph is the hydrograph used to determine the minimum crest elevation of the emergency spillway. It is used to establish the principal spillway capacity and determine the associated minimum floodwater retarding storage.

Quick return flow is the diminishing discharge directly associated with a specific storm that occurs after surface runoff has reached its maximum. It includes base flow, prompt subsurface discharge (commonly called interflow) and delayed surface runoff.

A ramp spillway is a vegetated spillway constructed over an earth dam in a manner such that the spillway is a part of the embankment.

The retarding pool is the portion of the reservoir allotted to the temporary impoundment of floodwater. Its upper limit is the elevation of the crest of the emergency spillway.

Retarding storage is the volume in the retarding pool.

A rock spillway is an open channel spillway through competent, non-erodible, natural rock materials.

The sediment pool is the portion of the reservoir allotted to the accumulation of submerged sediment during the design life of the dam.

The sediment pool elevation is the elevation of the surface of the anticipated submerged sediment accumulation at the dam.

Sediment storage is the reservoir capacity allocated to total sediment (submerged and aerated) accumulation during the life of the dam.

A spillway is an open or closed channel, conduit or drop structure used to convey water from a reservoir. It may contain gates, either manually or automatically controlled, to regulate the discharge of water.

Storage is the capacity of the reservoir below the elevation of the crest of the emergency spillway.

A vegetated spillway is a vegetated open channel spillway in earth materials.

Visual Focal is an element in the landscape upon which the eyes automatically focus because the element's size, form, color, or texture contrast clearly with its surroundings.

General

Dam Classification

In determining dam classification, a number of factors are to be considered. Consideration is to be given to the damage that might occur to existing and future developments should the dam suddenly release large quantities of water downstream due to a breach, failure or landslide into the reservoir. The effect of failure on public confidence is an important factor. State and local regulations and the responsibility of the involved public agencies are to be recognized. The stability of the spillway materials, the physical characteristics of the site and the valley downstream, and the relationship of the site to industrial and residential areas including controls of future development all have a bearing on the amount of potential damage in the event of a failure.

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Class (a). --Dams located in rural or agricultural areas where failure may damage farm buildings, agricultural land, or township and country roads.

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Class (c). --Dams located where failure may cause loss of life, serious damage to homes, industrial and commercial buildings, important public utilities, main highways, or railroads.

Peak Breach Discharge Criteria

Breach routings are used to help delineate the area potentially impacted by inundation should a dam fail and can be used to aid dam classification.

Stream routings made of the breach hydrograph are to be based upon topographic data and hydraulic methodologies mutually consistent in their accuracy and commensurate with the risk being evaluated.

The minimum peak discharge of the breach hydrograph 1/ regardless of the technique used to analyze the downstream inundation area, is as follows:

1/ The breach hydrograph is the outflow hydrograph attributed to the sudden release of water in reservoir storage due to a dam breach.

1. For depth of water at the dam at the time of failure $\geq$ 103 feet.

$$Q_{max} = 65\ H_w^{\ 1.85}$$

2. For depth of water at the dam at the time of failure < 103 feet.

$$Q_{max} = 1100 B_r^{\ 1.35}, \text{ where } B_r = \frac{V_s\ H_w}{A}$$

but is not to be less than

$$Q_{max} = 3.2\ H_w^{\ 2.5}$$

and need not exceed

$$Q_{max} = 65\ H_w^{\ 1.85}$$

3. When the dam crest length, L, due to limited valley width, is less than

$$T = \frac{65\ H^{0.35}}{0.416}$$

then determine Q_{max} by following

$$Q_{max} = 0.416\ L\ H^{\ 1.5}$$

in lieu of

$$Q_{max} = 65\ H_w^{\ 1.85}$$

in each of catagories 1 and 2 above.

Q_{max} = the peak breach discharge, cfs.

B_r = breach factor, acre.

V_s = reservoir storage at the time of failure, acre-feet.

H_w = depth of water at the dam at the time of failure, feet.

A = cross-sectional area of embankment at the location of breach, usually the template section (normal to the dam longitudinal axis) at the general flood plain location, square feet.

H = height of the dam at its centerline, height from the bottom of the breach to the dam crest, feet.

T = theoretical breach width for the enveloping upper Q_{max} limit, feet.

L = crest length of dam between abutments, feet.

The peak discharge value determined by using principles of erosion, hydraulics, and sediment transport may be used in lieu of the peak discharge computed using the above equations.

Utility Cables and Pipelines

Existing pipelines, cables, and conduits of a wide variety of sizes, materials, and functions are frequently encountered at dam sites. These conduits are usually located at shallow depths in the flood plain. They constitute a hazard to the safety of the dam and are to be (1) relocated away from the site or (2) reconstructed or modified to provide the durability, strength, and flexibility equal in all aspects to the principal spillway designed for the site in accordance with Service criteria and procedures. Overhead cables or power lines are to be relocated or raised as necessary to prevent damage or hazard to the public.

Every reasonable effort is to be made to have such conduits, cables, and pipelines removed from the site. Most utilities and industries will want their facility removed from the site for easy maintenance. Only as a last resort and under the limitations imposed below are conduits to be permitted to remain under an earth dam embankment.

Conduits permitted to remain under any part of the embankment below the crest of the emergency spillway are to be (1) provided with seepage control against potential piping, (2) properly articulated on all yielding foundations, (3) encased in concrete or otherwise treated to insure durability and strength equal to that of the principal spillway, and (4) made watertight against leaking either into or out of the pipe.

Enclosure of the conduit, cable, or pipeline within another conduit which meets the requirements of this section and which is positively sealed at the upstream end to prevent seepage into the enclosing conduit is acceptable. Such an enclosing conduit is to extend the full distance through which the conduit, cable or pipeline being enclosed is beneath the embankment.

Cut Slope Stability

Natural and excavation cut slopes are to be planned and formed in a stable and safe manner. Spillways, inlet and outlet channels, borrow pits, reservoir edges, abutment areas and foundation excavations are all locations where these considerations are needed. Field investigations, methods of analysis, design and construction requirements and resultant specifications are to recognize and provide for safe functional performance.

Joint Use of Reservoir Capacity

A reservoir site may be used more efficiently where hydrologic conditions permit joint use of storage capacity by flood water and conservation storage. The following requirements must be met for joint use storage dams.

1. There is reasonable assurance that water will be available to meet objectives.

2. Flood protection objectives of the project are satisfied.

3. Spillway conditions are such that the dam will perform safely.

Special hydrologic studies are to be made to show that the requirements can be met. This may include hydrometeorologic instrumentation and analysis.

Hydraulic features are to include an ungated spillway outlet at the top of the joint use pool. A gated opening is to be provided at the bottom of the joint use pool adequate for use of the conservation storage and evacuation of the joint use pool.

Provision are to be made for operation of the joint use pool to insure functioning of the dam as designed. These are to include a competent operating and maintaining organization and a specific operation and maintenance plan. These requirements are to be a part of the planning process and agreed to by the sponsors or owner.

Visual Resource Design

The public generally prefers lake or waterscape scenery. Therefore, when permanent pools are created by dam construction, they can enhance the visual resource if the water views are emphasized. A visual design objective is to focus public views toward the permanent pool and reduce the visual focal effects of the structural elements.

Visual focus on the lake is achieved by locating roads and walkways so that the entering or first perceptions of the site are of the waterscape scenery. In most landscapes, the lake will automatically predominate if other elements are visually designed to be subordinate.

Borrow areas are to be shaped to blend with the surrounding topography. These areas are to be revegetated with herbaceous and woody plants to visually fit the existing surrounding vegetation. Fences are to be constructed parallel to the contour as much as possible, be located behind existing vegetation as seen from the major view points and be placed low in the landscape. Dams are to be shaped to blend with the natural topography to the extent feasible.

Safety and Protection

Many dams are hazardous to the public. Features designed for recreation or fish and wildlife are especially attractive to the public since they provide an opportunity to use the water. All dams are to be designed to avoid hazardous conditions where possible. Open-top risers, steep-walled channels and chutes, plunge pools and stilling basins are hazardous and require special attention. All dams are to be provided with safety fences, guard rails or other safeguards as necessary to protect the public and operation and maintenance personnel.

The embankment and spillways are to be fenced where necessary to protect the dam from livestock and foot and vehicular traffic.

Water Supply Pipes

Water supply pipes or conduits for other purposes installed under any part of the embankment below the crest of the emergency spillway are to: (1) provide durability, strength and flexibility equivalent to the principal spillway, (2) be watertight against anticipated pressures, (3) be adequate for their intended use and (4) be provided with seepage control against potential piping.

Streamflow Diversion During Construction

Streamflow past the dam site, unless controlled, occurs at a somewhat random time with variable frequency of magnitudes. A hazard exists during dam construction beginning when the embankment, cofferdam or other ancillary structures obstruct the natural streamflow. During construction a greater risk usually exists for some time period than after the dam is completed. The risk is different for each dam because of the varying factors of construction time, climate, watershed size and diversion capacity. An evaluation is to be made of the risk from embankment failure by overtopping and other similar hazards during construction. The risk involved in overtopping during construction increases with the following factors.

1. Dams of higher hazard class.
2. Greater volume of reservoir storage.
3. Dams with larger watersheds.
4. Longer critical construction time periods.
5. Smaller diversion "release" rates (less unit discharge per unit watershed area).

The consequence of overtopping during construction may vary from a slight amount of erosion on a homogeneous clay dam, to a breach of an embankment including loss of a temporary diversion coffer dam. The erosion or breach causes increased inundation and sedimentation of downstream areas.

The risk may be evaluated based upon experience of comparable dams constructed in the same hydrologic setting. An evaluation may also be made using available streamflow records to obtain stage-duration-frequency information for a range of diversion rates. Streamflow data should be used when available, otherwise, an evaluation may be made using climatalogical record data, for generation of synthetic hydrographs to develop stage-duration-frequency information for a range of diversion rates.

The size of diversion is to be designed to provide an acceptable level of risk. The probability required to protect against overtopping varies from 20 percent to 5 percent chance in any one year. A 10 percent chance probability is frequently used when the critical construction period is limited to one construction season. An alternative to a larger diversion capacity is to provide protection against erosion to the embankment surface (reinforcement) up to the desired elevation of acceptable risk.

Reservoir Conservation Storage

Reservoirs with water stored for conservation purposes are to be analyzed using a water budget to determine a dependable water supply.

For most purposes a dependable water supply is defined as one that is available at least 8 out of 10 years or has a probability of 80 percent chance in any one year. A purpose such as municipal and industrial water may require a 95 percent chance probability of existing in any one year. Other purposes, such as recreation requires an analysis of the reservoir surface elevation fluctuation in order to evaluate the acceptable percent chance of occurrence.

HYDROLOGY

This section describes hydrologic criteria for determining spillway discharges and floodwater storage volumes. Detailed procedures for developing principal spillway, emergency spillway and freeboard hydrographs are contained in Chapter 21, Section 4, Hydrology, of the SCS National Engineering Handbook (NEH-4). Methods of flood routing hydrographs through reservoirs and spillway systems are contained in Chapter 17, NEH-4. "Special Studies," as used in this text, refer to all site specific studies with prior concurrence of selected procedures.

Precipitation and Runoff Volumes

Principal Spillway

Precipitation data are to be obtained from the most recent National Weather Service (NWS) reference which is applicable to the area under study. References which contain precipitation data for return periods up to 100 years and for durations up to 10 days are listed in Sections A and B of Table 2-1.

The return period for design precipitation amounts is dependent on the dam classification, purpose, size, location and type of emergency spillway. Minimum return periods are shown in Table 2-2. The minimum allowable areal adjustment ratios for 1 and 10-day precipitation amounts are tabulated in Table 2-3, part A.

A storm duration of not less than 10 days is to be used for sizing the principal spillway. The procedure in Chapter 21, NEH-4 for developing the storm distribution uses both the 1-day and 10-day runoff volumes.

The procedure for estimating runoff volumes is to be selected based on which one requires the higher emergency spillway crest elevation when the principal spillway hydrograph is routed through the structure. Procedures to be used to estimate runoff volumes include:

(1) The runoff curve number (CN) procedure described in NEH-4. Use average antecedent moisture conditions (AMC II) or greater unless a special study shows that a different condition is justified. The CN adjustment for a 10-day storm is estimated from Table 2-3, part B.

(2) Runoff volumes from Figures 2-1 (A) and (B) or Figures 2-2 (A) and (B).

A special study may show that local streamflow records can be used directly or regionalized to develop design runoff volumes.

Transmission losses reducing the runoff volume in arid and semiarid climatic areas may be used if the climatic index, as defined in Chapter 21, NEH, is less than one. If transmission losses appear to be significant even though the climatic index is one or more, such as in cavernous areas, special studies are required.

Obtain quick return flow from the map, Figure 2-1(C), or Table 2-4 as appropriate.

Emergency Spillway and Freeboard

Table 2-5 establishes the minimum design precipitation amounts by dam class. The most recent NWS references are to be used to estimate precipitation amounts in all areas. The references are listed in Sections A and C of Table 2-1. The generalized maps, Figures 2-3 and 2-4, can be used to establish the 10-square mile, 100-year and the probable maximum precipitation (PMP) for the 37 contiguous states east of the 103rd meridian. Figure 2-5 shows the location in which the NWS references for PMP are applicable.

Areal adjustment and storm distribution factors contained in the NWS references, listed on Figure 2-5, are to be used in their respective regions. For areas not covered by an NWS publication minimum areal adjustment ratios for design precipitation amounts are shown on Graph A, Figure 2-6. No areal adjustments are to be made for areas less than 10 square miles.

The minimum storm duration to be used is 6-hours. If the time of concentration (Tc) exceeds 6-hours, the minimum design storm duration is to be equal to the Tc. When the Tc exceeds 6-hours, the precipitation amounts must be increased by the values in the applicable NWS references (Figure 2-5). The duration adjustment shown on Graph B, Figure 2-6 may be used in areas where the NWS references are not applicable.

For those locations where NWS references provide estimates of local storm (thunderstorm) and general storm PMP values, the storm duration and distribution that result in the maximum reservoir stage when the hydrograph is routed through the structure should be used. Unless a specific distribution is recommended in a NWS reference, the distribution of precipitation with time should be approximately the same as that shown in Graph C, Figure 2-6. For longer duration storms recommended distributions from NWS references should be used.

The runoff curve number (CN) procedure in NEH-4 is used to determine runoff volumes using AMC II or greater. The CN applies throughout the design storm regardless of the storm duration.

The NWS may be requested to make special PMP studies for any location. This includes, but is not limited to drainage areas larger than 100 square miles, areas of significant variation in elevation, or areas located at the boundary of two studies where discontinuities in published values occur.

Design Hydrographs

Principal Spillway Hydrographs

Procedures in Chapters 16 and 21, NEH-4 and applicable national computer programs are to be used to develop the principal spillway hydrograph using precipitation and runoff amounts as described in the preceding section.

When the area above a proposed dam is hydrologically complex, the area should be divided into two or more hydrologically homogeneous subbasins for developing the design hydrograph.

Streamflow records may be used to develop the principal spillway hydrograph where a special study shows they are adequate for this purpose.

Emergency Spillway & Freeboard Hydrographs

Procedures in Chapters 16 and 21, NEH-4, and applicable national computer programs are to be used to develop emergency spillway and freeboard hydrographs using precipitation and runoff amounts and subbasins if necessary as described in the preceding sections.

Dams in Series

Upper Dam

The hydrologic criteria and procedures for the design of an upper dam in a system of dams in series are to be the same as, or more conservative than those for dams downstream if failure of the upper dam could contribute to failure of the lower dam. The dam breach criteria contained on page 1-1 will be used to develop the breach hydrograph peak discharge.

