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Earth Dams and Reservoirs

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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. NRCS plans, designs, and constructs complex dams under widely varying conditions. It is essential that these dams be constructed with uniform criteria to ensure consistent performance. As new experience, materials, and knowledge become available, this document will be revised.

This TR applies to all Low Hazard Class dams with a product of storage times the effective height of the dam of 3,000 acre-feet^2 or more, those more than 35 feet in effective height, and all Significant and High Hazard Class 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 alternate 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 ensure satisfactory performance.

This edition of the TR incorporates all previously issued revisions and amendments, as well as significant changes in chapters 2 (24-hour Design Storms), 5 (Slope Stability Analysis), and 7 (New Earth Spillway Breach Model) that were widely distributed and reviewed. This edition also makes numerous editorial corrections, including SCS to NRCS; emergency spillway to auxiliary spillway; and Class a, b, c to Low, Significant, High Hazard Class, respectively.

Contents:

Earth Dams and Reservoirs

General

Dam classification

In determining dam classification, a number of factors must be considered. Consideration must 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 must 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-NEM (*National Engineering Manual*), Part 520, Subpart C, DAMS.

Classes of dams

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

- Low Hazard Class—dams located in rural or agricultural areas where failure may damage farm buildings, agricultural land, or township and country roads.
- Significant Hazard Class—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.
- High Hazard Class—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 will 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, regardless of the technique used to analyze the downstream inundation area, is:

1. For depth of water at the dam at the time of failure where $H_w \geq 103$ ft

$$Q_{max} = (65)H_w^{1.85}$$

2. For depth of water at the dam at the time of failure where $H_w < 103$ ft

$$Q_{max} = (1{,}100)B_r^{1.35} \qquad \text{where } B_r = \frac{(V_s)(H_w)}{A}$$

but not less than $Q_{max} = (3.2)H_w^{2.5}$ nor more than $Q_{max} = (65)H_w^{1.85}$

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 1 and 2 above with,

$$Q_{max} = (0.416)(L)H_w^{1.5}$$

where:

Q_{max} = the peak breach discharge, ft^3/sec

B_r = breach factor, acre

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

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, ft

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, ft^2

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

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

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 must be either relocated away from the site or 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 must be relocated or raised as necessary to prevent damage or hazard to the public.

Every reasonable effort should 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 permitted to remain under an earth dam embankment.

Conduits permitted to remain under any part of the embankment below the crest of the auxiliary spillway are to be:

- provided with seepage control against potential piping;
- properly articulated on all yielding foundations;
- encased in concrete or otherwise treated to ensure durability and strength equal to that of the principal spillway; and
- made watertight against leaking either into or out of the pipe.

Enclosure of the conduit cable or pipeline within another conduit that meets the requirements of this section and is positively sealed at the upstream end to prevent seepage into the enclosing conduit is acceptable. Such an enclosing conduit must 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 must 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 must 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.

- There is reasonable assurance that water will be available to meet objectives.
- Flood protection objectives of the project are satisfied.
- Spillway conditions are such that the dam will perform safely.

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

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

Provisions must be made for operation of the joint use pool to ensure functioning of the dam as designed. These must include a competent operating and maintaining organization and a specific operation and maintenance plan. These requirements must 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 must 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 must be shaped to blend with the surrounding topography. These areas must be revegetated with herbaceous and woody plants to visually fit the existing surrounding vegetation. Fences must be constructed parallel to the contour as much as possible, be located behind existing vegetation, as seen from the major viewpoints, and be placed low in the landscape. Dams must 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 must 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 must 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 must 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 auxiliary spillway are to:

- provide durability, strength, and flexibility equivalent to the principal spillway;
- be watertight against anticipated pressures;
- be adequate for their intended use; and
- 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 and 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 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 should 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:

- dams of higher hazard class
- greater volume of reservoir storage
- dams with larger watersheds
- longer critical construction time periods
- 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 climatological record data for generation of synthetic hydrographs to develop stage duration-frequency information for a range of diversion rates.

The size of diversion must 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 must 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, require an analysis of the reservoir surface elevation fluctuation 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, auxiliary spillway, and freeboard hydrographs are found in the NRCS *National Engineering Handbook*, chapter 21, section 4, Hydrology (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 amounts

Principal spillway

Precipitation data must be obtained from the most recent National Weather Service (NWS) reference which is applicable to the area under study. References that 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 auxiliary spillway. Table 2-2 shows minimum return period. The minimum allowable areal adjustment ratios for 1- and 10-day precipitation amounts are tabulated in table 2-3 (a).

A storm duration of not less than 10 days must 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 must be selected based on which one requires the higher auxiliary spillway crest elevation when the principal spillway hydrograph is routed through the structure. Procedures to be used to estimate runoff volumes include:

- The runoff curve number (CN) procedure described in NEH-4. Use average antecedent runoff conditions (ARC II) or greater unless a special study shows that a different condition is justified. The CN adjustment for 10-day storm is estimated from table 2-3(b).
- Runoff volumes, based on stream gage studies which also account for snow melt, from figures 2-1(a) and (b) or 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-4, 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.

Auxiliary spillway and freeboard

The most recent NWS references applicable to the location of the dam site shall be used to determine precipitation amounts, spatial distributions, and temporal distributions. Table 2-1 provides references current as of date of this publication. See the NWS Web site, *http://www.nws.noaa.gov/oh/hdsc/index.html*, for the most current references.

Minimum precipitation amounts shall be in accordance with table 2-5.

The discharge capacity, stability (surface erosion potential), and integrity (breaching potential) of the auxiliary spillway shall be evaluated as follows:

- Both a short duration (6 hour or longer) and a long duration (24 hour or longer) storm shall be analyzed and the most critical results used to check the discharge capacity and the integrity of the auxiliary spillway.
- Only the short duration storm shall be used to check the stability of the auxiliary spillway.
- For locations where NWS references provide estimates of local storm and general storm values, both storms shall be analyzed. For other locations, at least a 6-hour and a 24-hour duration storm shall be analyzed.

In areas without applicable NWS references for spatial distribution, minimum areal adjustment ratios shown in figure 2-3 may be used. Spatial adjustments shall not be applied for drainage areas less than 10 square miles.

In areas without applicable NWS reference for temporal distribution, the dimensionless auxiliary and freeboard storm distribution shown in figure 2-4 may be used. Alternately, the 24-hour storm can be constructed by critically stacking incremental rainfall amounts of successive 6-, 12-, and 24-hour durations as described in Hydrometeorological Report 52 (HMR52).

Table 2-1 National Weather Service References[1/]: 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. I, Montana	Vol. II, Wyoming	Vol. III, Colorado
Vol. V, Idaho	Vol. VI, Utah	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)

Technical Paper 51. Hawaii (1965)

Technical Paper 52. Alaska (1965)

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

C. Durations from 5 minutes to 60 days and return periods to 100 years

NOAA Atlas 14 Volume 1. Precipitation Frequency Atlas for the Semiarid Southwest United States (2003)

NOAA Atlas 14 Volume 2. Precipitation Frequency Atlas for the Ohio River Basin and Surrounding States (2004)

D. Probable maximum precipitation (PMP)

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

Hydrometeorological Report 51. For 37 contiguous states east of the 105rd 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 55A. Between the Continental Divide and the 103rd meridian (1988)

Hydrometeorological Report 56. Tennessee Valley (1986)

Hydrometeorological Report 57. Pacific Northwest (1994)

Hydrometeorological Report 59. California (1999)

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

Technical Report 47. Alaska (1963)

1/ National Weather Service, National Oceanic and Atmospheric Administration (NOAA), U.S. Department of Commerce

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 auxiliary spillway types: [1/]	
				Earth	Vegetated
Low	single irrigation only [2/]	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}
Significant	single or multiple	all	none or any	P_{100}	P_{50}
High	single or multiple	all	none or any	P_{100}	P_{100}

[1] Precipitation amounts by return period 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 the 25 inches.

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

[4] Low Hazard Class dams involving industrial or municipal water are to be designed with a minimum criteria equivalent to that of Significant Hazard Class.

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

The maximum 6-hour rainfall should occur in the second 6-hour quadrant. The next highest 6-hour incremental rainfall should occur in the third 6-hour quadrant, the next highest in the first, etc.

The NRCS runoff curve number procedure defined in NEH-630 and NEH-4 shall be used to determine runoff volumes. Antecedent runoff condition (ARC) II or greater shall be assumed. The same curve number shall apply throughout the entire storm.

Special probable maximum precipitation (PMP) studies can be considered and may be conducted by NWS or other hydrometeorologists with experience in such work. Useful special studies may have been conducted by federal or state agencies or major dam owners. Special studies should be considered especially for large drainage areas, areas of significant variation in elevation, or areas located at the boundary of two studies where discontinuities occur.

Methods in the *Federal Guidelines for Dam Safety – Inflow Design Floods*, FEMA 94, may alternately be used to proportion the embankment and auxiliary spillway, provided downstream land use controls exist to prevent voiding incremental risk assumptions after the dam is completed.

Design hydrographs

Principal spillway hydrographs

Procedures in chapters 16 and 21, NEH-4 and applicable national computer programs shall 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 sub-basins 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.