Lower Dam

For the design of a lower dam, hydrographs are to be developed for the areas controlled by the upper dams based on the same hydrologic criteria as the lower dam. The hydrographs are routed through the spillways of the upstream dams and the outflows routed to the lower dam where they are combined with the hydrograph from the intermediate uncontrolled drainage area. The combined principal spillway hydrograph is used to determine the capacity of the principal spillway and the floodwater retarding storage requirement for the lower site. The combined emergency spillway hydrograph and the combined freeboard hydrograph are used to determine the size of the emergency spillway and the height of dam at the lower site.

If upon routing a hydrograph through the upper dam, the dam is overtopped, or its safety is questionable, it is to be considered breached. For design of the lower dam the breach hydrograph is to be routed downstream to the lower dam and combined with the uncontrolled area hydrograph.

In design of the lower dam, the time of concentration (Tc) of the watershed above an upper dam is used to develop the hydrographs for the upper dam. The Tc of the uncontrolled area above the lower site is used to develop the uncontrolled area hydrographs. If the Tc for the total area exceeds 6-hours, the precipitation amounts for the emergency spillway and freeboard hydrographs must be increased by the values in the applicable NWS references (Figure 2-5).

The minimum precipitation amounts for each of the required hydrographs may be reduced by the areal reduction factor for the total drainage area of the dam system.

Large Drainage Areas

When the area above a proposed dam approaches 50 square miles, it is desirable to divide the area into hydrologically homogeneous subbasins for developing the design hydrographs. Generally, the drainage area for a subbasin should not exceed 20 square miles. Watershed modeling computer programs, such as the SCS Technical Release 20-Project Formulation-Hydrology or DAMS2-Structure Site Analysis, may be used for inflow hydrograph development.

If the Tc for the entire drainage area is greater than 6 hours, storm durations longer than the Tc should be tested to determine the duration that gives the maximum reservoir stage for the routed emergency spillway and freeboard hydrographs.

Precipitation amounts may exhibit marked variation in a large watershed. This variation is based upon topographical and meteorological parameters such as aspect, orientation, mean elevation of subbasin, and storm orientation. Consideration is to be given to having the NWS make a special PMP study for large watersheds with drainage areas more than 100-square miles. Individual watershed PMP studies can take into account orographic features that are smoothed in the generalized precipitation studies. A special study also may be warranted in areas where significant snow melt can occur during the design storms.

Studies to make use of available stream flow records are encouraged for purposes such as unit hydrograph development, watershed storage and timing effects, and calibration of watershed models.

TABLE 2-1

NATIONAL WEATHER SERVICE REFERENCES* - PRECIPITATION DATA

A. Durations to 1 day and return periods to 100 years

Technical Memorandum HYDRO-35. Durations 5 to 60 minutes for the eastern and central states (1977)

Technical Paper 40. 48 contiguous states (1961) (Use for 37 contiguous states east of the 105th meridian)

Technical Paper 42. Puerto Rico and Virgin Islands (1961)

Technical Paper 43. Hawaii (1962)

Technical Paper 47. Alaska (1963)

NOAA Atlas 2. Precipitation Atlas of the Western United States (1973).

Vol. 1, Montana	Vol. II, Wyoming	Vol. III, Colorado
Vol. IV, New Mexico	Vol. V, Idaho	Vol. VI, Utah
Vol. VII, Nevada	Vol. VIII, Arizona	Vol. IX, Washington
Vol. X, Oregon	Vol. XI, California	

B. Durations from 2 to 10 days and return periods to 100 years

Technical Paper 49. 48 contiguous states (1964) (Use SCS West National Technical Center Technical Note - Hydrology - PO-6 Rev. 1973, for states covered by NOAA Atlas 2).

Technical paper 51. Hawaii (1965)

Technical paper 52. Asaska (1965)

Technical paper 53. Puerto Rico and Virgin Islands (1965)

C. Probable maximum precipitation (PMP) (See Figure 2-5)

Hydrometeorological Report 36. California Pacific drainage (1961)

Hydrometeorological Report 39. Hawaii (1963) (PMP maps in TP-43** are based on HMR-39)

Hydrometeorological Report 43. Northwest states Pacific drainage (Rev. 1981)

Hydrometeorological Report 49. Colorado River and Great Basin Drainages (1977)

Hydrometeorological Report 51. For 37 contiguous states east of the 103rd meridian (1978)

Hydrometeorological Report 52. Application of PMP estimates, states east of the 105th meridian (1982)

Hydrometeorological Report 53. Seasonal variation of 10 square-mile PMP estimates, states east of the 105th meridian (1980)

Hydrometeorological Report 54. PMP and snowmelt criteria for southeast Alaska (1983)

Hydrometeorological Report 55. Between the Continental Divide and the 103rd meridian (1984)

Technical Paper 42.** Puerto Rico and Virgin Islands (1961)

Technical Paper 47.** Alaska (1963)

New studies are in progress in the Tennessee River Watershed

*National Weather Service National Oceanic and Atmospheric Administration (NOAA), U.S. Department of Commerce; formerly U.S. Weather Bureau.

**Technical papers listed in both A and C.

TABLE 2-2

MINIMUM PRINCIPAL SPILLWAY HYDROLOGIC CRITERIA

Class of Dam	Purpose of Dam	Product of Storage x Effective Height	Existing or Planned Upstream Dams	Precipitation Data for Maximum Frequency of Use of Emergency Spillway Types: 1/	
				Earth	Vegetated
(a)	single 2/ irrigation only	less than 30,000	none	1/2 design life	1/2 design life
		greater than 30,000		3/4 design life	3/4 design life
	single or multiple 4/	less than 30,000	none	P_{50}	P_{25} 3/
		greater than 30,000		1/2 (P_{50} + P_{100})	1/2 (P_{25} + P_{50})
		all	any 5/	P_{100}	P_{50}
(b)	single or multiple	all	none or any	P_{100}	P_{50}
(c)	single or multiple	all	none or any	P_{100}	P_{100}

1/ Precipitation amounts by return periods in years. In some areas direct runoff amounts determined by figure 2-1 and 2-2 or procedures in Chapter 21, NEH-4 should be used in lieu of precipitation data.

2/ Applies to irrigation dams on ephemeral streams in areas where the annual rainfall is less than 25 inches.

3/ The minimum criteria are to be increased from P_{25} to P_{100} for a ramp spillway.

4/ Class (a) dams involving industrial or municipal water are to be designed with a minimum criteria equivalent to that of class (b).

5/ Applies when the upstream dam is located so that its failure could endanger the lower dam.

TABLE 2-5

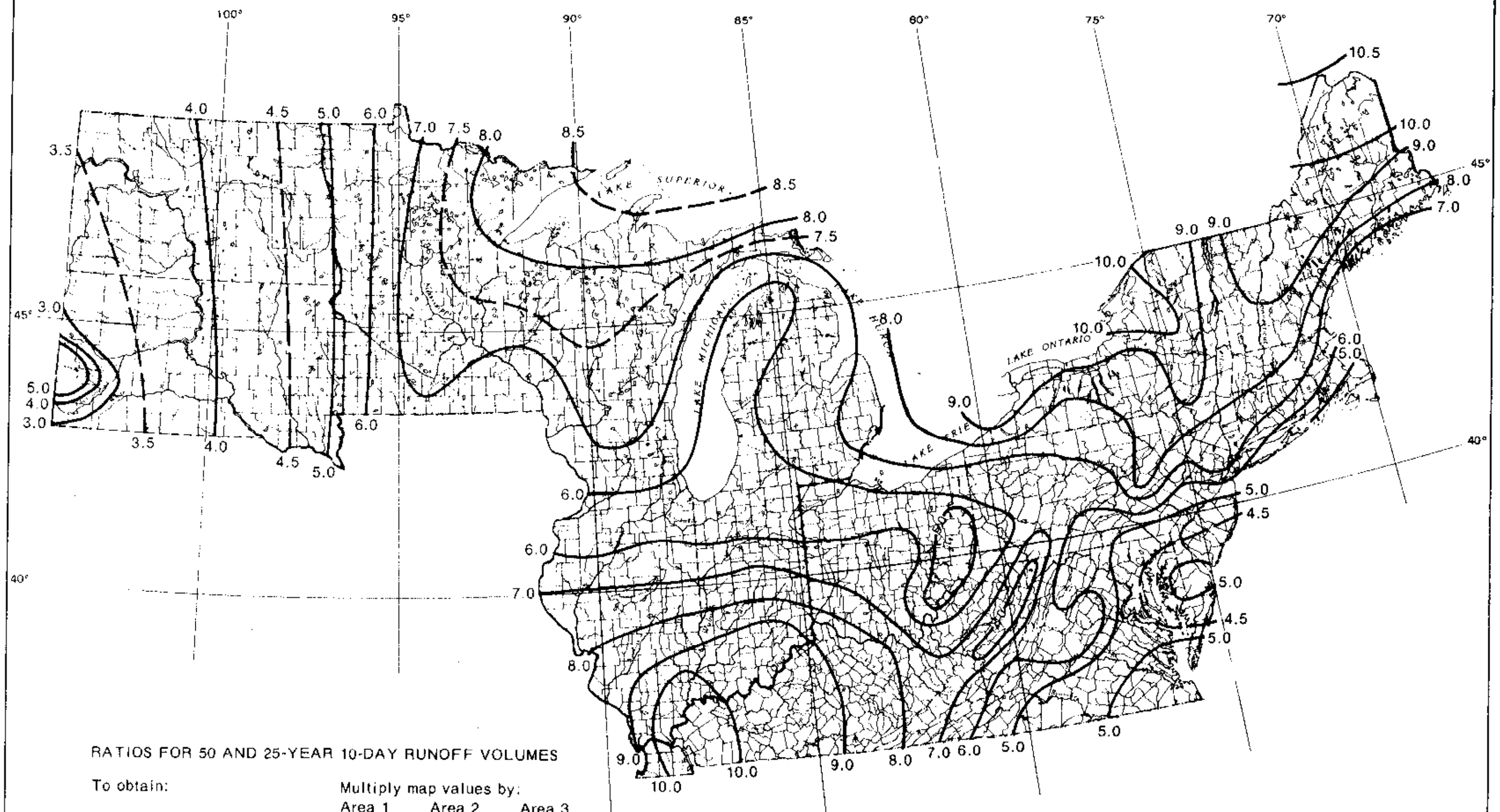
MINIMUM EMERGENCY SPILLWAY HYDROLOGIC CRITERIA

Class of Dam	Product of Storage x Effective Height	Existing or Planned Upstream Dams	Precipitation Data for [1/]	
			Emergency Spillway Hydrograph	Freeboard Hydrograph
(a) [2/]	less than 30,000	none	P_{100}	$P_{100} + 0.12\ (PMP - P_{100})$
	greater than 30,000	none	$P_{100} + 0.06\ (PMP - P_{100})$	$P_{100} + 0.26\ (PMP - P_{100})$
	all	any [3/]	$P_{100} + 0.12\ (PMP - P_{100})$	$P_{100} + 0.40\ (PMP - P_{100})$
(b)	all	none or any	$P_{100} + 0.12\ (PMP - P_{100})$	$P_{100} + 0.40\ (PMP - P_{100})$
(c)	all	none or any	$P_{100} + 0.26\ (PMP - P_{100})$	PMP

[1/] P_{100} = Precipitation for 100-year return period. PMP = Probable maximum precipitation.

[2/] Dams involving industrial or municipal water are to use minimum criteria equivalent to that of class (b).

[3/] Applies when the upstream dam is located so that its failure could endanger the lower dam.



RATIOS FOR 50 AND 25-YEAR 10-DAY RUNOFF VOLUMES

To obtain:	Multiply map values by: Area 1	Area 2	Area 3
50-YEAR 10-DAY RUNOFF	0.85	0.90	0.92
25-YEAR 10-DAY RUNOFF	0.70	0.80	0.85

FIGURE 2-1 (A)
100-YEAR, 10-DAY RUNOFF (INCHES)
PRINCIPAL SPILLWAY HYDROGRAPH

JANUARY 1971, REV. APRIL 1976

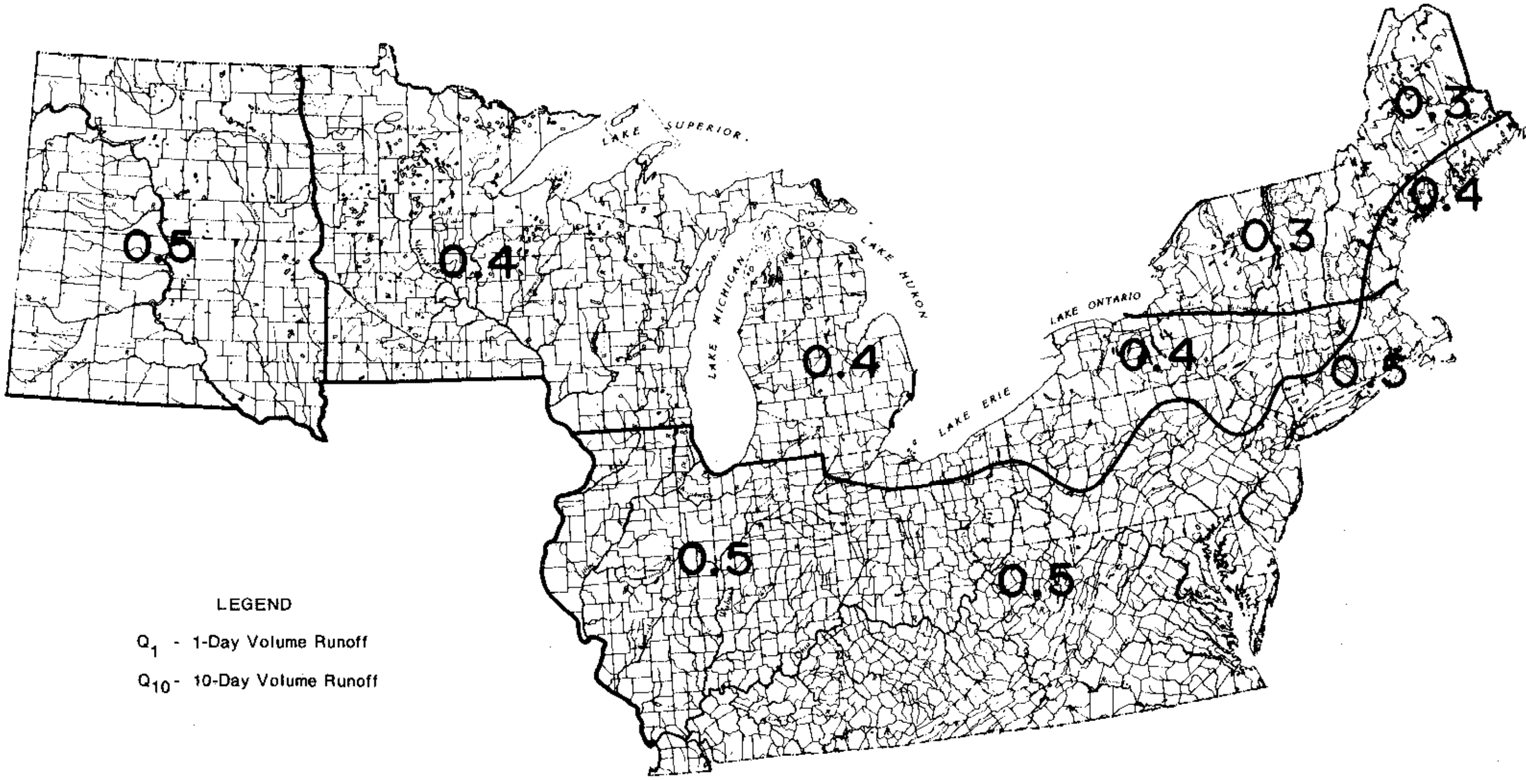


FIGURE 2-1 (B)

RATIOS OF VOLUMES OF RUNOFF (Q_1/Q_{10})

PRINCIPAL SPILLWAY HYDROGRAPH

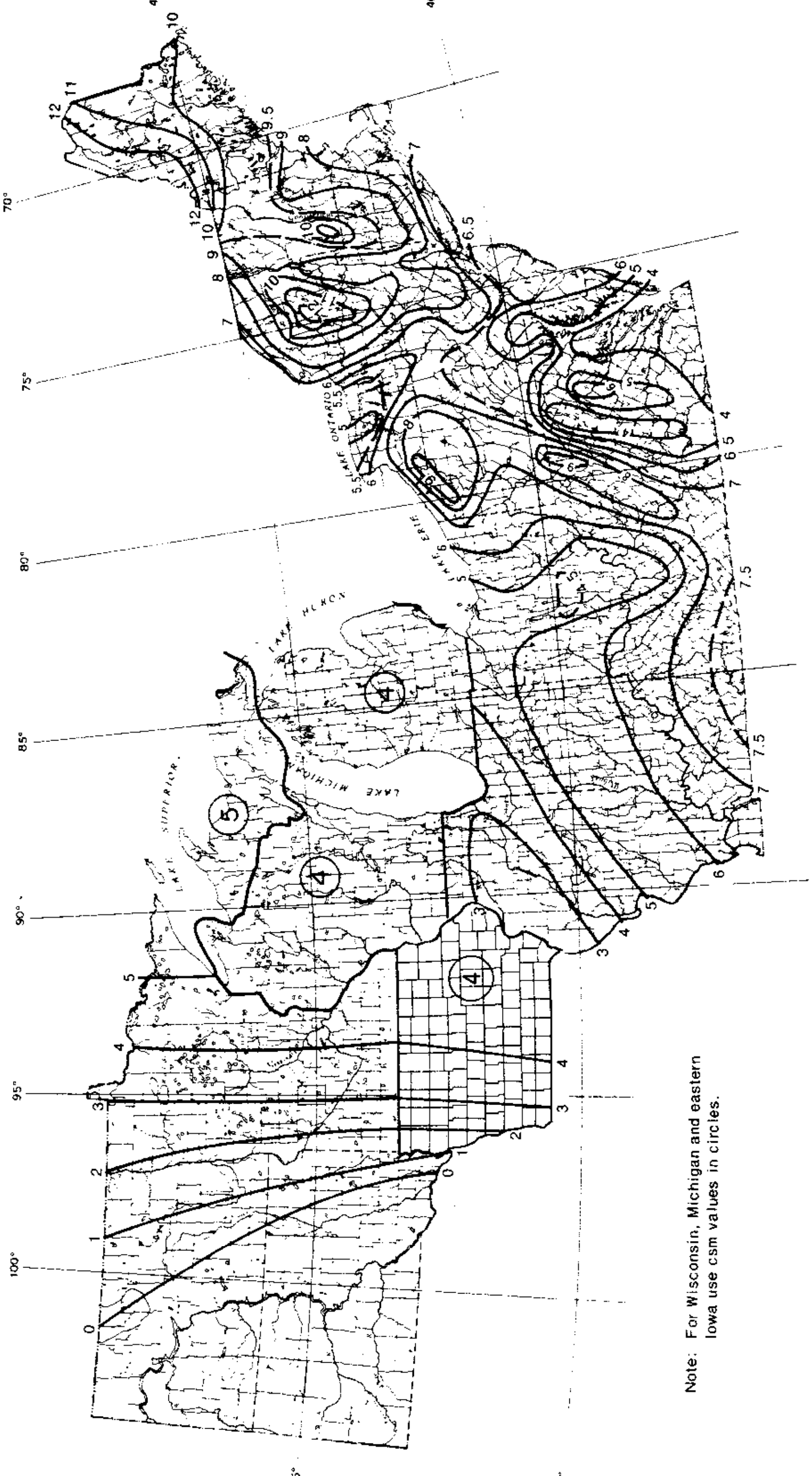
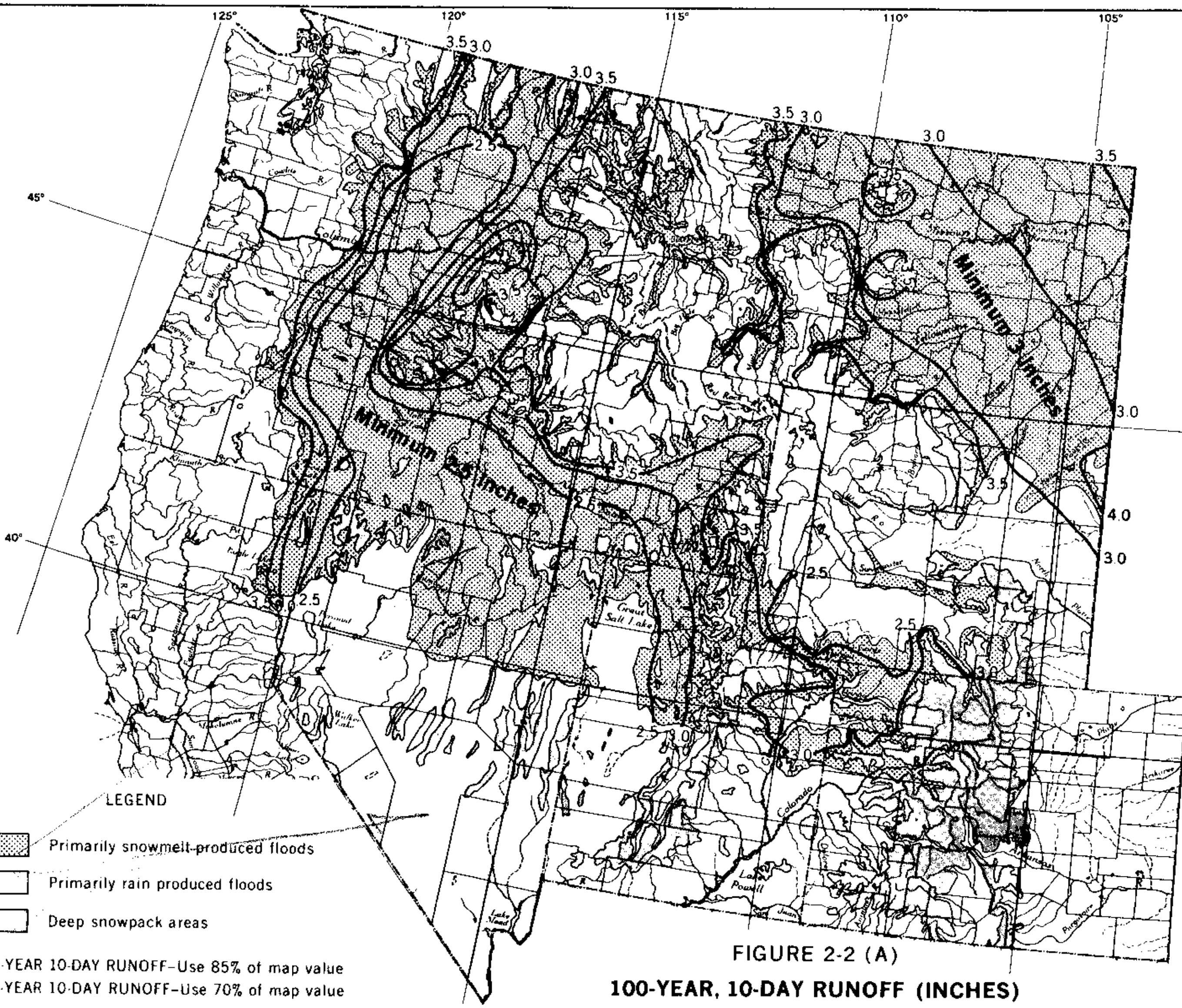


FIGURE 2-1 (C)
QUICK RETURN FLOW (csm)
PRINCIPAL SPILLWAY HYDROGRAPH

JANUARY 1971, REV. APRIL 1976



LEGEND

Primarily snowmelt-produced floods

Primarily rain produced floods

Deep snowpack areas

50-YEAR 10-DAY RUNOFF–Use 85% of map value
25-YEAR 10-DAY RUNOFF–Use 70% of map value

January 1971, Rev. April 1976

FIGURE 2-2 (A)

100-YEAR, 10-DAY RUNOFF (INCHES)

PRINCIPAL SPILLWAY HYDROGRAPH SNOWMELT PRODUCING FLOOD AREAS

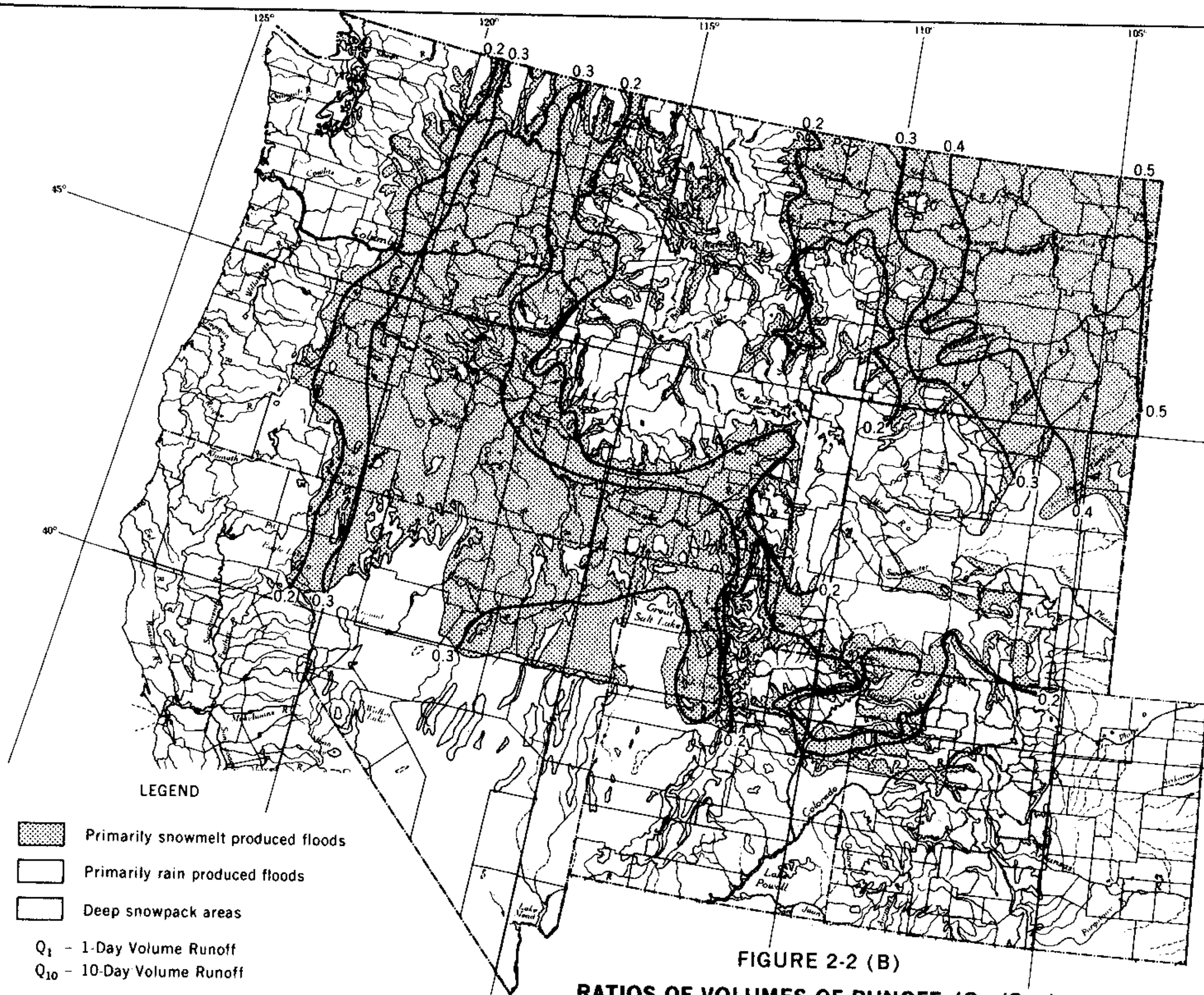


FIGURE 2-2 (B)

RATIOS OF VOLUMES OF RUNOFF (Q_1/Q_{10})