Stability design (auxiliary spillway) and freeboard hydrographs

Procedures in chapters 16 and 21, NEH-4 and applicable national computer programs shall be used to develop stability design (auxiliary spillway) and freeboard hydrographs using precipitation and runoff amounts and sub-basins, 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 must 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 described earlier will be used to develop the breach hydrograph peak discharge.

Lower dam

For the design of a lower dam, hydrographs shall 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 stability design (auxiliary spillway) hydrograph is used to evaluate the stability (erosion resistance) of any vegetated or earth spillway at the lower site. The combined freeboard hydrograph is used to determine the height of dam and to evaluate the integrity of any vegetated or earth spillway at the lower site.

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

In design of the lower dam, the time of concentration (T_c) of the watershed above an upper dam is used to develop the hydrographs for the upper dam. The T_c of the uncontrolled area above the lower site is used to develop the uncontrolled area hydrographs. If the T_c for the total area exceeds the storm duration, the precipitation amounts for the stability design (auxiliary spillway) and freeboard hydrographs must be increased by the values in the applicable NWS references (table 2-1).

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 sub-basins for developing the design hydrographs. Generally, the drainage area for a sub-basin should not exceed 20 square miles. Watershed modeling computer programs, such as the NRCS Technical Release (TR) 20 – Project Formulation-Hydrology or NRCS SITES – Water Resources Site Analysis, may be used for inflow hydrograph development. This software can be downloaded from *http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-models.html*.

If the T_c for the entire drainage area is greater than 24 hours, storm durations longer than the T_c should be tested to determine the duration that gives the maximum reservoir stage for the routed stability design (auxiliary 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 sub-basin, and storm orientation. Consideration shall 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 snowmelt 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 2-3 Principal spillway volume adjustments

(a)

Area (m^2)	--Area/point ratio-- 1 day	10 days	Area (m^2)	--Area/point ratio-- 1 day	10 days
10	1.000	1.000	45	0.951	0.976
15	0.977	0.991	50	0.948	0.974
20	0.969	0.987	60	0.944	0.972
25	0.965	0.983	70	0.940	0.970
30	0.961	0.981	80	0.937	0.969
35	0.957	0.979	90	0.935	0.977
40	0.954	0.977	100	0.932	0.966

* If area is greater than 100 square miles, request PMP from Conservation Engineering Division (CED).

(b)

Runoff curve numbers 1 day	10 days	1 day	10 days	1 day	10 days
100	100	80	65	60	41
99	98	79	64	59	40
98	96	78	62	58	39
97	94	77	61	57	38
96	92	76	60	56	37
95	90	75	58	55	36
94	88	74	57	54	35
93	86	73	56	53	34
92	84	72	54	52	33
91	82	71	53	51	33
90	81	70	52	50	32
89	79	69	51	49	31
88	77	68	50	48	30
87	76	67	49	47	29
86	74	66	47	46	28
85	72	65	46	45	28
84	71	64	45	44	27
83	69	63	44	43	26
82	68	62	43	42	25
81	66	61	42	41	24

* This table is used only if the 100-year frequency 10-day point rainfall is 6 or more inches. If it is less, the 10-day CN is the same as that for the 1-day CN.

Table 2-4 Minimum quick return flow for prinicipal spillway hydrographs

Ci	------- QRF ------- in/d	ft^3/sec/mi^2	Ci	------- QRF -------- in/d	ft^3/sec/mi^2
1.00	0	0	1.50	0.233	6.28
1.02	0.011	0.30	1.52	0.239	6.42
1.04	0.022	0.60	1.54	0.244	6.56
1.06	0.033	0.90	1.56	0.249	6.70
1.08	0.045	1.20	1.58	0.254	6.83
1.10	0.056	1/50	1.60*	0.259	6.95
1.12	0.067	1.80	1.65	0.270	7.26
1.14	0.078	2.10	1.70	0.280	7.53
1.16	0.089	2.40	1.75	0.290	7.79
1.18	0.100	2.70	1.80	0.299	8.05
1.20	0.112	3.00	1.85	0.309	8.30
1.22	0.122	3.29	1.90	0.318	8.54
1.24	0.133	3.58	1.95	0.326	8.77
1.26	0.144	3.86	2.00	0.335	9.00
1.28	0.153	4.12	2.05	0.343	9.22
1.30	0.163	4.37	2.10*	0.351	9.44
1.32	0.171	4.61	2.20	0.367	9.86
1.34	0.180	4.83	2.30	0.382	10.26
1.36	0.188	5.05	2.40	0.396	10.65*
1.38	0.195	5.25	2.50	0.410	11.02
1.40	0.202	5.44	2.60	0.423	11.38
1.42	0.209	5.63	2.70	0.436	11.73
1.44	0.216	5.80	2.80	0.449	12.07
1.46	0.222	5.97	2.90	0.461	12.41
1.48	0.228	6.13	3.00**	0.473	12.73

* Change in tabulation interval
** For Ci greater than 3, use:

$$QRF\ (ft^3/sec/mi^2) = 9(Ci-1)^{0.5}$$

or

$$QRF\ (in/d) = 0.03719\ [QRF\ (ft^3/sec/mi^2)]$$

where:

QRF = quick return flow
Ci = climatic index
in/d = inches per day

Table 2-5 Minimum auxiliary spillway hydrologic criteria

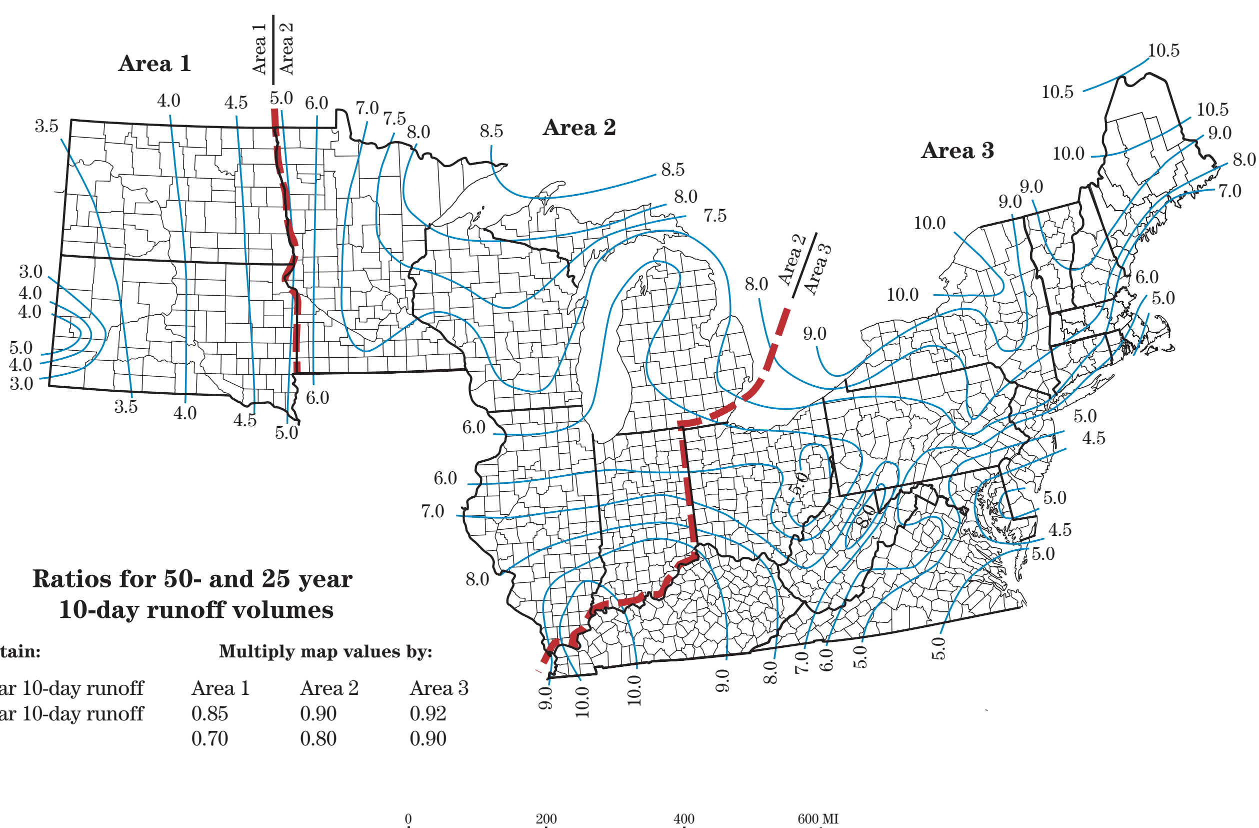
Class of Dam	Product of storage X effective height	Existing or planned up-stream dams	Precipitation data for [1]	
			Auxiliary spillway hydrograph	Freeboard hydrograph
Low [2]	less than 30,000	none	P_{100}	$P_{100} + 0.12(PMP - P_{100})$
	greater than 30,000		$P_{100} + 0.06(PMP - P_{100})$	$P_{100} + 0.26(PMP - P_{100})$
	all	any [3]	$P_{100} + 0.12(PMP - P_{100})$	$P_{100} + 0.40(PMP - P_{100})$
Significant	all	none or any	$P_{100} + 0.12(PMP - P_{100})$	$P_{100} + 0.40(PMP - P_{100})$
High	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 Significant Hazard Class.
3 Applies when the upstream dam is located so that its failure could endanger the lower dam