PRINCIPAL SPILLWAY HYDROGRAPH SNOWMELT PRODUCING FLOOD AREAS

JANUARY 1971, REV. APRIL 1976

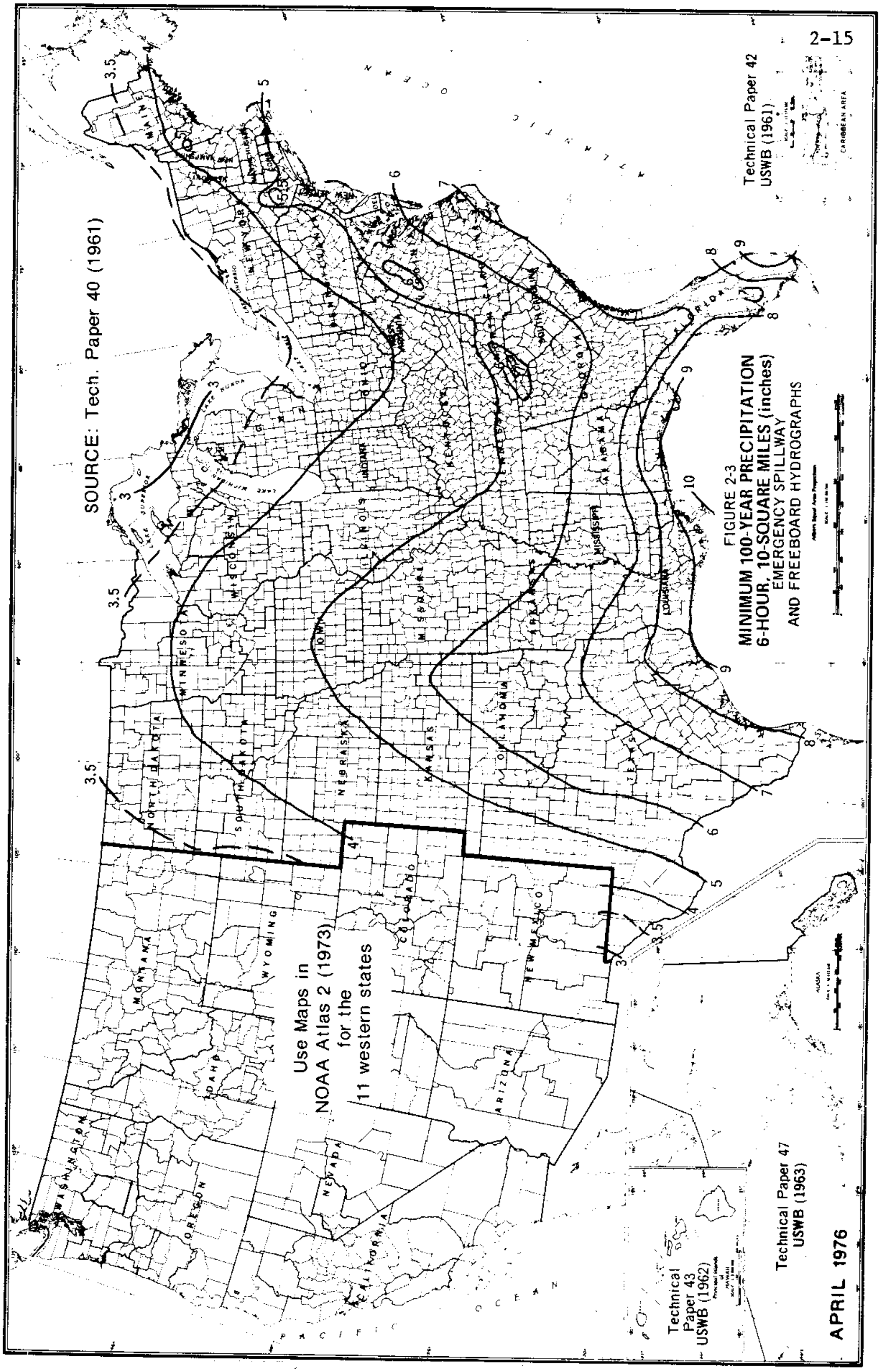


FIGURE 2-3
MINIMUM 100-YEAR PRECIPITATION
6-HOUR, 10-SQUARE MILES (inches)
EMERGENCY SPILLWAY
AND FREEBOARD HYDROGRAPHS

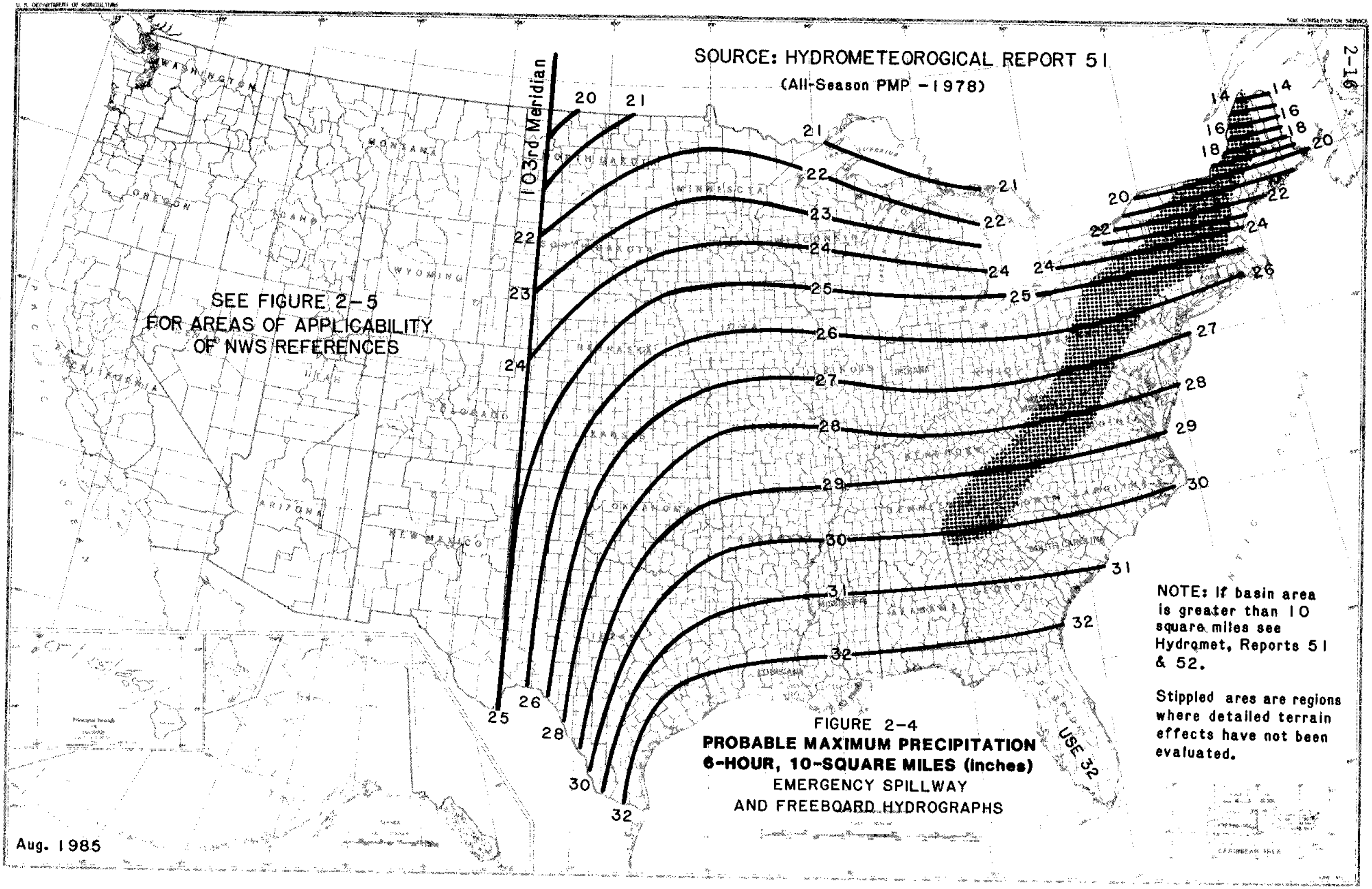


FIGURE 2-4
PROBABLE MAXIMUM PRECIPITATION
6-HOUR, 10-SQUARE MILES (inches)
EMERGENCY SPILLWAY
AND FREEBOARD HYDROGRAPHS

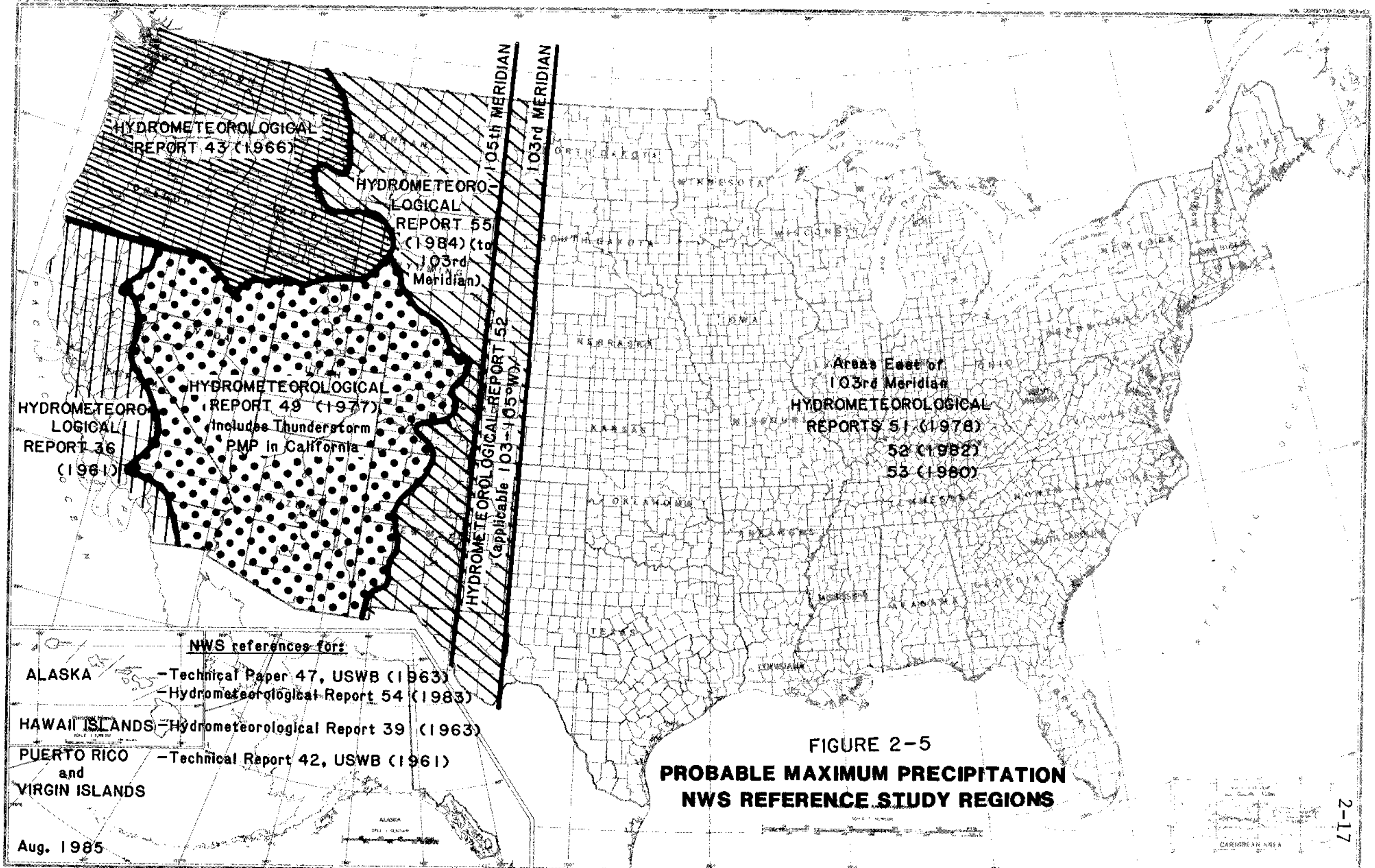
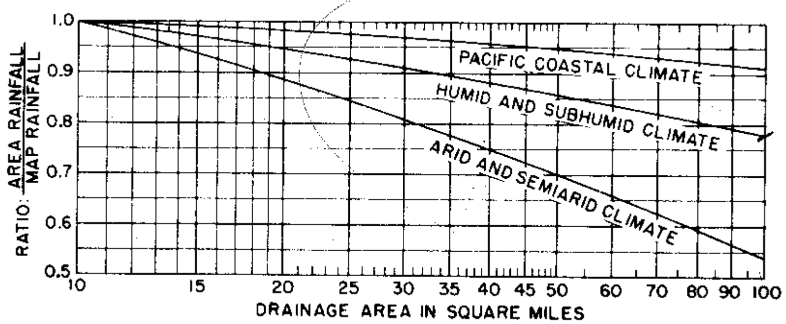
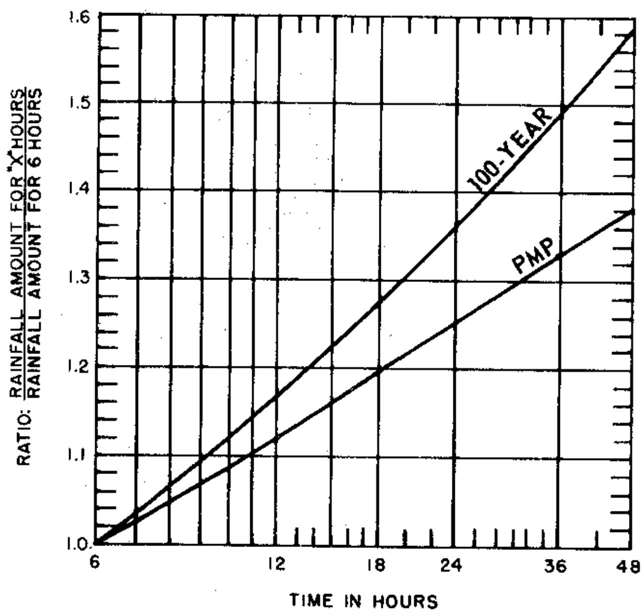


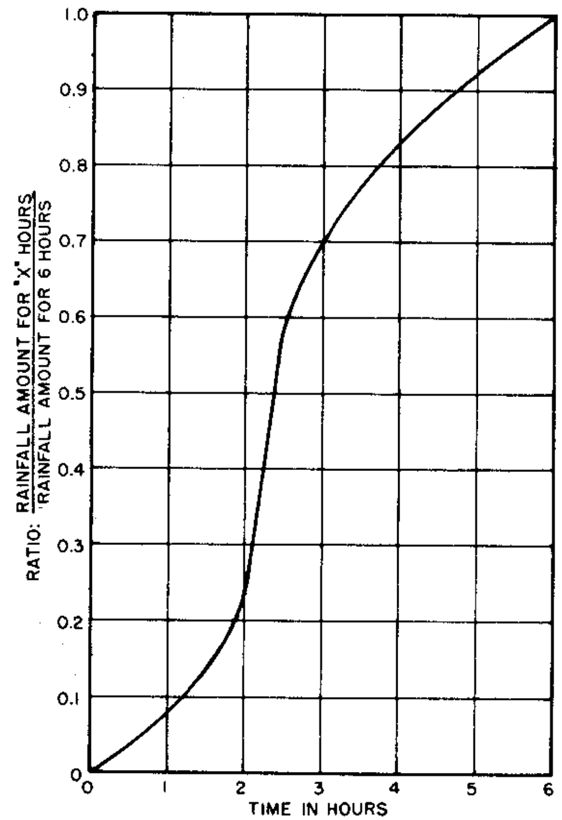
FIGURE 2-5
PROBABLE MAXIMUM PRECIPITATION
NWS REFERENCE STUDY REGIONS



(A) AREAL PRECIPITATION ADJUSTMENTS FOR DRAINAGE AREAS 10 to 100 SQUARE MILES



(B.) RELATIVE INCREASE IN RAINFALL AMOUNT FOR STORM DURATIONS OVER SIX HOURS



(C.) SIX HOUR DESIGN STORM DISTRIBUTION

FIGURE 2-6

EMERGENCY SPILLWAY AND FREEBOARD

VOLUME ADJUSTMENTS AND STORM DISTRIBUTION

FOR AREAS WHERE NWS REFERENCES DO NOT CONTAIN AN APPLICABLE PROCEDURE

Aug. 1981

SEDIMENTATION

Reservoirs used to store or retard water from surface runoff will trap and store a large portion of the sediment in the runoff water. Therefore, allocate storage capacity for the calculated sediment accumulation during the design life of the reservoir. Criteria and general procedures needed to determine the volume required for sediment accumulation and its allocation in the reservoir are contained in NEH-3, Sedimentation and TR 12. The latter also includes procedures for determining:

1. The sediment yield for present conditions and for the future after planned land treatment and other measures are applied in the drainage area of the dam.
2. The trap efficiency of the reservoir.
3. The distribution and types of sediment expected to accumulate.
4. The proportion of the sediment that will be continously submerged vs. that aerated, and
5. The densities to which the sediment will become compacted.

If the amount of sediment accumulation calculated exceeds two watershed inches in 50 years for the uncontrolled drainage area of the dam, reevaluate the entire watershed to determine if more economical methods of reducing sediment yield or trapping sediment may be feasible and applicable.

GEOLOGIC INVESTIGATIONS

The intensity and detail of geologic site investigations are to be consistent with the class of dam, complexity of site geology and the data needed for design. General requirements, procedures, and criteria are set forth in 210-V, Part 531 and NEH-8.

Geologic conditions that require special consideration beyond the minimum investigations spelled out in the above reference are:

1. Seismic Assessment -- Dams in zones 3 and 4, Alaska, Puerto Rico and the Virgin Islands and high hazard [class(c)] dams in zone 2 (see figure 4-1) require special investigations to determine liquefaction potential of noncohesive strata, including very thin layers, and the presence at the site of any faults active in Holocene time. As part of this investigation, a map is to be prepared showing the location and intensity of magnitude of all intensity V or magnitude 4 or greater earthquakes of record, and any historically active faults, within a 100 kilometer radius of the site. (Obtain earthquake information for this map in printout form from the Environmental Data Service attention D62, NOAA, Boulder, Colorado 80302. Telephone: FTS 323-6472; Commercial (303) 499-1000, ext. 6472). The report should also summarize other possible earthquake hazards such as ground compaction, landslides, excessive shaking of unconsolidated soils, seiches, and in coastal areas, tsunamis.