Figure 2-1 Principal spillway runoff volumes in north-central and southeastern states

(a) 100-year, 10-day runoff (inches), principal spillway hydrograph

100-year 10-day runoff volumes (inches)
for developing the principal spillway hydrograph



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	0.90

Figure 2-1 Principal spillway runoff volumes in north-central and southeastern states–continued

(b) Ratios of volumes of runoff (A1/Q10), principal spillway hydrograph

Ratios of volumes runoff (Q_1/Q_{10})
for developing the principal spillway hydrograph

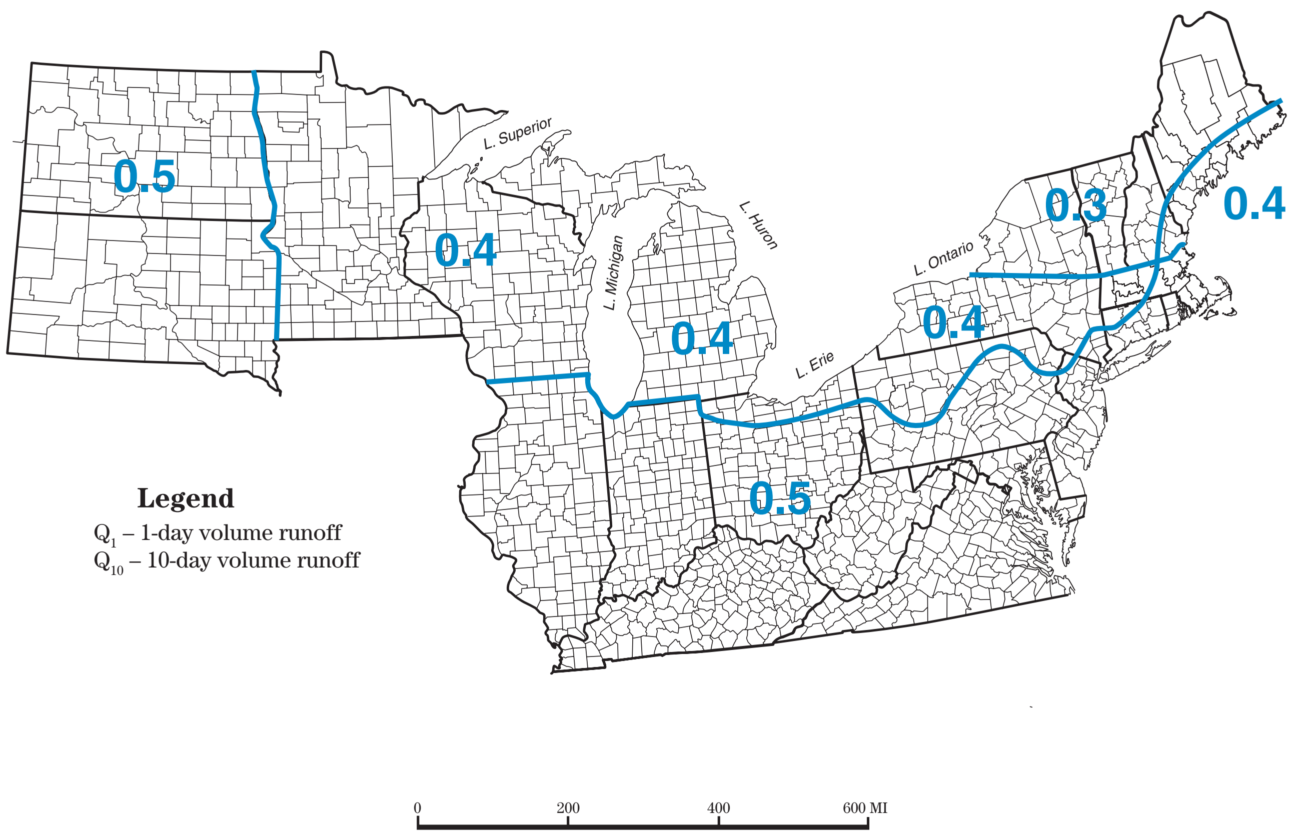


Figure 2-1 Principal spillway runoff volumes in north-central and southeastern states–continued

(c) Quick return flow

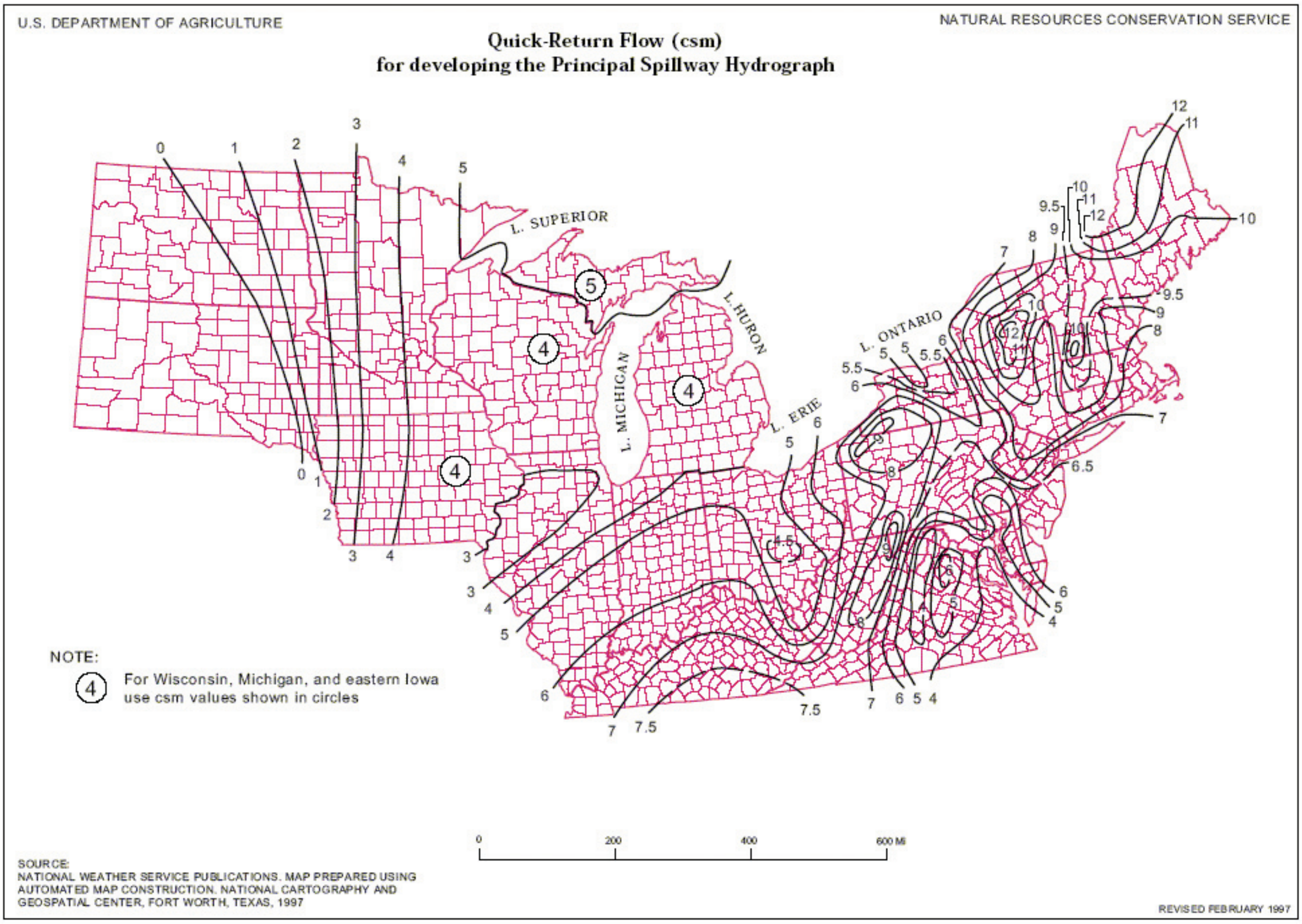


Figure 2-2 Principal spillway runoff volumes in snowmelt producing flood areas

(a) 100-year, 10-day runoff (inches)