2. Subsidence -- Investigate the potential for surface subsidence due to past or future solid, liquid (including ground water) or gaseous mineral extraction. Consult the state geological and/or evaluation. National Engineering Manual 210-V, Part 531, Subpart D sets forth criteria for these evaluations.

   Evaluate the impact of the preemption of mineral deposits, including sand and gravel, by dams and reservoirs.

   In arid and semiarid areas and in eolean deposits determine the potential of moisture deficient soil materials to collapse upon saturation or wetting. If the potential exists, make extensive and intensive site investigations to provide quantitative information for design and construction.

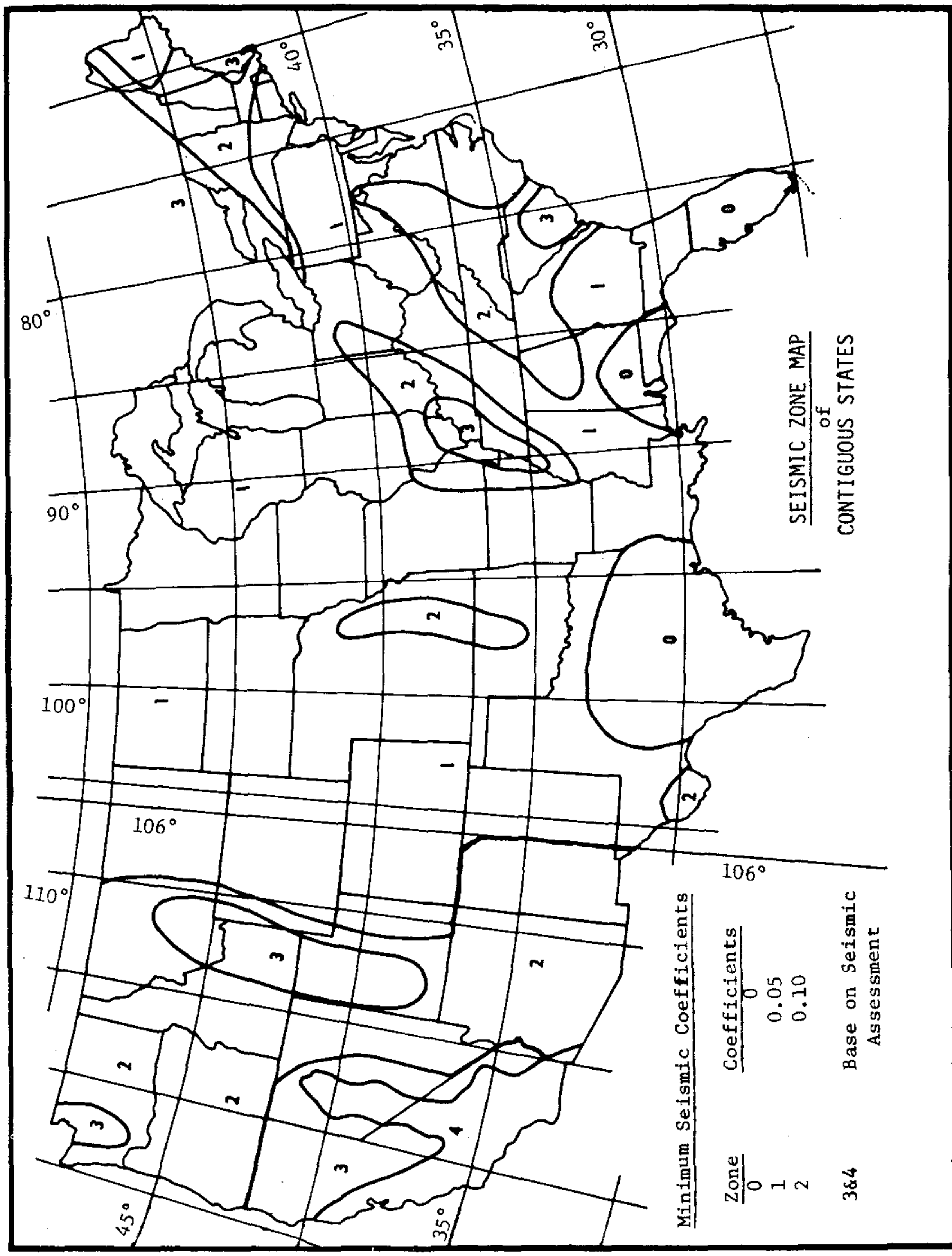
3. Emergency Spillways -- Large dams with emergency spillways in soft rock or cemented soil materials that cannot be classified as soil as defined in TR 52 nor as rock as generally defined for engineering purposes, and spillways in rocks with extraordinary defects require a special individual evaluation.

4. Mass Movements -- Evaluate landslides and landslide potential at dam and reservoir sites, especially those in shales and where unfavorable dip-slope or other adverse rock attitudes occur. Summarize the history of mass movement in the project area. Emergency spillway cuts and reservoir effects must be given careful consideration.

5. Karstic areas -- Limestone and gypsum in reservoirs and at damsites require special investigational methods and careful evaluation of subsidence, leakage hazards and construction costs. Multipurpose structures in these areas are especially critical.

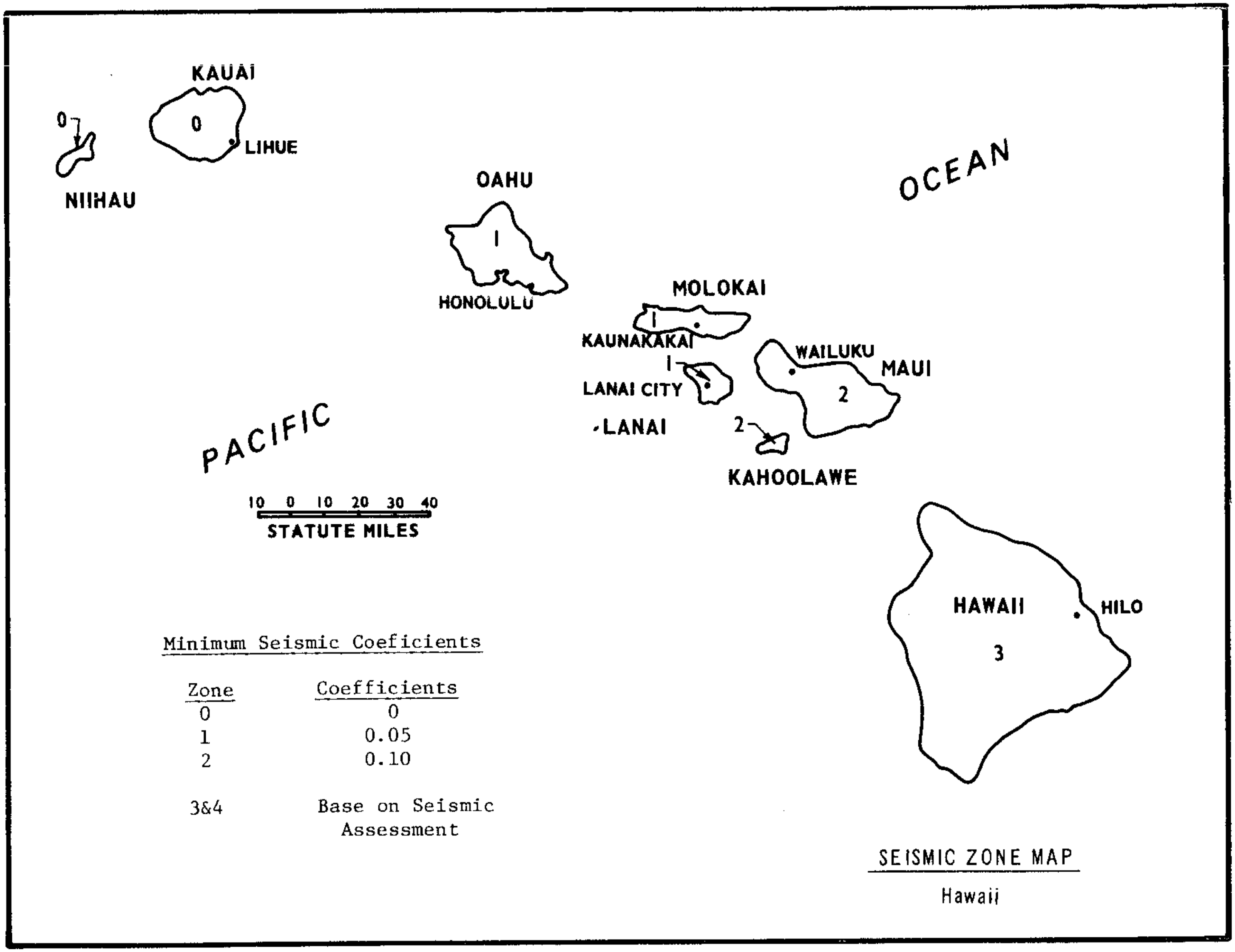
6. Multipurpose Dams -- Investigate the ground water regime and hydraulic characteristics of the entire reservoir area of water storage dams and evaluate for leakage. Use the water budgets to determine the need for reservoir sealing.

7. Other -- Special studies and evaluations may be necessary where such things as compaction shales; some types of siliceous, calcareous or pyritic shales; rebound joints; dispersed soils or artesian waters occur at a site.



Adapted From TM 5-809-10 NAVFAC P-355 AFM 88-3, Chapter 13; April 1973

Figure 4-1 Sheet 1 of 2



Minimum Seismic Coeficients

Zone	Coefficients
0	0
1	0.05
2	0.10
3&4	Base on Seismic Assessment

SEISMIC ZONE MAP

Hawaii

Adapted From TM 5-809-10 NAVFAC 0-355 AFM 88-3, Chapter 13; April 1973

Figure 4-1 Sheet 2 of 2

EARTH EMBANKMENTS AND FOUNDATIONS

Earth embankments constructed of soil and rock are the principal means of impounding water. The earth embankment and its foundation are to withstand the anticipated loads without movements leading to failure. Measures are to be provided for adequate seepage control.

Height

The design height of an earth embankment is to be sufficient to prevent overtopping during passage of either the (1) freeboard hydrograph or (2) the emergency spillway hydrograph plus the freeboard required for frost conditions or wave action, whichever is larger. The design height must also meet the requirements for minimum emergency spillway depth. The design height of the dam is to be increased by the amount needed to compensate for settlement.

Top Width

The minimum top width of embankment is shown in Table 5-1.

Table 5-1

Minimum Top Width of Embankment

Total Height of Embankment (H) (feet)	Top Width (feet)		
	All Dams	Single Purpose Floodwater Retarding	Multipurpose or other Purposes
14 or less	8		
15 - 19	10	NA	NA
20 - 24	12		
25 - 34	14		
35 - 95	NA	14	$\frac{H + 35}{5}$
Over 95	NA	16	26

The width may need to be greater than the above minimums to: (1) meet state and local standards, (2) accommodate embankment zoning, (3) provide roadway access and traffic safety, and (4) provide structural stability. An increase in top width is a major design feature in preventing breaching after embankment slumping caused by earthquake ground motion.

When the embankment top is used as a public roadway, the minimum width is to be 16 feet for one-way and 26 feet for two-way traffic. Guardrails or other safety measures are to be used and are to meet the requirements of the responsible road authority.

Embankment Slope Stability

Analyze the stability of embankment slopes using generally accepted methods based on sound engineering principles. Document all analyses or considerations in appropriate design reports and files. Design the embankment cross section to provide adequate factors of safety against sliding, sloughing or rotation in the embankment and foundation. Use the appropriate degree of conservatism in the analysis that is consistent with the adequacy of the site investigation and soil testing program, and with the complexity of the site and consequences of failure. Minimum factors of safety are listed in table 5-2.

Evaluate the effect of seismicity on each site. Include the determination of whether the site is in a seismically active area, its proximity to active faults, and the predicted site ground motion intensity. Use the minimum seismic coefficient shown in Figure 4-1 in the slope stability analysis when no special seismic assessment is made.

Analyze embankment slope stability for the conditions and periods during the design life of the structure that are most critical or severe. Use conditions in the analysis based on possible critical water levels and loadings of the embankment and foundation. Consider and document the following conditions or periods: (1) end of construction, (2) steady seepage, (3) rapid drawdown, and (4) during seismic loading. Use the shear strength parameters and minimum factors of safety indicated in Table 5-2 for each case. The nomenclature for the various shear strength tests is listed in Table 5-3.

Calculate the factor of safety based on the ratio of the shear strength available to the shear strength mobilized.

Clearly document the conditions not analyzed, correlated shear strength parameters, or correlation to field performance.

Consider the following additional details in conjunction with Table 5-2.

I. End of Construction. Make a detailed analysis of this case when significant pore pressure development during construction is expected in either the embankment or foundation soils. Embankment soils are to be tested at the highest likely placement water content and foundation soils are to be tested at saturation. Select shear strength parameters according to Table 5-2.

II. Rapid Drawdown. Analyze the stability of the upstream embankment slope for the condition created by a rapid drawdown of the water level in the reservoir. Use a phreatic surface in the embankment developed from the emergency spillway crest, with the water level in the reservoir drawn down to the crest of the lowest gated or ungated outlet. Failure surfaces are to be confined to the embankment only if foundation soils are free-draining. Use infinite slope stability equation appropriate to drainage conditions of the embankment soils whenever the c or $\bar{c}$ value being used in the analysis is zero. In addition to the infinite slope analysis, circular arch or sliding wedge analyses may be needed to check stability where failure surfaces could extend into weaker core zones or into the foundation. Select shear strength parameters according to Table 5-2.

III. Steady Seepage Without Seismic Forces. Analyze the downstream slope for the condition of steady seepage developed from the water surface at the principal spillway crest. In the analysis, saturated soils will be subjected to uplift forces simulated by a piezometric surface developed from the emergency spillway crest. Phreatic surfaces and piezometric surfaces may be developed using flow nets or Casagrande procedures. Select shear strength parameters according to Table 5-2.

IV. Steady Seepage With Seismic Forces. Analyze the downstream slope for the condition of steady seepage developed from the water surface at the principal spillway crest with a horizontal force equal to the effective weight multiplied by the earthquake coefficient applied at the failure surface (pseudo-static analysis). Uplift forces due to the reservoir at the emergency spillway level need not be included because of the very remote potential for the simultaneous occurrence of an earthquake and an emergency spillway storm event. A two or three dimensional computer analysis may be used instead of a pseudo-static analysis if desired. Select shear strength parameters according to Table 5-2.

V. Steady Seepage With and Without Seismic Forces. Determine the results of the analysis using either the total stress or effective stress shear strength parameters that are the most limiting (those resulting in the lowest factor of safety). Analyses are to include the use of various combinations of total stress parameters in certain zones and effective stress parameters in other zones. The combinations that may be most limiting can usually be narrowed to one or two by inspection and judgement without analyzing all possible combinations.

VI. Additional Guidance.

A. Effective stress shear strength parameters are generally the most limiting for any free draining soil.
B. Total stress shear strength parameters are generally most limiting for non-free draining foundation soils that exhibit positive pore pressure buildup during undrained shear testing.
C. Effective stress shear strength parameters are generally most limiting for non-free draining foundation soils that exhibit negative pore pressure buildup during undrained shear testing.
D. Analyses will generally have to be made using both total and effective stress shear parameters to determine the most limiting for non-free draining embankment soils that exhibit either small negative or positive pore pressures during undrained shear testing.
E. Effective stress shear strength parameters are generally most limiting for non-free draining embankment soils that exhibit highly negative pore pressures during undrained shear testing.
F. Total stress shear strength parameters are generally most limiting for non-free draining embankment soils that exhibit highly positive pore pressures during undrained shear testing.