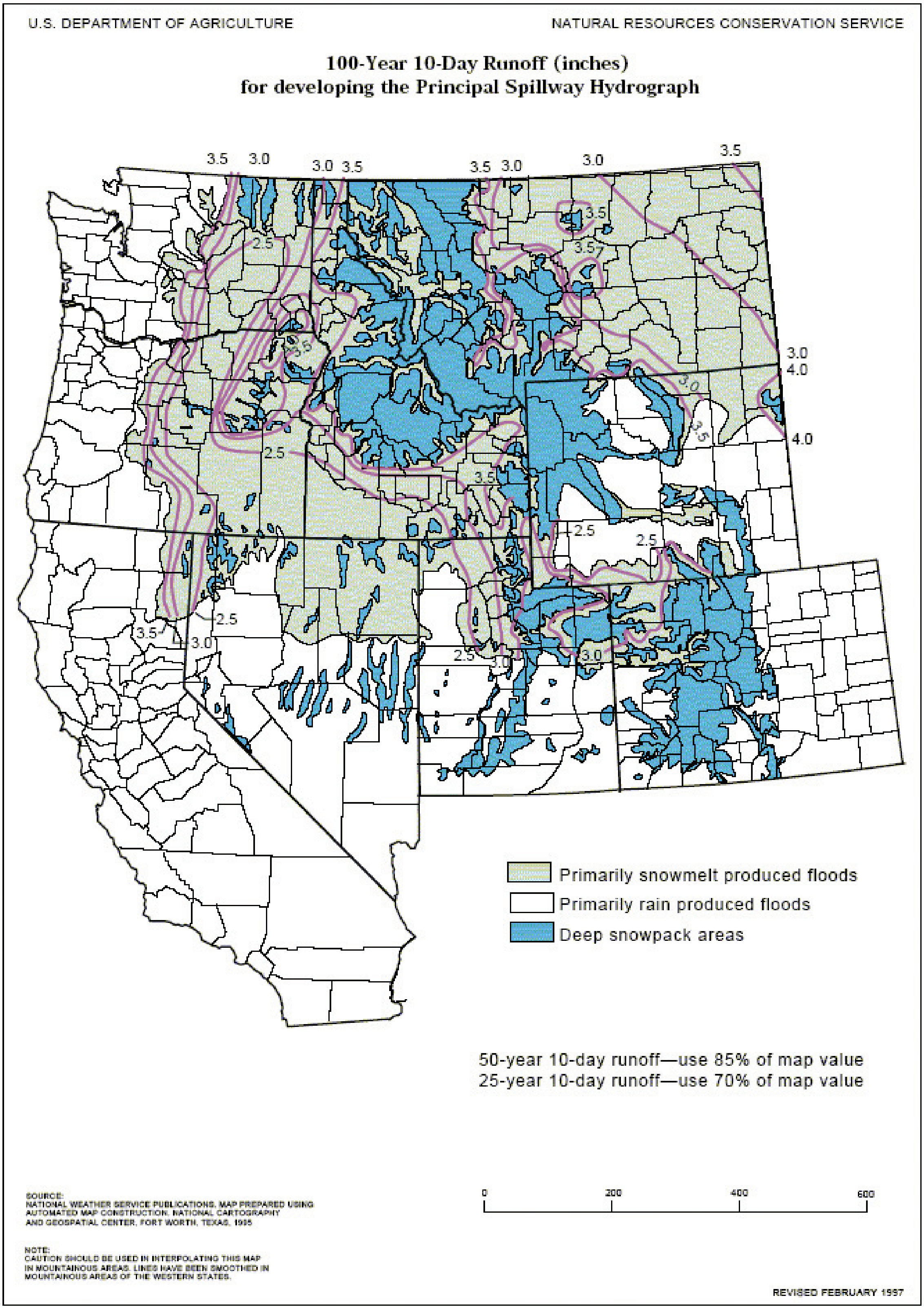


Figure 2-2 Principal spillway runoff volumes in snowmelt producing flood areas—continued

(b) Ratios of volumes of runoff (Q_1/Q_{10})

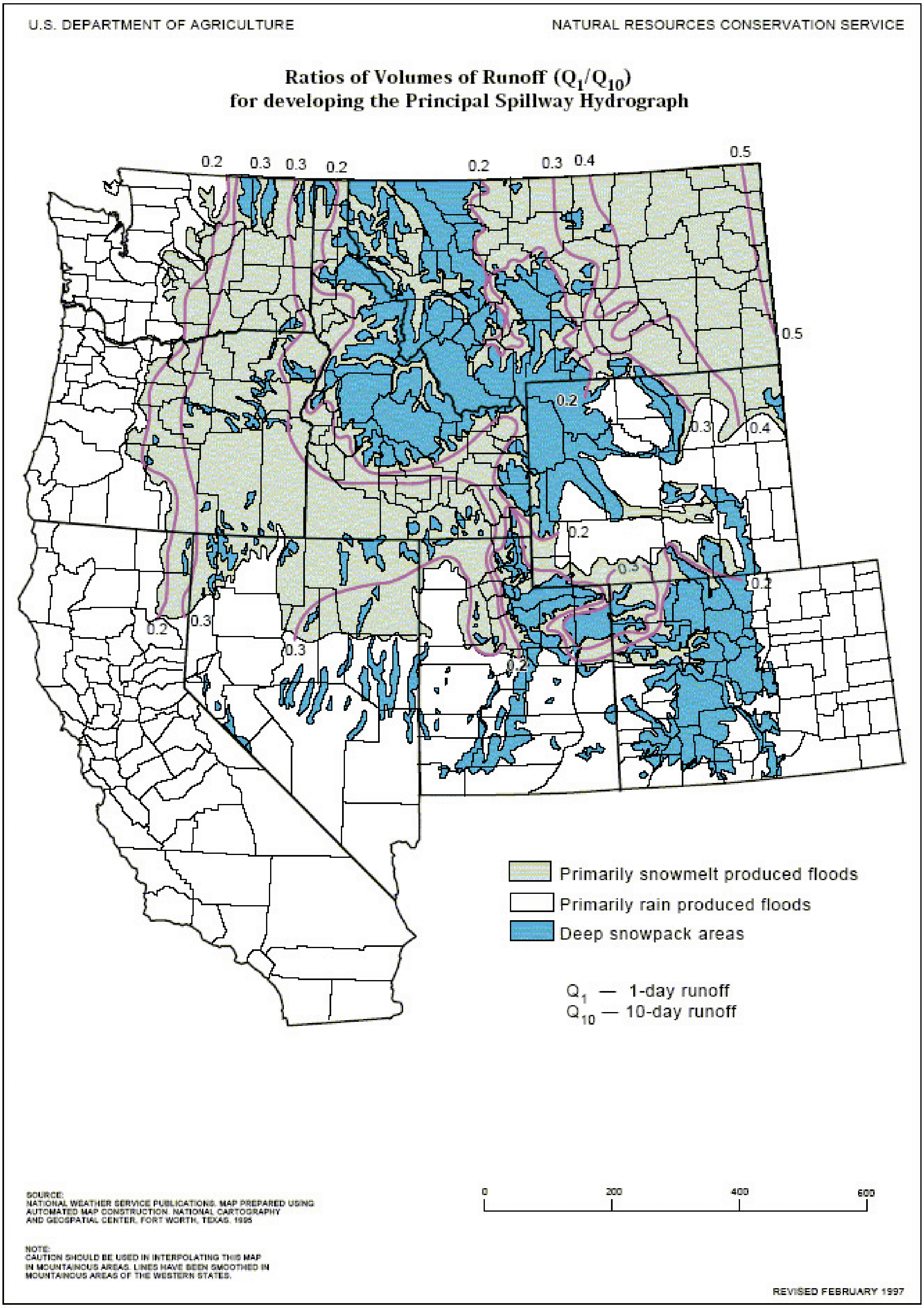


Figure 2-3 Areal adjustment, auxiliary spillway and freeboard

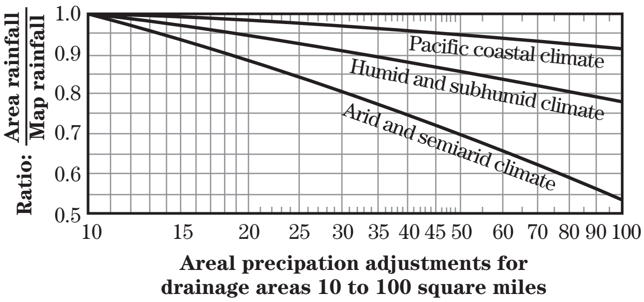
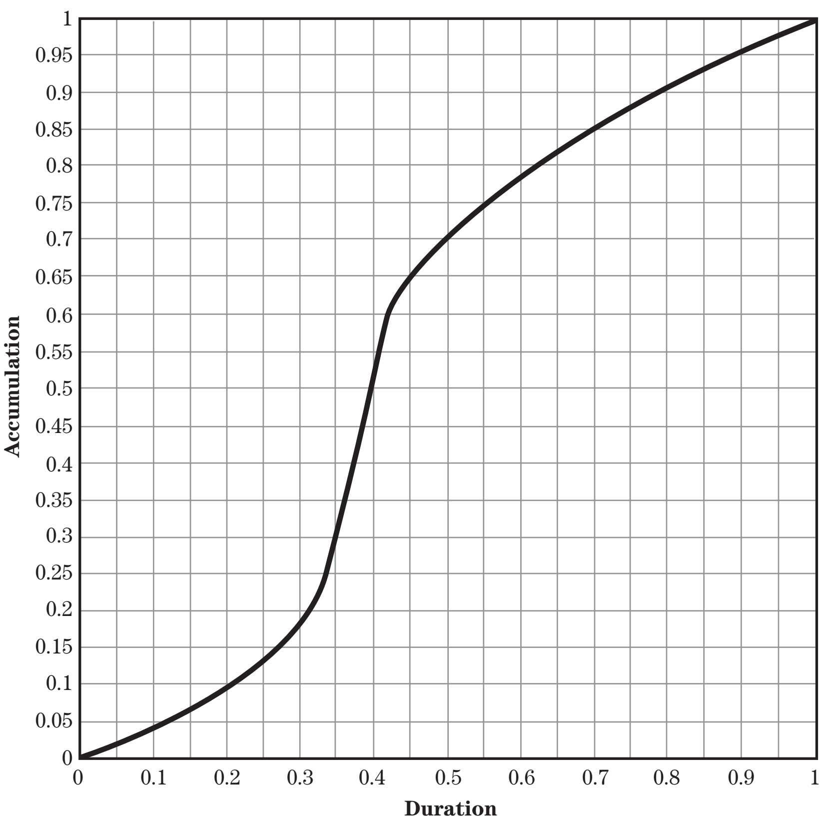