TABLE 5-2

SLOPE STABILITY CRITERIA FOR DAMS

Design Condition	Primary Assumption	Remarks	Shear strength To Be Used[1,2]	Minimum Factor of Safety
I. End of construction (upstream or downstream slope)	Significant construction pore pressures expected in certain zones of the embankment or layers in the foundation.	1. Embankment containing impervious soils at water contents equal to or greater than optimum water content and/or 2. Saturated impervious foundation strata too thick to fully consolidate during construction.	UU	1.4 (1.3 is acceptable for embankments on strong foundations, i.e. the failure surface is located entirely in the embankment).
		Pervious embankment zones and/or foundation strata.	$\overline{CU}$ or CD	
II. Rapid drawdown (upstream slope)	Drawdown from the emergency spillway crest to the crest of the lowest gated or ungated outlet.	Failure surface can be confined to the embankment or extend into the foundation if low permeability soils are involved.	Use alternative 1 or 2 below - 1. Lowest shear strength from composite envelope of CU and $\overline{CU}$ or 2. Most limiting[3] of CU or $\overline{CU}$ or a combination of both. (For alternative 2, only use shear strength failure criteria of $\bar{\sigma}_1 \div \bar{\sigma}_3$ maximum for dilative soils tested with backpressure saturation techniques. This is to avoid reliance on high cohesion developed by negative pore pressure as a result of water tension).	1.2

NOTE: See bottom of next sheet for explanation of footnotes.

TABLE 5-2 (Cont'd.)
SLOPE STABILITY CRITERIA FOR DAMS

Design Condition	Primary Assumption	Remarks	Shear strength To Be Used[1,2]	Minimum Factor of Safety
III. Steady seepage without seismic forces (downstream slope)	1. Water surface at the principal spillway crest and phreatic surface fully developed through the embankment.	Failure surface confined to the embankment only.	$\overline{CU}$ or CD	1.5
	2. Piezometric surface for estimating uplift in saturated soil zones is determined with water level at the emergency spillway crest.	Failure surface extends into the foundation.	Most limiting[3] of CU or $\overline{CU}$ or a combination of both.	1.5
IV. Steady seepage with seismic forces (downstream slope)	1. Water surface at the crest of the principal spillway and phreatic surface fully developed through the embankment.	Failure plane may be confined to the embankment or may extend into the foundation.	Most limiting[3] of CU or $\overline{CU}$ or a combination of both.	1.1

[1] Use $\overline{CU}$ or CD shear strength for pervious soil zones within the foundation or embankment.

[2] Use infinite slope stability analysis whenever the C or $\overline{C}$ intercept strength is zero for soils that are tested to simulate low confining pressures. This situation exists for failure surfaces located near the embankment surface. Minimum factor of safety for the infinite slope stability analysis is 1.1.

[3] Most limiting is the combination resulting in the lowest factor of safety.

TABLE 5-3

NOMENCLATURE OF SHEAR STRENGTH TESTS

Test Condition	Parameters Obtained	SCS Description	Common Abbreviation
Unconsolidated, Undrained	Total 1/ Stress	UU	Q
Consolidated, Undrained	Total 1/ Stress	CU	R
Consolidated, Undrained with pore pressures measured	Effective 2/ Stress	$\overline{CU}$	R
Consolidated, Drained	Effective 2/ Stress	CD 3/	S

1/ Total stress shear strength parameters are determined when no drainage is allowed during the test using the external stresses imposed on the soil specimen. When used in a total stress analysis, the assumption made is that the pore pressure developed (negative or positive) during loading is the same in the laboratory as it is in the field.

2/ Effective stress parameters are determined from undrained tests by using the intergranular stresses (total stress minus the pore water stress) on the soil particles. When used in an effective stress analysis, any effects of pore pressure development (positive or negative) resulting from load application must be accounted for in the analysis.

3/ The CD tests allow drainage during load application. If the load is applied slow enough, complete drainage occurs so that no pore pressure develops and the shear strength parameters obtained should be the same as $\overline{CU}$ parameters.

Seepage

To the extent needed, an analysis is to be made of anticipated seepage rates and pressures through the embankment, foundation, abutments and reservoir perimeter (when storage is desired). Controls and treatment should be adequate to (1) accomplish the intended reservoir function, (2) provide a safely operating structure, and (3) prevent damage to downstream property.

Zoning

Embankment zoning can be used when needed to (1) obtain a stable structure with the most economical use of available materials, (2) control seepage in a safe manner, or (3) reduce to a minimum the uncertainties of material strengths and resultant stability.

Embankment zones should be a minimum of 10 feet wide except for filters and drains with specified and controlled gradation. Drains and filters should meet the requirements contained in Soil Mechanics Notes.

Soil materials which exhibit significant shrinkage, swell or dispersion are to be used only with extreme care. If possible, they should not be used for embankment construction. When there is no economical alternative to their use they are to be (1) treated to improve their performance, (2) placed in zones where effects will not be detrimental or (3) protected by use of filters and drains or self-healing transition zones.

Surface Protection

Embankment surfaces are to be protected against surface erosion. Protection may be vegetative, gravel, rock riprap, soil cement, structural, or similar treatment of durable quality and proven satisfactory performance.

Vegetative Protection

Vegetative protection may be used on surfaces where the following conditions can be met: (1) inundation of the surfaces is of such frequency that vegetative growth will not be inhibited, (2) vigorous growth can be sustained under average climatic conditions by normal maintenance without irrigation, and (3) stable protection can be designed according to the procedures in TR-56.

Structural Protection

Protection against wave erosion by riprap or other structural measures is to be provided as follows: (1) for dams where vegetation will not provide effective control, (2) for multiple purpose dams, and (3) for dams with fluctuating normal water levels.

Protection is to extend from the lowest drawdown elevation that presents an erosion hazard, to a few feet above the crest of the lowest ungated spillway. The upper limit is to be based on an analysis of anticipated wave height and run up.

Quality of riprap and other structural protection is to be consistent with the anticipated life of the dam and designed to be structurally stable.

PRINCIPAL SPILLWAYS

The structural design and detailing of principal spillways are to conform to the recommendations of National Engineering Handbook, Section 6, "Structural Design" and SCS standard drawings. All component parts of principal spillways except easily replaceable parts such as gates and trash racks are to be equally durable.

Capacity of Principal Spillway

The required capacity of the principal spillway depends on (1) purpose of the dam, (2) the amount of storage provided by the retarding pool, (3) the kind of emergency spillway, (4) stream channel capacity and stability downstream, (5) potential damage from prolonged storage in the retarding pool, (6) potential damage downstream from prolonged high outflow rates, (7) possibility of substantial runoff from two or more storms in the time required to empty the retarding pool, (8) limitations imposed by water rights or other legal requirements (9) environmental concerns, (10) planned or potential alterations of the channel downstream and (11) the necessity to pass base and flood flows during construction.

The principal spillway may be single-stage, having an ungated inlet at only one elevation, or multiple-stage, having inlets at two or more elevations. In the case of multiple-stage, spillways, the lower stage or stages usually perform the primary flood control function and the high stage has the capacity needed to prevent the emergency spillway from functioning more frequently than permissible.

The principal spillway capacity should be adequate to empty the retarding pool in 10 days or less. This requirement is considered to be met if 15 percent or less of the maximum volume of retarding storage remains after 10 days. Where low release rates are required to meet the objectives of the project, a longer period than 10 days may be needed. For these situations, additional storage is required to minimize the opportunity for increased frequency of emergency spillway flow due to recurring storms.

Compute the 10-day drawdown from the time the maximum water surface elevation is attained during the passage of the principal spillway hydrograph. The entire design inflow hydrograph including quick return flow, upstream releases, and outflow are to be considered in determining the evacuation time of the retarding storage. The inflow from storm runoff must be considered for the entire evacuation time.

For dams where more than 15 percent of the retarding storage volume remains after 10 days, the elevation of the crest of the emergency spillway will be raised. The raised crest elevation is determined by adding the remaining retarding storage volume to the initial retarding storage volume.

Elevation of Principal Spillways

Single Purpose Floodwater Retarding Dams

The crest of the principal spillway, or of the low stage inlet of a two-stage principal spillway is to be set at the sediment pool elevation. For dry dams, the elevation of the principal spillway inlet is to be placed as described above and provisions are to be made to drain the reservoir in a reasonable time and thus satisfy the functional or legal requirements of the dam.

Other Dams

When conservation storage is to be provided, the elevation of the crest of the lowest ungated inlet of the principal spillway is to be determined by the volume, area, or depth of water required for the planned purpose or purposes and the required sediment storage. The lowest crest may be the crest of the low-stage inlet, single-stage inlet, or an open spillway.

Routing of Principal Spillway Hydrographs

Reservoir flood routing used to proportion dams and associated spillways is to be based on the assumption that all sedimentation expected in the reservoir during its design life has occurred. The reservoir stage-storage curve used for routing should reflect the anticipated accumulation of sediment. The initial reservoir stage for principal spillway hydrograph routing is to be at the crest of the lowest ungated inlet or (if not subtracted from the stage-storage curve) the anticipated elevation of the sediment storage, whichever is higher, except as provided in items 1, 2, and 3.

1. For dams with significant base flow, principal spillway hydrograph routings must start not lower than the elevation of the water surface associated with the base flow. Significant base flow is average annual or seasonal flow that would produce at least 0.5 feet of head over the lowest principal spillway inlet immediately prior to a flood or occupy more than 10 percent of the floodwater storage capacity.

2. For dams with joint use storage capacity, when one of the uses is floodwater detention, routing of the principal spillway hydrograph may begin at the lowest anticipated elevation of the joint use pool in accordance with the operation plan.

3. Single purpose class (a) irrigation dams with gated outlets and earth or vegetated emergency spillways, which are located on ephemeral streams in areas where the average annual precipitation is less than 25 inches, may be considered to have discharged up to 70 percent of the storage, exculsive of sediment storage in determining the elevation to start routing.

Design of Principal Spillways

Hydraulics

The principal spillway is to be designed to carry the planned flow for expected head and tailwater conditions. TR-29, Design Note No. 8, NEH-5, the Engineering Field Manual for Conservation Practices and other appropriate references are to be used for hydraulic design.

Risers

Risers for drop inlet spillways are to be designed to maintain the reservoir pool level at or near the inlet crest elevation during low flow periods, to establish full pipe flow at as low a head over the crest as practical, and to operate without excessive surging, noise, vibration, or vortex action at any reservoir stage. This requires the riser to have a larger cross sectional area than the conduit. Standard risers have an inside width equal to the width (diameter) D, of the conduit and an inside length equal to three times the width (diameter) of the conduit (D x 3D cross section).

Risers are to be designed to exclude trash too large to pass freely through the spillways, including the outlet structure and to facilitate the passage of smaller trash. Standard D x 3D risers tend to line up longer pieces of trash and facilitate their passage into and through the conduit. Covered risers with standard skirted or baffle inlets should be used in most cases because they are most effective in excluding trash without becoming clogged. Skirted inlets, having a cover with skirts extending below the weir crest elevation, are applicable where backfill or settlement levels are to be at least two times the conduit width (diameter) below the crest. Baffle inlets are applicable for risers that are to be backfilled to the crest elevation or where sediment is expected to build up to the crest elevation.

Risers are to be designed structurally to withstand all water, earth, ice, and earthquake loads to which they may be subjected. Articulation is to be provided to allow movement of the riser with respect to the conduit.

Risers with low-stage inlets at or near the bottom are to be provided with concrete aprons to prevent erosion of soil and undermining of the riser footing by high velocity flow approaching the inlet.

Standard risers are to be used where applicable for class (a) dams with an effective height of more than 35 feet and for all class (b) and class (c) dams. Prefabricated pipe risers are permissible, where hydraulically and structurally adequate, for class (a) dams not more than 35 feet in effective height. The riser pipe is to be of the same material as the conduit and at least one standard pipe size larger then the conduit pipe.

Special riser designs are required for spillways having maximum conduit velocities more then 30 feet per second and for spillways having conduits larger then 48 inches in width (diameter). Generally, these should be similar to standard risers but a special elbow and transition is required at the junction of the riser and conduit and special design of the inlet may be necessary. Hydraulic model testing should be considered if the maximum total head on the spillway is more than 75 feet or the conduit velocity exceeds 50 feet per second.

Conduit

The conduit should be straight in alignment when viewed in plan. Changes from straight alignment if required, are to be accomplished by watertight angle changes at joints or by special elbows having a radius equal to or greater than the diameter or width of the conduit. Thrust blocks of adequate strength are to be provided if special pipe elbows are used. They are to be designed to distribute the thrust due to change in direction for the maximum possible discharge. Drop inlet conduits are to be installed with enough slope to insure free drainage to the outlet of all parts of the conduit (including camber) at the time of construction and under the maximum anticipated settlement.

All conduits under earth embankments are to support the external loads with an adequate factor of safety. They are to withstand the internal hydraulic pressures without leakage under full external load and settlement. They are to convey water at the design velocity without damage to the interior surface of the conduit.

Principal spillway conduits under earth dams may be designed to support fill heights greater than the original constructed height if there is a reasonable possibility that the embankment height may be raised later to incorporate additional storage for some approved beneficial use.

Rigid principal spillway conduits are to be designed as positive projecting conduits in accordance with the principles and procedures given in TR-5.

Principal spillway conduits are to be of reinforced concrete pressure pipe or cast-in-place reinforced concrete, unless corrugated steel or welded steel pipe is used in accordance with subsection 2, which follows.

Cast-in-place rectangular reinforced concrete conduits are to be designed in accordance with principles and procedures in TR-42, TR-45 or other appropriate design aids.

For Reinforced Concrete Water Pipe. - - Steel Cylinder Type, Prestressed, meeting specification AWWA C301, the 3-edge bearing strength at the first 0.001-inch crack is to be used with a safety factor of at least one.

For Reinforced Concrete Water Pipe. - - Steel Cylinder Type, Not Prestressed, meeting specification AWWA C300; for Reinforced Concrete Water Pipe - Noncylinder Type, Not Prestressed, meeting specification AWWA C302, and for other types of reinforced concrete pipe, the 3-edge bearing strength at the first 0.01-inch crack is to be used with a safety factor of at least 1.33.

Elliptical or other systems of reinforcement requiring special orientation of pipe sections are not permitted in spillway conduits.

Reinforced concrete pipe is to be designed to support at least 12 feet of earth fill above the pipe at all points along the conduit.

1. Reinforced Concrete Pipe - (A) Minimum Inside Diameters on Yielding Foundations, Class (a) dams: The minimum diameter of the principal spillway conduit is to be 30 inches, unless a joint extension safety margin of at least 1.5 inches is used, in which case the minimum diameter is to be 18 inches for maximum fill heights up to 50 feet at the centerline of the dam and 24 inches for greater fill heights.

   Class (b) dams: The minimum diameter of the principal spillway conduit is to be 30 inches, unless a joint extension safety margin of 1.5 inches is used, in which case the minimum diameter is to be 24 inches.

   Class (c) dams: The minimum diameter of the principal spillway conduit is to be 30 inches.

   (B) Minimum Inside Diameters on Non-Yielding Foundations: The minimum diameter of the principal spillway conduit for class (a) dams is to be 18 inches for heights up to 50 feet at the centerline of the dam and 24 inches for heights greater than 50 feet, and 24 inches for all class (b) and (c) dams. The conduit and cradle or bedding are to rest directly on firm bedrock thick enough so that there is essentially no foundation consolidation under the conduit. Under these conditions the cradle or bedding under the conduit need not be articulated.

2. Corrugated Steel Pipe or Welded Steel Pipe - Principal spillways of corrugated steel or welded steel pipe may be used for single purpose class (a) dams with the product of storage times effective height of dam less than 10,000. While installation costs of steel pipes may be less, concrete may compare favorably with steel when replacement costs and associated problems are considered.

   In each case the following limitations apply:

   a. Diameter of pipe not less than 18 inches.
   b. Height of fill over the pipe not more than 25 feet.
   c. Provision for replacement if the materials will not last for the design life of the structure.
   d. Pipe structurally strong enough to withstand outside loads and hydraulic pressure.
   e. Pipe Watertight.

   Corrugated steel pipe is to be close riveted, asbestos treated, and asphalt coated, with watertight connecting bands. The minimum gage is to be that designed for 35 feet of fill over the pipe.

   Welded steel pipe conduits are to be structurally designed as rigid pipe. A joint extension safety margin of 1.5 inches is to be provided for conduits on yielding foundation. Welded steel pipe is to be protected by a Class A exterior coating as defined in the specifications guide for steel irrigation pipeline, Engineering Practice Standard 432-F (NEH-2) or by an exterior coating of coal tar-epoxy paint conforming to Paint System I, Construction Specification 82 (NEH 20).

Joints between lengths of corrugated steel or welded steel pipe, other than welded joints, are to be electrically bridged on the outside of the pipe with insulated copper wire, #6 AWG or larger, securely attached to the uncoated pipe metal at both sides of the joint. This requirement applies whether or not the cathodic protection is completed by the installation of anodes, etc. The wire should have a tough, waterproof insulation designed for direct burial, with a rating of at least 600 volts. Bare wire and exposed pipe metal at the points of connection are to be thoroughly coated with a coating equivalent to the original pipe coating to prevent the entry of moisture.

Soil investigations for resistivity and pH of the subgrade and backfill materials to be adjacent to the conduit are to be made. The resistivity measurements are to be made on saturated samples.

Cathodic protection is to be provided for welded steel pipe conduits according to the criteria in Engineering Practice Standard 432-F (NEH-2) and 432-F specifications guide.

Cathodic protection meeting the above requirements is to be provided for corrugated steel pipe in soil whose resistivity in a saturated condition is less than 4000 ohms-cm or whose pH is lower than 5.0.

If cathodic protection is not required according to the above criteria and anodes are not installed during construction of the dam, pipe-to-soil potentials are to be measured within the first 2 years after construction or after the water level has stabilized and when the soil around the conduit is estimated to be at its normal post-construction moisture content. Cathodic protection is to be installed at this time if such measurements indicate it is needed.

Joints

Conduit joints are to be designed and constructed to remain watertight under maximum anticipated hydrostatic head and maximum probable joint opening as computed from Standard Drawing ES-146 and related procedures of TR 18, including the effects of joint rotation and the required margin of safety. The required joint extensibility is equal to the unit horizontal strain in the earth adjacent to the conduit multiplied by the length (in inches) of the section of conduit between joints plus the extension (in inches) due to calculated joint rotation plus a margin of safety.

A margin of safety of not less than 0.5 inch shall be used. The required joint extensibility plus the maximum permissible joint gap equals the required joint length. The required joint extensibility depends on the maximum potential foundation consolidation under the spillway barrel. For class (b) and (c) dams, the consolidation is to be estimated from adequate foundation borings and samples, soil mechanics laboratory tests, and engineering analysis. For class (a) dams where undisturbed foundation samples are not taken for other purposes, approximate procedures based on soil classification and experience may be used for estimating foundation consolidation.

Only joints incorporating a round rubber gasket set in a positive groove which will prevent its displacement from either internal or external pressure under the required joint extensibility are to be used on precast concrete pipe conduits. Concrete pipe must have steel joint rings providing rubber to steel contact in the joint.

Articulation of the conduit (freedom for required rotation) is to be provided at each joint in the conduit, at the junction of the conduit with the riser and any outlet structure. Concrete bedding for pipe conduits need not be articulated. Cradles are to be articulated if on yielding foundations. Welded steel pipe conduits need not be articulated if the pipe and bedding rest directly on firm bedrock.

Piping and Seepage Control - Use a filter and drainage diaphragm around any structure that extends through the embankment to the downstream slope. Design the diaphragm with single or multizones to meet the requirements of Soil Mechanics Note No. 1 except that the maximum D_{15} shall be 0.35 mm for filters to protect base soils with PI greater than 15.

Locate the diaphragm aligned approximately parallel to the centerline of the dam or approximately perpendicular to the direction of seepage flow. Extend the diaphragm horizontally and vertically into the adjacent embankment and foundation to intercept potential cracks, poorly compacted soil zones or other discontinuities associated with the structure or its installation.

Design the diaphragms to extend the following minimum distances from the surface of rigid conduits:

1. Horizontally and vertically upward 3 times the outside diameter of circular conduits or the vertical dimension of rectangular box conduits except that:

    a. the vertical extension need be no higher than the maximum potential reservoir water level and,

    b. the horizontal extension need be no further than 5 feet beyond the sides and slopes of any excavation made to install the conduit.

2. Vertically downward:

    a. for conduit settlement ratios (δ) of 0.7 and greater (reference SCS Technical Release No. 5), the greater of (1) 2 feet or (2) 1 foot beyond the bottom of the trench excavation made to install the conduit. Terminate the diaphragm at the surface of bedrock when it occurs within this distance. Additional control of general seepage through an upper zone of weathered bedrock may be needed.

b. 1.5 times the outside diameter of circular conduits or the outside vertical dimension of box conduits for conduit settlement ratios (δ) less than 0.7.

Design the diaphragms to extend in all directions a minimum of 2 times the outside diameter from the surface of flexible conduits, except that the diaphragm need not extend beyond the limits in 1a and 1b above nor beyond a bedrock surface beneath the conduit.

Provide minimum diaphragm thickness of 3 feet and a minimum thickness of 1 foot for any zone of a multizone system. Use larger thickness when needed for (1) capacity, (2) tieing into embankment or foundation drainage systems, (3) accommodating construction methods, or (4) other reasons.

For homogeneous dams, locate the diaphragm in the downstream section of the dam such that it is:

1. Downstream of the cutoff trench,

2. Downstream of the centerline of the dam when no cutoff trench is used, and

3. Upstream of a point where the embankment cover (upstream face of the diaphragm to the downstream face of the dam) is at least one-half of the difference in elevation between the top of the diaphragm and the maximum potential reservoir water level.

For zoned embankments, locate the diaphragm downstream of the core zone and/or cutoff trench, maintaining the minimum cover as indicated for homogeneous dams. When the downstream shell is more pervious than the diaphragm material, locate the diaphragm at the downstream face of the core zone.

It is good practice to tie these diaphragms into the other drainage systems in the embankment or foundation. Foundation trench drains and/or embankment chimney drains that meet the minimum size and location limits are sufficient and no separate diaphragm is needed.

Design the minimum capacity of outlets for diaphragms not connected to other drains by assuming the coefficient of permeability (k) in the zone upstream of the diaphragm is 100 times the coefficient of permeability in the compacted embankment material. Assume this zone has a cross-sectional area equal to the diaphragm area and the seepage path distance equal to that from the embankment upstream toe to the diaphragm. This higher permeability simulates a sealed filter face at the diaphragm with partially filled cracks and openings in the upstream zone.

For channels, chutes or other open structures, seepage and piping control can be accomplished in conjunction with drainage for reduction of uplift and water loads. The drainage, properly designed to filter the base soils, is to intercept areas of potential cracking caused by shrinkage, differential settlement or heave and frost action. These structures usually require the use of footings, keywalls and counterforts and drainage is properly located immediately downstream of these features. This drainage when properly designed can control piping and provide significant economies due to the effect on soil loads, uplift pressures, overturning forces and sliding stability.

Outlets

The choice of outlet is to be based on a careful consideration of all site and flow conditions that may affect operation and energy dissipation.

1. Cantilever outlet and plunge pools may be installed where their use:

    a. Does not create a piping hazard in the foundation of the structure.

    b. Is compatible with other conditions at the site.

    Plunge pools are to be designed to dissipate the energy and be stable. Unless the pool is to be in bedrock or very erosion resistant materials, riprap will be necessary to insure stability. Design Note 6, entitled, "Armored Scour Hole for Cantilever Outlet", is to be used for design.

    Cantilever outlets are to be supported on bents or piers and are to extend a minimum of 8 feet beyond the bents or piers. The bents are to be located downstream from the intersection of the downstream slope of the earth embankment with the grade line of the channel below the dam. They are to extend below the lowest elevation anticipated in the plunge pool. The invert of the cantilever outlet is to be at least 1 foot about the tailwater elevation at maximum discharge.

2. SAF basins may be used when there is adequate control of tailwater. Use TR-54 for structural design and NEH-14 for hydraulic design.

3. Impact basins may be used when positive measures are taken to prevent large debris from entering the conduit. TR-49 is to be used for hydraulic design.

Trash Racks

Trash racks are to be designed to provide positive protection against clogging of the spillway under any operating level. The average velocity of flow through a clean trash rack is not to exceed 2.5 feet per second under the full range of stage and discharge. Velocity is to be computed on the basis of the net area of opening through the rack.

If a reservoir outlet with a trash rack or a ported concrete riser is used to keep the sediment pool drained the trash rack or riser is to extend above the anticipated sediment elevation at the riser to provide for full design flow through the outlet during the design life of the dam. The velocity through the net area of the trash rack above the maximum sediment elevation must not exceed 2 feet per second when the water surface in the reservoir is 5 feet above the top of the trash rack or riser inlet.

Antivortex Device

All closed conduit spillways designed for pressure flow are to have adequate antivortex devices.

High Sulfate Areas

Under certain conditions concrete is susceptible to deterioration from sulfate ions, especially those derived from sodium and magnesium sulfates. In areas where experience or soil tests indicate the potential for problems, the following table will be used for design purposes:

Sulfate Concentration [1/] (parts per million)	Hazard	Corrective Measures
0 - 150	Low	None
150 - 1,000	Moderate	Use Type II Cement. (ASTM C-150). Adjust mix to protect against sulfate action.
1,000 - 2,000	High	Use Type V Cement (ASTM C-150). Adjust mix to protect against sulfate action. Use soils in contact with concrete surfaces that are low in sulfates.
2,000 - up		Do not use concrete materials unless measures are taken to protect concrete surfaces from sulfates. Product manufacturers should be consulted.

[1/] Sulfate concentration is for soil water at the concrete surface.

EMERGENCY SPILLWAYS

Emergency spillways are provided to convey excess water through, over or around a dam. They are usually open channels excavated in natural earth, earthfill, rock or constructed of reinforced concrete.

Closed Type Spillways

An open channel emergency spillway is to be provided for each dam except as provided below:

1. Class (a) dams with a product of storage times the effective height of the dam of less than 10,000 - A closed conduit principal spillway having a conduit with a cross-sectional area of 12 square feet or more, an inlet which will not clog, an elbow designed to facilitate the passage of trash, and large enough to pass the routed freeboard hydrograph is the minimum acceptable design without an open channel emergency spillway. The minimum capacity will be taken from Figure 7-1.

2. Dams with drainage areas of 10 square miles or less (except those covered by item 1) - A closed conduit principal spillway having a conduit with a cross-sectional area of 20 square feet or more, an inlet which will not clog, an elbow designed to facilitate the passage of trash and large enough to pass the routed freeboard hydrograph peak discharge without overtopping the dam is the minimum acceptable design without an open channel emergency spillway. In no case is the capacity of the spillway provided to be less than that determined from Figure 7-1.

3. Dams with drainage areas greater than 10 square miles (except those covered by item 1) - A closed type primary emergency spillway may be used in lieu of an open channel emergency spillway. Drop inlet spillways with a standard two-way covered top inlet are to have a minimum unobstructed cross-sectional area of each opening of the conduit of 40 square feet. All other closed type primary emergency spillways are to have a minimum unobstructed cross-sectional area of each opening of 80 square feet. The ratio of width to height in both cases is to be between 0.75 and 1.33. The spillways must be large enough to pass the routed freeboard hydrograph peak discharge without overtopping the dam. In no case is the capacity of the primary emergency spillway provided to be less than that determined from Figure 7-1.

Spillway Requirements

Capacity of Emergency Spillways

Emergency spillways are to be proportioned so they will pass the emergency spillway hydrograph at the safe velocity determined for the site. They are to have sufficient capacity to pass the freeboard hydrograph with the water surface in the reservoir at or below the elevation of the design top of the dam. In no case is the capacity of the emergency spillway to be less than that determined from Figure 7-1. The minimum difference in elevation between the crest of the emergency spillway and the settled top of the dam is three feet.

State law may establish minimum capacity or depth greater than those given above.

Elevation of the Crest of the Emergency Spillway
Table 2-2, Page 2-6 gives the maximum allowable frequency of use of earth and vegetated emergency spillways. The minimum retarding storage volume and the associated principal spillway discharge are to be such that: (1) the discharge through the emergency spillway will not occur during the routing of the principal spillway hydrograph, and (2) the 10-day drawdown requirement is met, or the crest elevation of the emergency spillway is raised as noted on page 6-1, Capacity of Principal Spillway.

For earth spillways, it refers to sites where peak flows of short duration may be expected, and where erosion resistant soils and moderate slopes exist. When vegetated spillways are used, the sites must have these same characteristics, and in addition, conditions must be such that vigorous vegetation can be maintained without irrigation. When conditions are less favorable, emergency spillways are to be designed for less frequent use. This may be done by (1) raising the crest elevation, (2) increasing the capacity of the principal spillway, (3) adding a structural primary emergency spillway or (4) a combination of the above.

The maintenance required for the emergency spillway will be increased as the frequency and duration of flow increase. Good design requires balancing the spillway maintenance cost against the increased cost of modifying the other elements of the dam to reduce the flow frequency.

Emergency Spillway Routings
The emergency spillway and the freeboard hydrographs are to be routed through the reservoir starting with the water surface at the elevation of the lowest ungated principal spillway inlet, the anticipated elevation of the sediment storage, the elevation of the water surface associated with significant base flow or the pool elevation after 10 days of drawdown from the maximum stage attained when routing the principal spillway hydrograph, whichever is higher, except as provided in items 1 and 2.

1. Dams with gated spillways and joint use storage capacity - Emergency spillway and freeboard hydrograph routings are to be started at or above the elevation of the lowest ungated outlet or at the elevation of the water surface associated with the average annual base flow, whichever is higher.

2. Single purpose class (a) irrigation dams - Emergency spillway and freeboard hydrograph routings are to be started at or above the water surface elevation of the irrigation storage.

Hydraulic Design
The relationship between the water surface elevation in the reservoir and the discharge through the emergency spillway is to be evaluated by computing the head losses in the inlet channel upstream of the control section or, if a control section is not used, by computing the water surface profile through the full length of the spillway. Bernoulli's equation and Manning's formula are to be used to evaluate friction losses, compute water surface profiles and determine velocities. Policy on the selection of "n" values is given in the discussion of the various types of emergency spillways.

Structural Stability
The spillway is to be investigated, analyzed, designed and constructed adequately to establish and maintain stability during the passage of design flows without blockage or breaching. Excavated open cut spillways are to have cut and fill slopes in earth and rock which are stable against sliding. Cut slope stability is to be evaluated for the long term weathered, natural moisture condition and for adverse moisture conditions associated with rapid drawdown from the emergency spillway design discharge.

Vegetated and Earth Emergency Spillways

Vegetated and earth emergency spillways are open channels and usually consist of an inlet channel, a control section, and an exit channel. Subcritical flow exists in the inlet channel and the flow is usually supercritical in the exit channel.