Figure 2-4 Dimensionless design storm distribution, auxiliary spillway and freeboard



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. The latter also includes procedures for determining:

- 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;
- trap efficiency of the reservoir;
- distribution and types of sediment expected to accumulate;
- proportion of the sediment that will be continuously submerged vs. that aerated; and
- 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 shall 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 the NEM-531 and NEH-8.

Following are the geologic conditions that require special consideration beyond the minimum investigations spelled out in the above reference.

Seismic assessment

Dams in zones 3 and 4, Alaska, Puerto Rico, and the Virgin Islands, and High Hazard Class dams in zone 2 (fig. 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 must 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 (62-mile) radius of the site. (Obtain earthquake information for this map from NOAA at *www.ngdc.noaa.gov/seg/hazard/int_srch.shtml* and USGS at *wwwneic.cr.usgs.gov/*.) 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.

Subsidence

Investigate the potential for surface subsidence due to past or future solid, liquid (including ground water) or gaseous mineral extraction. NEM-531, subpart B 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 eolian 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.

Auxiliary spillways

Large dams with auxiliary spillways in soft rock or cemented soil materials that cannot be classified as soil as defined in NEH-628, chapter 52 or as rock, as generally defined for engineering purposes, and spillways in rocks with extraordinary defects require a special individual evaluation.

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. Auxiliary spillway cuts and reservoir effects must be given careful consideration.

Karstic areas

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

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.

Other

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

Figure 4-1 Seismic zone map

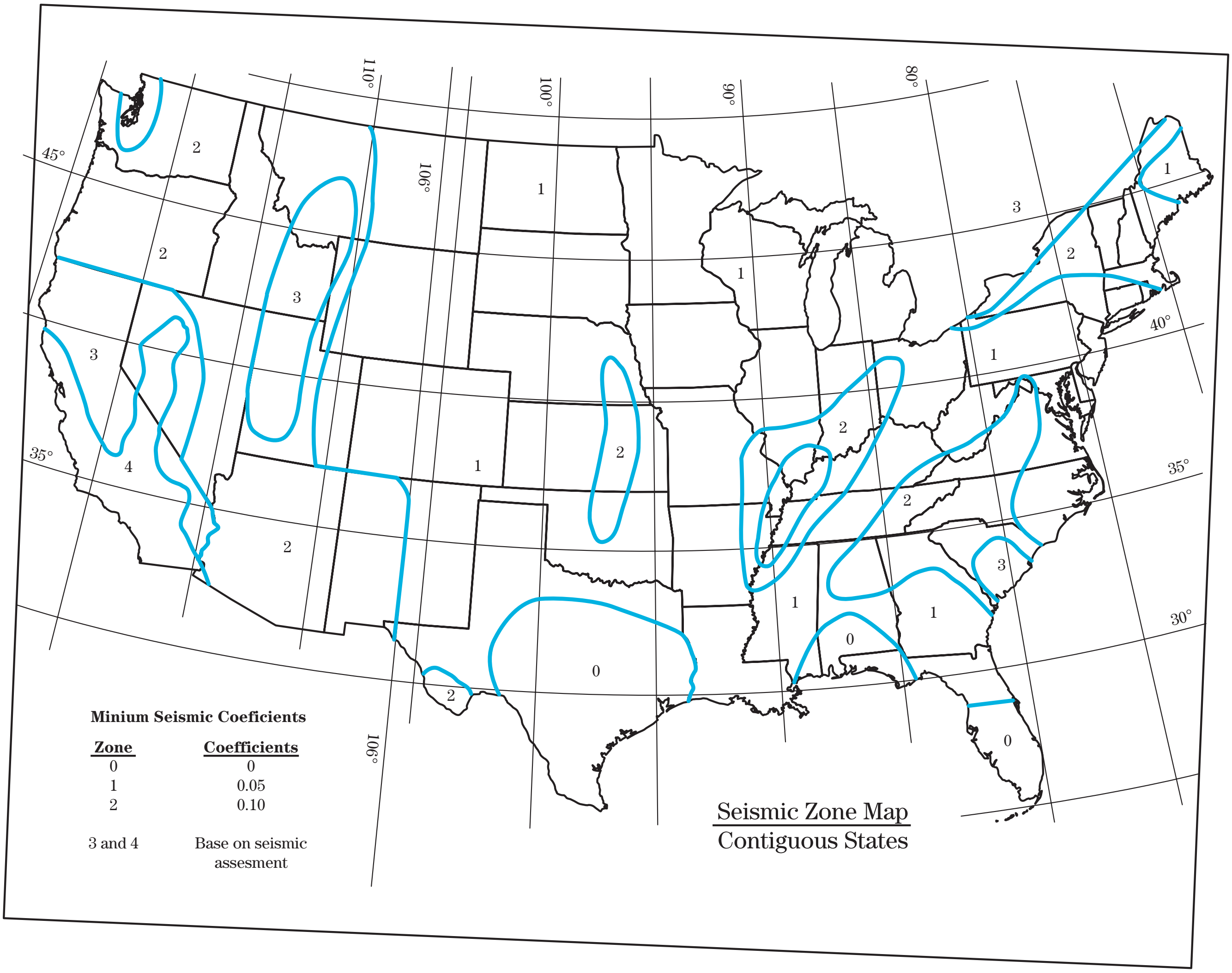
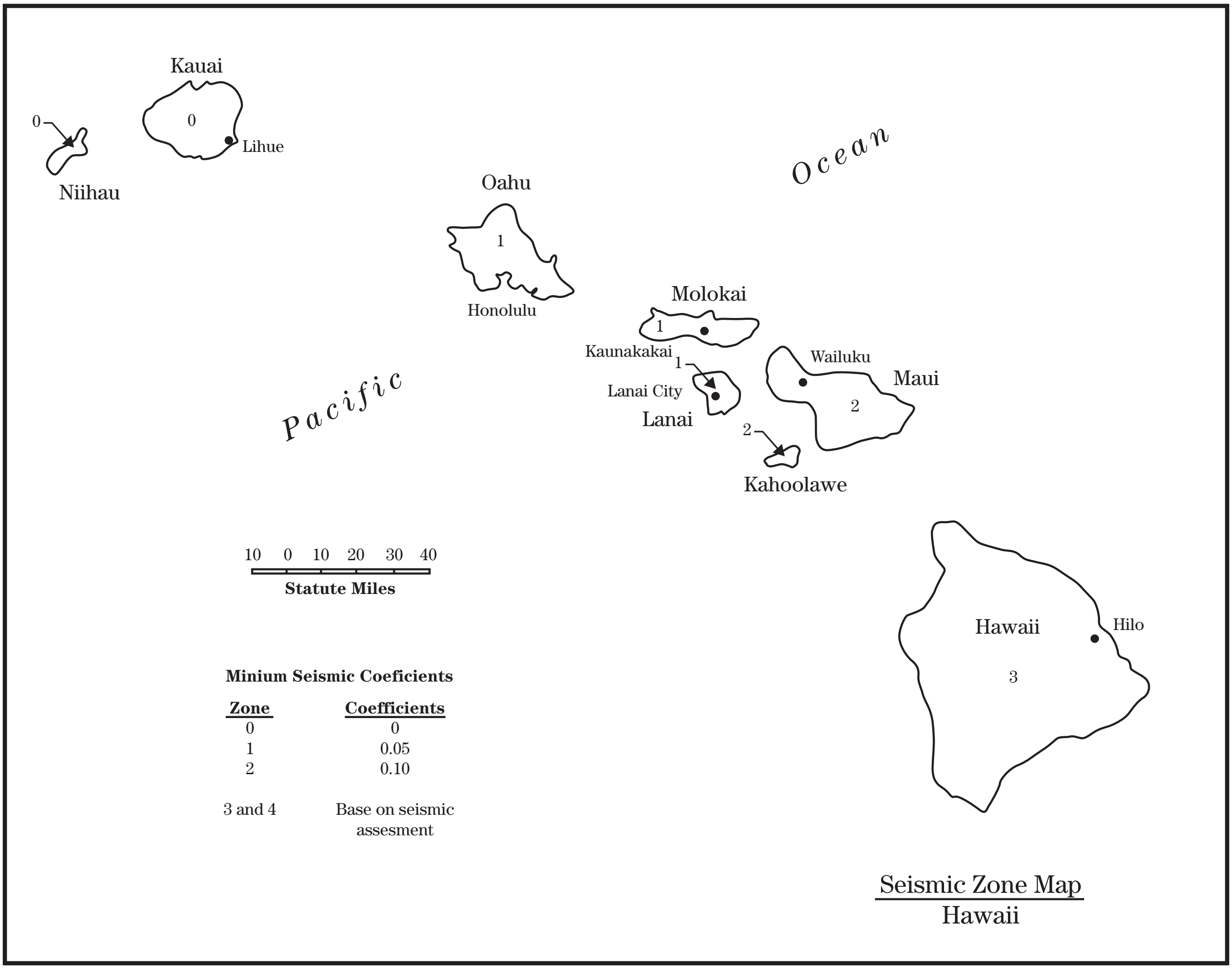


Figure 4-1 Seismic zone map—continued



Earth embankments and foundations

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

Height

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

Top width

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

The width may need to be greater than the above minimums to:

- meet state and local standards;
- accommodate embankment zoning;
- provide roadway access and traffic safety; and
- 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 shall be 16 feet for one-way and 26 feet for two-way traffic. Guardrails or other safety measures shall be used and must 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 including assumptions regarding shear strength parameters for each zone of the embankment and each soil type or horizon in the foundation. Documentation should include methods used for analyses and a summary of results. Design features necessary to provide required safety factors should be noted.

Table 5-1 Minimum top width of embankment

Total height of embankment, H, (ft)	Top Width (ft) All dams	Top Width (ft) Single purpose floodwater retarding	Top Width (ft) Multipurpose or other purposes
14 or less	8	N/A	N/A
15–19	10	N/A	N/A
20–24	12	N/A	N/A
25–34	14	N/A	N/A
35–95	N/A	14	(H+35)/5
Over 95	N/A	16	26

Use the appropriate degree of conservatism in the analysis that is consistent with the adequacy of the site investigation and the soil-testing program. Consider the complexity of the site and consequences of failure in determining the level of detail in the analyses. Minimum required safety factors are summarized in table 5-2 for each condition analyzed.

Evaluate the effect of seismicity on each site. Determine whether the site is in a seismically active area, its proximity to active faults, and the predicted ground motion intensity at the site. If the site is located in zone 3 or 4 shown in figure 4-1, perform special seismic studies. Otherwise, use the horizontal acceleration factors shown in figure 4-1 in a pseudo-static stability calculation using conditions summarized in table 5-2.

Analyze embankment stability for each of the following conditions in the design life of the structure that are appropriate to the site. If a condition is not analyzed, clearly document the reasons. Document any correlated shear strength parameters, including correlations to field performance, used to justify a lack of detailed analyses of a particular condition.

End of construction

This case should be analyzed when either embankment or foundation soils (or both) are predicted to develop significant pore pressures during embankment construction. Factors determining the likelihood of this occurring include the height of the planned embankment, the speed of construction, the saturated consistency of foundation soils, and others. Perform appropriate shear tests to model placement conditions of embankment soils, as summarized in table 5-2. Consider the highest likely placement water content of embankment soils in the shear-testing program. Either field vane shear tests or laboratory tests should determine the unconsolidated/undrained strength of slowly permeable foundation soils. The undrained strength of foundation clays should be corrected for plasticity index when field vane shear tests are used for measurement.

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 from the reservoir level from which a phreatic line is likely to develop. Ordinarily, assume a phreatic line has developed from the normal full reservoir elevation. Then, assume the water in the reservoir is rapidly lowered to the elevation of the lowest gated or ungated outlet and analyze the stability of the embankment following this drawdown. For rare situations, the upstream drawdown condition should be analyzed assuming that soils are saturated from temporary pool storage to the elevation of the auxiliary spillway. This condition should be analyzed if it is possible that in the future the riser could become plugged or other circumstances could cause the temporary impoundment to this higher elevation. For this condition, assume a zone of saturated soils in the embankment occurs based on transient flow nets that determine the portion of the embankment likely to become saturated during the temporary storage. Transient flow nets may be used to examine the temporary saturation of the portion of the embankment above permanent normal pool. Select shear parameters for the analyses according to table 5-2, as illustrated in figure 5-1.

Use infinite slope equations and appropriate safety factors in analyzing the stability of zones in the exterior slope of the embankment with soils having zero effective cohesion parameters, when the critical failure is wholly within soils of this character. If cohesionless zones occur with other soil types in a cross section being analyzed, circular arc or wedge-shaped surfaces should also be explored that intersect the soil zones with cohesion to locate the minimum safety factor. Note that different safety factors are considered adequate for infinite slope analyses than for failure surfaces that are deeper within the profile. See table 5-2 for details.

Figure 5-1 Mohr-Coulomb Envelope for upstream drawdown

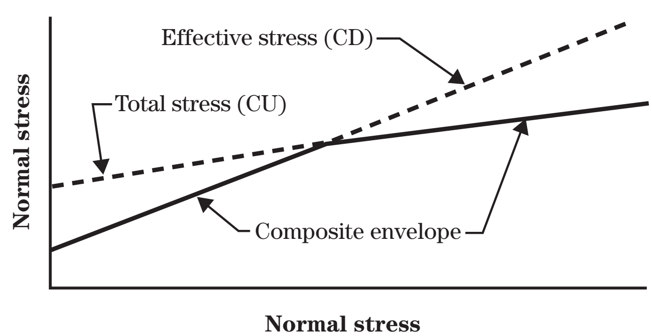


Table 5-2 Slope stability criteria

Design condition	Primary assumption	Remarks	Shear strength to be used	Minimum safety factor
1. End of construction (upstream or downstream slope)	Zones of the embankment or layers of the foundation are expected to develop significant pore pressures during construction	Embankment soils that are slowly permeable should be tested at water contents that are as wet as likely during construction (usually wet of optimum) Saturated slowly permeable foundation soils that are not predicted to fully consolidate during construction Permeable embankment zones and/or foundation strata	UU – includes triaxial UU tests, unconfined compression (qu) tests, and field vane shear tests CU' or CD	1.4 for failure surfaces extending into foundation layers 1.3 for embankments on stronger foundations where the failure surface is located entirely in the embankment
2. Rapid drawdown (upstream slope)	Drawdown from the highest normal pool to the lowest ungated outlet	Consider failure surfaces both within the embankment and extending into the foundation	Lowest shear strength from a composite envelope of CU and CD envelopes (fig. 5-1)	1.2 1.1 for infinite slope analysis
3. Steady seepage without seismic forces (downstream slope)	Phreatic line developed from pool at the principal spillway crest Uplift pressure simulated by phreatic line developed from auxiliary spillway crest applied to saturated embankment and foundation soils	Consider failure surfaces both within the embankment and extending into the foundation	Lowest shear strength from a composite envelope of CU and (CU+CD)/2 envelopes (fig. 5-2)	1.5 1.1 for infinite slope analysis
4. Steady seepage with seismic forces (downstream slope)	Phreatic line developed from principal spillway crest with no uplift	Consider failure surfaces both within the embankment and extending into the foundation	Lowest shear strength from a composite envelope of CU and (CU+CD)/2 envelopes (fig. 5-2)	1.1

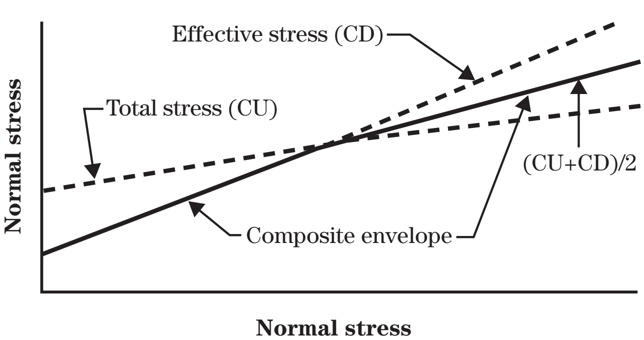
Steady seepage without seismic forces
Using shear parameters, as specified in table 5-2 and illustrated in figure 5-2, analyze the downstream slope considering a phreatic line developed from the reservoir at the principal spillway crest. Subject saturated soils below the phreatic line to an uplift force simulated by a phreatic surface developed from the auxiliary spillway crest. Phreatic surfaces for the analyses may be developed using flow nets or Casagrande procedures.

Use infinite slope equations and appropriate safety factors as under Rapid Drawdown condition.

Steady seepage with seismic forces
Using shear parameters, as specified in table 5-2, analyze the downstream slope considering a phreatic line developed from a pool at the principal spillway crest. Apply a horizontal acceleration constant appropriate to the seismic zone in which the site is located, as specified in figure 4-1. Do not use uplift forces due to a reservoir stage at the auxiliary spillway because the likelihood of a simultaneous occurrence of an earthquake and an auxiliary spillway flow event is extremely remote.