Vegetated emergency spillways are usually trapezoidal in cross-section and are protected from damaging erosion by a grass cover. They are adapted to sites where a vigorous grass growth can be sustained by normal maintenance without irrigation.

Earth spillways are used in those areas where vegetative growth cannot be maintained. They are similar to vegetated spillways but are designed for lower permissible velocities and less frequent use. Normally, they will require more maintenance after a flow occurs.

Earth and vegetated emergency spillways are designed on the basis that some erosion or scour may be permissible if its occurrence is infrequent, if maintenance facilities are provided, and the spillway will not breach during passage of the freeboard storm.

TR 2 outlines layout procedures and TR 39 contains hydraulic data to be used in the design of vegetated or earth emergency spillways. A Manning's "n" of 0.04 is to be used for determining the velocity and capacity in vegetated spillways. Design velocities in earth spillways will be based on an "n" value of 0.02 but the capacity of earth spillways will be based on an appraisal of the roughness condition at the site.

Layout
Emergency spillways are to be located away from the dam whenever possible. Topographic saddles generally make good sites. The layout and profile of vegetated or earth spillways is to provide a maximum bulk of material to

provide safety against breaching of the spillway during the passage of the freeboard hydrograph.

The inlet channel is to be level for a minimum distance of 30 feet upstream from the control section. This level part of the inlet channel is to be the same width as the exit channel, and its centerline is to be straight and coincident with the centerline of the exit channel. A curved centerline is permissible in the inlet channel upstream from the level section, but it must be tangent to the centerline of the level section.

The centerline of the exit channel is to be straight and perpendicular to the control section for a distance equal to a least one-half of the maximum base width of the dam. Curvature may be introduced below this point if it is certain that the flowing water will not impinge on the dam should the channel fail at the curve.

The exit channel is to be as long as reasonably practical, but when a control section is used the grade is to be sufficient to insure supercritical flow for all discharges equal to or greater than 25 percent of the maximum discharge through the emergency spillway during the passage of the freeboard hydrograph. However, the slope in the exit channel need not exceed 4 percent (s = 0.04 ft/ft) to meet this requirement.

The spillway discharge may be released by an exit channel at a point some distance above the stable grade of the natural stream channel. When this is done, the discharge is allowed to spread naturally over the existing topography and find its way to the channel downstream. Since this layout involves no consideration of velocities beyond the exit channel there may be considerable erosion on those reaches not designed on a permissible velocity basis.

Another approach is to construct a channel from the end of the exit channel to stable grade below. In this case, the lower constructed channel may be designed with higher velocities than are permissible in the exit channel proper. This assumes that erosion in the lower, well defined, improved channel may be less damaging than that occurring where the flow is permitted to meander over the natural relief before reaching stable grade.

In both layouts, erosion will occur wherever the permissible velocities are exceeded and maintenance will be required to protect the integrity of the spillway. Land rights are to be considered in making the decision on how to handle the return flow to the natural or constructed stream channel downstream from the dam.

Stability Design of Earth and Vegetated Earth Spillways

Limitations during routing of the emergency spillway hydrograph - The maximum velocity limitations given below for vegetated or earth emergency spillways apply to the exit channel. They must not be exceeded in the reach where an exit channel failure might cause the flow to impinge on the toe of the dam. The velocity limitations are based on the maximum discharge in routing the emergency spillway hydrograph and the assumption that uniform flow conditions exist in the exit channel.

Vegetated Emergency Spillways -- When the anticipated average use of a vegetated emergency spillway is more frequent than once in 50 years, the maximum permissible velocity will be in accordance with the values given in SCS Technical Publication 61, (SCS-TP-61) "Handbook of Channel Design for Soil and Water Conservation." The values may be increased 10 percent when the anticipated average use is once in 50 years or 25 percent when the anticipated average use is once in 100 years. Table 7-1 summarizes the recommendations of SCS TP-61. Values for grasses or mixtures not shown in the table are to be determined by comparing their characteristics with those shown in the table. Where special studies or investigations have been made to determine the permissible velocity for a species, soil, and site, these values may be used in lieu of those shown in Table 7-1.

Earth Emergency Spillways -- The permissible velocity in earth spillways will be chosen after consideration of the soils involved, the frequency of use of the spillway and other pertinent factors. Table 7-2 is taken form Fortier and Scobey's study, "Permissible Canal Velocities After Aging," and may be helpful in determining this velocity. The values given for noncohesive soils should not be exceeded unless special studies have demonstrated that higher velocities are permissible. The table is not strictly applicable for cohesive soils since it applies to canal beds that are seasoned (perhaps permitting higher velocities) and subject to continuous flow and for conditions where erosion damage cannot be tolerated (requiring lower velocities).

Ramp Spillways -- Ramp spillways may be used only when a reasonable alternative solution is not practicable. This type of spillway is not generally favored by the engineering profession and when accepted requires a very conservative design. It may be used only on class (a) dams in humid areas where soils and rainfall are such that a vigorous growth of grass can be maintained without irrigation. The allowable frequency of use is 100 years, the dams must not be in series and the height-storage product cannot exceed 30,000. Slope of the exit channel is to be uniform and may not exceed 10 percent (0.10 ft/ft). It is to be located to discharge for its full width onto an approximately level flood plain and with the anticipated or constructed plunge pool below the principal spillway not closer than 50 feet from the outside toe of the exterior confining levee. Easily eroded soils and soils subject to excessive shrink or swell are not to be used within 3 feet of the wetted perimeter of the ramp spillway.

Limitations during routing of the freeboard hydrograph - Emergency spillway stability is to be based on gully erosion and formation principles. The design is to be such that the spillway will not breach during passage of the freeboard storm. The procedures in TR-52 are to be used for design of all spillways, including ramp spillways, regardless of drainage area. erosion

Special Precautions for Class (c) dams -- Special consideration is to be given to the layout of spillways on Class (c) dams to assure the spillway will not breach under the most exteme conditions of flow. The length of the exit channel is to be increased to the maximum extent possible so that the area most susceptable to erosion is at a considerable distance from the dam. Within the limitations of the site, the profile of the spillway is to be such that a maximum bulk of material is provided.

It is preferable that the flow be confined without the use of levees, but when they are necessary they are to be high enough to contain the peak flow of the routed freeboard hydrograph. Levees are to be constructed of erosion resistant materials and compacted to the degree necessary to develop this resistance. They are to have a top width not less than 12 feet and, if not protected with riprap, have side slopes not steeper than 3 horizontal to 1 vertical on the side where water flows. When constructed on a foundation subject to piping or undermining, they are to be keyed into the foundation with a compacted core having a bottom width not less than the top width of the levee and of sufficient depth to reach sound material, or to a depth equal to the height of the levee, whichever is less.

Crest control structures are to be provided to maintain a uniform surface where the soils are highly erodible from on-site runoff and very low flows through the spillway. The effective bulk length may be increased by installing barriers that will effectively stop a gully advancing through the spillway. Consideration is to be given to the reduction of the duration and volume of flow through the emergency spillway by raising the elevation of the crest of the emergency spillway, thereby increasing the volume of storage in the retarding pool. An alternate or complementary procedure is to increase the capacity of the principal spillway by means of a two stage inlet of sufficient size to carry an appreciable amount of the outflow hydrograph.

Rock Emergency Spillways

Some of the principles used for the layout of earth emergency spillways are applicable to rock emergency spillways. Allowable average frequency of use and permissible velocities must be ascertained for the specific site based on a knowledge of the hardness, condition, durability, attitude, weathering characteristics and structure of the rock formation. An individual appraisal is necessary to determine the proper roughness coefficient, "n". The design is to be such that the emergency spillway will not breach during passage of the freeboard storm.

Structural Emergency Spillways

Reinforced concrete structural spillways are to be designed so that the passage of the freeboard hydrograph will not cause serious damage to the embankment or the structures themselves. The configuration of a structural spillway is to be compatible with the foundation conditions at the site, the channel stability downstream from the spillway, the possible range of tailwater depth, and the proximity of the spillway to the embankment. The inlet portion of a chute spillway is to consist of a straight inlet, a box drop inlet, an ogee crest, or other appropriate hydraulic structure, which will produce critical flow at the crest and result in a determinate stage-discharge relationship.

The hydraulic design of structural emergency spillways is to be in accordance with the principles set forth in NEH-5, Hydraulics; NEH-11, Drop Spillways; NEH-14, Chute Spillways; and U.S. Department of Interior, Bureau of Reclamation publications; or based on model studies; with consideration given to the effects of air entrainment by water traveling at supercritical velocities.

The design discharge for hydraulic proportioning of structural emergency spillway is to be not less than two-thirds of the planned structure capacity during passage of the routed freeboard hydrograph, except that all headwalls and sidewalls are to be designed to prevent overtopping during passage of the full maximum freeboard discharge. When the magnitude of a structural emergency spillway exceeds that of structures commonly designed by SCS, model studies or other special studies are to be made.

The outlet section of concrete chute spillways is to consist of a hydraulic jump basin, such as a SAF, deflector bucket, roller bucket, or other appropriate hydraulic structure which will dissipate the energy of the high velocity discharge.

Structural emergency spillways are to be designed to withstand lateral earth pressures, uplift, seepage and other hydrostatic and hydrodynamic pressures. They are to be structurally designed for the full maximum freeboard discharge with uplift and sliding safety factors of not less than 1.0 and in accordance with the principles set forth in NEH-6, Structural Design; NEH-11, Drop Spillways and NEH-14, Chute Spillways, utilizing TR-50, TR-54 and other appropriate and available design working aids.

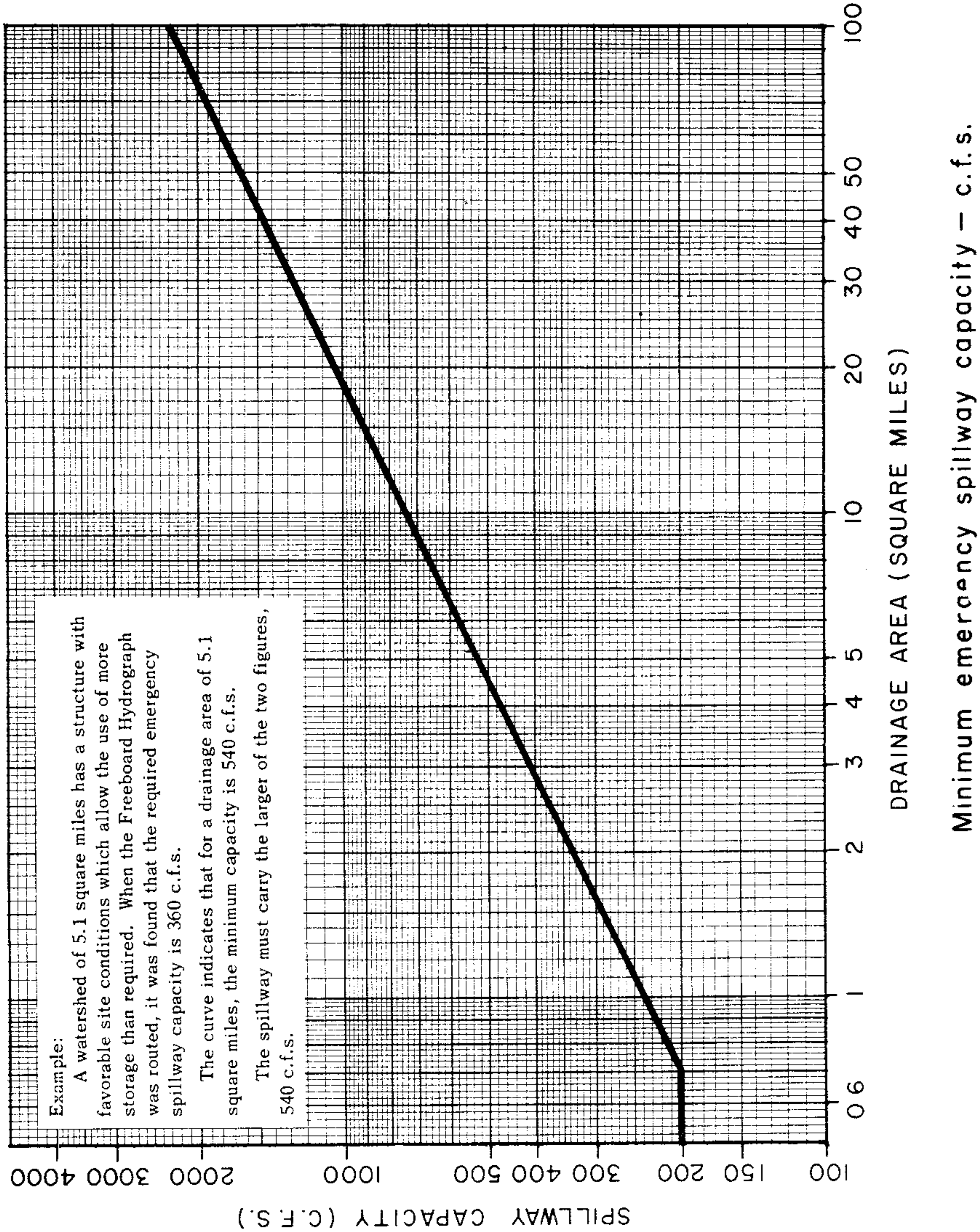

Minimum emergency spillway capacity — c.f.s.
DRAINAGE AREA (SQUARE MILES)
0.6
1
2
3
4
5
10
20
30
40
50
100
SPILLWAY CAPACITY (C.F.S.)
100
150
200
300
400
500
1000
2000
3000
4000
Example:
A watershed of 5.1 square miles has a structure with favorable site conditions which allow the use of more storage than required. When the Freeboard Hydrograph was routed, it was found that the required emergency spillway capacity is 360 c.f.s.
The curve indicates that for a drainage area of 5.1 square miles, the minimum capacity is 540 c.f.s.
The spillway must carry the larger of the two figures, 540 c.f.s.

Figure 7-1

TABLE 7-1

PERMISSIBLE VELOCITIES FOR VEGETATED SPILLWAYS 1/

	Permissible velocity 2/ feet per second			
	Erosion resistant 3/ soils		Easily erodible 3/ soils	
	Slope of exit channel Percent		Slope of exit channel Percent	
	0 to 5	5 thru 10	0 to 5	5 thru 10
Bermudagrass Bahiagrass	8	7	6	5
Buffalograss Kentucky bluegrass Smooth bromegrass Tall fescue Reed Canarygrass	7	6	5	4
Sod forming grass-legume mixtures	5	4	4	3
Lespedeza sericea Weeping lovegrass Yellow bluestem Native grass mixtures	3.5	3.5	2.5	2.5

1/ SCS-TP-61

2/ Increase values 10 percent when the anticipated average use of the spillway is not more frequent than once in 50 years or 25 percent when the anticipated average use is not more frequent than once in 100 years.

3/ As defined in TR-52

TABLE 7-2

PERMISSIBLE CANAL VELOCITIES AFTER AGING 1/

Original material excavated	Feet/second
Fine sand, non-colloidal	1.50 2/
Sandy loam, non-colloidal	1.75
Silt loam, non-colloidal	2.00
Alluvial silts, non-colloidal	2.00
Ordinary firm loam	2.50
Volcanic ash	2.50
Fine gravel	2.50
Stiff clay, very colloidal	3.75
Graded, loam to cobbles, non-colloidal	3.75
Alluvial silts, colloidal	3.75
Graded, silt to cobbles, colloidal	4.00
Coarse gravel, non-colloidal	4.00
Cobbles and shingles	5.00
Shales and hardpans	6.00

1/ Recommended in 1926 by Special committee on Irrigation Research, American Soiety of Civil Engineers.

2/ Values shown apply to clear water, no detritus.

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