Use infinite slope equations and appropriate safety factors in analyzing the stability of zones in the exterior slope of the embankment with soils having zero effective cohesion parameters. The equations should incorporate the horizontal acceleration constant specified in figure 4-1.

Figure 5-2 Mohr-Coulomb Envelope for downstream steady seepage



Consider following additional guidance in analyzing slope stability.

- Only effective stress (CD) parameters are appropriate for soils that will consolidate as rapidly as load is applied. These parameters are applicable for all conditions of stability analyzed for these soil types.
- The end of construction condition is usually the one that controls the design of embankments when it is applicable. Special designs including staged construction or monitoring pore pressures during construction may be required to achieve objectives for some site conditions such as soft clay foundation soil horizons.
- Total stress parameters used in the construction of composite shear strength envelopes should not consider large negative pore pressures that may develop during shear testing. Either maximum deviator stress or maximum principal effective stress ratio failure criteria should usually be used to define total stress parameters. Total stress parameters interpreted from maximum arbitrary strain criteria should be used only when other criteria are less limiting.
- Infinite slope equations should model the predicted seepage pattern in the slope being analyzed. The three equations used are for no seepage, for horizontal seepage paths, and for seepage paths parallel to the slope face.
- Residual effective stress parameters should be used for modeling slope stability analyses involving fissured clays or shales if pre-existing movements have occurred. These parameters are based on drained shear strength tests where stresses are determined at high strain values beyond the peak strength. Residual parameters may also be considered for designing against shallow slope failures in desiccated clay embankments.

Seepage

To the extent needed, an analysis shall 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:

- accomplish the intended reservoir function;
- provide a safely operating structure; and
- prevent damage to downstream property.

Zoning

Embankment zoning can be used when needed to:

- obtain a stable structure with the most economical use of available materials;
- control seepage in a safe manner; or
- 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 should 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 must be:

- treated to improve their performance;
- placed in zones where effects will not be detrimental; or
- protected by use of filters and drains or self-healing transition zones.

Surface protection

Embankment surfaces must 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:

- inundation of the surfaces is of such frequency that vegetative growth will not be inhibited;
- vigorous growth can be sustained under average climatic conditions by normal maintenance without irrigation; and
- stable protection can be designed according to the procedures in TR-56.

Structural protection

Protection against wave erosion by riprap or other structural measures shall be provided for:

- dams where vegetation will not provide effective control;
- multiple purpose dams; and
- dams with fluctuating normal water levels.

Protection must 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 shall be based on an analysis of anticipated wave height and run up.

Quality of riprap and other structural protection must 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 must conform to the recommendations of NEH-6, Structural Design, and NRCS standard drawings. All component parts of principal spillways except easily replaceable parts such as gates and trash racks shall be equally durable.

Capacity of principal spillway

The required capacity of the principal spillway depends on the:

- purpose of the dam;
- amount of storage provided by the retarding pool;
- kind of auxiliary spillway;
- stream channel capacity and stability downstream;
- potential damage from prolonged storage in the retarding pool;
- potential damage downstream from prolonged high outflow rates;
- possibility of substantial runoff from two or more storms in the time required to empty the retarding pool;
- limitations imposed by water rights or other legal requirements;
- environmental concerns;
- planned or potential alterations of the channel downstream; and
- 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 auxiliary 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 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 auxiliary 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 must 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 auxiliary spillway must 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 shall be set at the submerged sediment pool elevation. For dry dams, the elevation of the principal spillway inlet shall be placed as described above and provisions 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 will be provided, the elevation of the crest of the lowest ungated inlet of the principal spillway is 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 shall 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 shall 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 below.

- 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.
- 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.
- Single purpose, low hazard class irrigation dams with gated outlets and earth or vegetated auxiliary 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, exclusive of sediment storage in determining the elevation to start routing.

Design of principal spillways

Hydraulics

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

Risers

Risers for drop inlet spillways must 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 shall 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 will be at least two times the conduit width (diameter) below the crest. Baffle inlets are applicable for risers that will be backfilled to the crest elevation or where sediment is expected to build up to the crest elevation.

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

Risers with low-stage inlets at or near the bottom must 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 must be used where applicable for low hazard class dams with an effective height of more than 35 feet and for all significant and high hazard class dams. Prefabricated pipe risers are permissible, where hydraulically and structurally adequate, for low hazard class dams not more than 35 feet in effective height. The riser pipe must be of the same material as the conduit and at least one standard pipe size larger than 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, must 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 must be provided if special pipe elbows are used. They must be designed to distribute the thrust due to change in direction for the maximum possible discharge. Drop inlet conduits shall be installed with enough slope to ensure 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 must support the external loads with an adequate factor of safety. They must withstand the internal hydraulic pressures without leakage under full external load and settlement. They must 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 shall be designed as positive projecting conduits in accordance with the principles and procedures given in TR-5.

Principal spillway conduits must be of reinforced concrete pressure pipe or cast-in-place reinforced concrete, unless corrugated steel or welded steel pipe is used.

Cast-in-place rectangular reinforced concrete conduits must 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 Standard C301, the 3-edge bearing strength at the first 0.001-inch crack shall be used with a safety factor of at least one.

For Reinforced Concrete Water Pipe—Steel Cylinder Type, Not Prestressed, meeting specification AWWA Standard C300; for Reinforced Concrete Water Pipe Noncylinder Type, Not Prestressed, meeting specification AWWA Standard C302, and for other types of reinforced concrete pipe, the 3-edge bearing strength at the first 0.01-inch crack shall 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 must be designed to support at least 12 feet of earth fill above the pipe at all points along the conduit.

- Reinforced Concrete Pipe
  - Minimum inside diameters on yielding foundations

    Low hazard class dams: The minimum diameter of the principal spillway conduit must be 30 inches, unless a joint extension safety margin of at least 1.5 inches is used, in which case, the minimum diameter shall be 18 inches for maximum fill heights up to 50 feet at the centerline of the dam and 24 inches for greater fill heights.

    Significant hazard class dams: The minimum diameter of the principal spillway conduit must be 30 inches, unless a joint extension safety margin of 1.5 inches is used, in which case, the minimum diameter shall be 24 inches.

    High hazard class dams: The minimum diameter of the principal spillway conduit must be 30 inches.
  - Minimum inside diameters on nonyielding foundations: The minimum diameter of the principal spillway conduit for low hazard class dams must 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 significant and high hazard class dams. The conduit and cradle or bedding must 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.
- Corrugated steel pipe or welded steel pipe

  Principal spillways of corrugated steel or welded steel pipe may be used for single purpose low hazard class 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:

- diameter of pipe not less than 18 inches;
- height of fill over the pipe not more than 25 feet;
- provision for replacement if the materials will not last for the design life of the structure;
- pipe structurally strong enough to withstand outside loads and hydraulic pressure; and

- pipe watertight.

Corrugated steel pipe shall be polymer-coated with watertight connecting bands. The minimum gage must be designed for 35 feet of fill over the pipe.

Welded steel pipe conduits must be structurally designed as rigid pipe. A joint extension safety margin of 1.5 inches shall be provided for conduits on yielding foundation. Welded steel pipe must be protected by a Class A exterior coating as defined in Conservation Practice Standard 430-FF, Irrigation Water Conveyance, Pipeline, Steel, or by an exterior coating of coal tar-epoxy paint conforming to Paint System F, Construction Specification 82 (NEH-645).

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 shall be made. The resistivity measurements are made on saturated samples.

Cathodic protection must be provided for welded steel pipe conduits according to the criteria in Conservation Practice Standard 432-FF in the National Handbook of Conservation Practices (NHCP).

Cathodic protection meeting the above requirements must 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 must 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 must be installed at this time if such measurements indicate it is needed.

Joints

Conduit joints shall 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 significant and high hazard class dams, the consolidation must be estimated from adequate foundation borings and samples, soil mechanics laboratory tests, and engineering analysis. For low hazard class 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 shall 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) shall 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 must 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 multiple zones to meet the requirements of NEH-633, chapter 23.

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:

- horizontally and vertical upward 3 times the outside diameter of circular conduits or the vertical dimension of rectangular box conduits except that:
  - vertical extension need be no higher than the crest of the auxiliary spillway, or higher than 2 feet below the embankment surface, and
  - horizontal extension need be no further than 5 feet beyond the sides and slopes of any excavation made to install the conduit.
- vertically downward:
  - for conduit settlement ratios (δ) of 0.7 and greater (reference NRCS 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.
  - 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 two times the outside diameter from the surface of flexible conduits, except that the diaphragm need not extend beyond the limits in the above or 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 capacity, tying into embankment or foundation drainage systems, accommodating construction methods, or other reasons.

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

- downstream of the cutoff trench;
- downstream of the centerline of the dam when no cutoff trench is used; and
- 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 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.

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

- does not create a piping hazard in the foundation of the structure.
- is compatible with other conditions at the site.

Plunge pools shall 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, Armored Scour Hole for Cantilever Outlet, shall be used for design.

Cantilever outlets shall be supported on bents or piers and must extend a minimum of 8 feet beyond the bents or piers. The bents must 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 must extend below the lowest elevation anticipated in the plunge pool. The invert of the cantilever outlet must be at least 1 foot about the tailwater elevation at maximum discharge.

- SAF basins may be used when there is adequate control of tailwater. Use TR-54 for structural design and NEH-14 for hydraulic design.
- Impact basins may be used where 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 shall 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 must not exceed 2.5 feet per second under the full range of stage and discharge. Velocity must 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 must 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 must 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 shall 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. (ASTH 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.

Auxiliary spillways

Auxiliary 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 auxiliary spillway must be provided for each dam except as provided below:

- Low hazard class 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 auxiliary spillway.
- 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 auxiliary spillway.
- Dams with drainage areas greater than 10 square miles (except those covered by item 1) – a closed type primary auxiliary spillway may be used in lieu of an open channel auxiliary spillway. Drop inlet spillways with a standard two-way covered top inlet must have a minimum unobstructed cross-sectional area of each opening of the conduit of 40 square feet. All other closed type primary auxiliary spillways must have a minimum unobstructed cross-sectional area of each opening of 80 square feet. The ratio of width to height in both cases is must 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.

Spillway requirements

Capacity of auxiliary spillways

Auxiliary spillways must be proportioned so they will pass the stability design hydrograph at the safe velocity determined for the site. They must 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 auxiliary spillway to be less than 200 ft^3/sec or 237 DA $^{0.493}$. The minimum difference in elevation between the crest of the auxiliary spillway and the settled top of the dam is 3 feet. State law may establish minimum capacity or depth greater than those given above.

Elevation of the crest of the auxiliary spillway

Table 2-2 gives the maximum allowable frequency of use of earth and vegetated auxiliary spillways. The minimum retarding storage volume and the associated principal spillway discharge must be such that the discharge through the auxiliary spillway will not occur during the routing of the principal spillway hydrograph and the 10-day drawdown requirement is met or the crest elevation of the auxiliary spillway is raised as noted under 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, auxiliary spillways shall be designed for less frequent use. This may be done by raising the crest elevation, increasing the capacity of the principal spillway, adding a structural primary auxiliary spillway, or a combination of the above.

The maintenance required for the auxiliary 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.

Auxiliary spillway routings

The stability design and the freeboard hydrographs must 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 the following:

- dams with gated spillways and joint use storage capacity – stability design 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.
- single purpose, low hazard class irrigation dams – stability design 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 auxiliary spillway shall 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 shall be used to evaluate friction losses, compute water surface profiles and determine velocities. Policy on the selection of flow resistance values is given in the discussion of the various types of auxiliary spillways.

Structural stability

The spillway must 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 must have cut and fill slopes in earth and rock which are stable against sliding. Cut slope stability must be evaluated for the long-term weathered, natural moisture condition and for adverse moisture conditions associated with rapid drawdown from the auxiliary spillway design discharge.

Vegetated and earth auxiliary spillways

Vegetated and earth auxiliary 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 auxiliary 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 velocities, lower stresses, and less frequent use. Normally, they require more maintenance after a flow occurs.

Earth and vegetated auxiliary spillways are designed on the basis that some erosion or scour will occur during passage of infrequent storms, but the spillway will not breach during passage of the freeboard storm.

Hydraulic data in TR-39 Hydraulics of Broad Crested Spillways can be used in the design of vegetated or earth auxiliary spillways. A minimum vegetal retardance curve index of 5.6 as defined in Agriculture Handbook 667 (AH-667) Stability Design of Grass-Lined Open Channels shall be used to determine hydraulic capacity and vegetal stress in vegetated spillways. A minimum Manning's n of 0.02 shall be used for earth spillways. Actual hydraulic capacity of the spillway will be based on an appraisal of the roughness condition at the site.

Layout

Guidance on the layout of auxiliary spillways is provided in NEH-628.50, Earth Spillway Design. Spillways must be located away from the dam whenever possible. The layout and profile of vegetated or earth spillways should provide safety against breaching of the spillway during the passage of the freeboard hydrograph. Both extending the length and flattening the grade of the exit channel to delay or prevent headcut formation, and maximizing the bulk of material to contain any headcut advancement should be considered.

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

The exit channel must be straight and perpendicular to the level part of the inlet channel for a distance equal to at 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.

When the upstream edge of the exit channel is considered as a control section for hydraulic calculations, the exit channel grade shall be sufficient to ensure supercritical flow for all discharges equal to or greater than 25 percent of the maximum discharge through the 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 exit channel can be terminated at some point above the maximum tailwater elevation, or can be extended to the principal spillway outlet or natural stream channel below the dam. The exit channel can contain several different grades. In either layout, erosion will occur wherever allowable stresses are exceeded and maintenance is required to protect the integrity of the spillway. Land rights must be considered in making the decision on how to handle the return flow to the natural or constructed stream channel downstream from the dam, and where eroded materials will be deposited.

Stability design of earth and vegetated earth spillways

Limitations during routing of the stability design (auxiliary spillway) hydrograph – the maximum stress limitations given below for vegetated or earth 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 stress limitations are based on the maximum discharge in routing the stability design (auxiliary spillway) hydrograph and the assumption that uniform flow conditions exist in the exit channel.

When the anticipated average use of an earth or vegetated spillway is more frequent than once in 50 years, the allowable stress will be determined in accordance with AH-667. The allowable values may be increased 20 percent when the anticipated average use is once in 50 years, or 50 percent when the anticipated average use is once in 100 years. The allowable stress shall be determined for the actual vegetal cover conditions that can be reasonably expected to exist at the time of the flow. Values for grasses or mixtures not included in AH-667 shall be determined by comparing their characteristics with those that are described. Where special studies or investigations have been made to determine the allowable stress for a species, soil, and condition, those values may be used in lieu of those shown in the handbook.

Ramp spillways are not generally favored by the dam engineering profession, but may be used where alternate solutions are not practical. Ramp spillways shall not be steeper than 10 per cent. Ramp spillways must be constructed with the same compaction procedures and quality control as the earth embankment. The upper one foot of vegetated ramp spillways should be top soil.

Integrity design of earth and vegetated earth spillways

The spillway shall be evaluated for headcut development and advancement during passage of the freeboard storm using the procedures in NEH-628.51, Earth Spillway Erosion Model, and NEH-628.52, Field Procedures Guide for the Headcut Erodibility Index. The spillway design must be such that the spillway will not breach (i.e., headcut will not advance beyond the upstream edge of the level part of the inlet channel) during passage of the freeboard storm.

Special precautions for high hazard class dams—Special consideration must be given to the layout of spillways on high hazard class dams to assure the spillway will not breach under the most extreme conditions of flow. The length of the exit channel is to be increased to the maximum extent possible so that the area most susceptible 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 must be constructed of erosion resistant materials and compacted to the degree necessary to develop this resistance. They must 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 must 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 shall 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 should be given to the reduction of the duration and volume of flow through the auxiliary spillway by raising the elevation of the crest of the 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 auxiliary spillway

Some of the principles used for the layout of earth auxiliary spillways are applicable to rock auxiliary spillways. Allowable average frequency of use and permissible velocities must be ascertained for the specific site based on 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 auxiliary spillway will not breach during passage of the freeboard storm.

Structural auxiliary spillways

Structural spillways shall 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 must 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 shall 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 auxiliary spillways shall 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 entertainment by water traveling at supercritical velocities.

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

The outlet section of concrete chute spillways must 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 auxiliary spillways must be designed to withstand lateral earth pressures, uplift, seepage and other hydrostatic and hydrodynamic pressures. They must 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.

Glossary

Auxiliary spillway. The spillway designed to convey excess water through, over, or around a dam.

Auxiliary spillway system. A single auxiliary or combination of auxiliary spillways designed to work together.

Base flow. 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.)

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

Conservation storage. Water impounded for consumptive uses such as municipal, industrial and irrigation and nonconsumptive uses such as recreation and fish and wildlife.

Control section. In an open channel spillway, it is that section where accelerated flow passes through critical depth.

Dam. An artificial barrier together with any associated spillways and appurtenant works that do or may impound or divert water.

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

Dry dam. 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.

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

Earth spillway. An open channel spillway in earth materials without vegetation.

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

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

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

Freeboard hydrograph. Used to evaluate the total spillway flow capacity of the dam and, consequently, establish the minimum settled elevation of the top of the dam. It is also used to evaluate the integrity (breaching resistance) of a vegetated or earth auxiliary spillway.

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

Joint extensibility. The amount of a pipe joint that can be extended from the fully engaged position without losing strength or watertightness. In case of rubber gasket 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. 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.

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

Primary auxiliary spillway. The spillway with the lowest crest elevation in an auxiliary spillway system.

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

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

Quick return flow. 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.

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

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

Retarding storage. The volume in the retarding pool.

Rock spillway. An open channel spillway through competent, nonerodible, natural rock materials.

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

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

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

Spillway. 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.

Stability design hydrograph. The hydrograph used to establish the dimensions of the auxiliary spillway.

Storage. The capacity of the reservoir below the elevation of the crest of the auxiliary spillway.

Vegetated spillway. A vegetated open-channel spillway in earth materials.

Visual focal. An element in the landscape upon which the eyes automatically focus because the element's size, form, color, or texture contrasts clearly with its surroundings.