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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. It provides uniform criteria for earth dams and reservoirs. NRCS plans, designs, and constructs complex dams under widely varying conditions and it is essential to construct these dams with uniform criteria to ensure consistent performance. NRCS periodically revises this document to incorporate new experience, materials, and knowledge.

This TR references numerous NRCS and industry documents. Designers should use the latest version of these references.

This TR applies to—

- All low hazard potential dams with a product of effective storage times the effective height of the dam of 3,000-acre-feet^2 or more.
- Dams more than 35 feet in effective height.
- All significant and high hazard-class dams.

This TR provides requirements as maximum or minimum limits that may not satisfy design criteria for all sites. In some cases, problems may arise that this document does not address. Where the state of practice has advanced or where proven solutions are not available, designers may need to evaluate alternate procedures before developing and selecting the best solutions. To ensure satisfactory performance, experience, State laws and regulations, investigations, analysis, expected maintenance, environmental considerations, or safety laws may dictate criteria that are more conservative.

This edition of the TR incorporates all previously issued revisions and amendments, as well as significant changes in Parts 2, “Hydrology”; 4, “Geologic and Geotechnical Considerations”; 5, “Earth Embankments and Foundations”; 6, “Principal Spillways”; and 7, “Auxiliary Spillways.”

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PART 1 – GENERAL

Dam Classification

Title 210, National Engineering Manual, Part 520, Subpart C, "Dams" (210-NEM-520-C), establishes NRCS policy on dam classification. Classification requires consideration of damage that may 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. Embankment, spillway, and reservoir characteristics; physical characteristics of the site and the valley downstream; and 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.

The assigned dam classification, as determined by the above conditions influencing the potential hazard to life and property should the dam suddenly breach or fail, provides the basis for design criteria. If there is a potential for change in hazard potential classification over the design life of the structure because of future development, design of the dam should incorporate criteria appropriate for the potential future hazard for elements of the dam and appurtenances that are problematic to update. These include such elements as—

- Foundation treatment.
- Embankment zoning.
- Principal spillway material.
- Areas needed for future increase to size of dam and auxiliary spillways.

Consideration of a future change in hazard potential classification and associated design criteria may also include physical or administrative provisions to facilitate modifying dam, and auxiliary spillway dimensions, or pool elevations. Designs must also recognize State and local regulations and the responsibility of the involved public agencies.

Classes of Dams

210-NEM-520 classifies dams as follows:

- **Low Hazard Potential**.—Dams in rural or agricultural areas where failure may damage farm buildings, agricultural land, or township and country roads.
- **Significant Hazard Potential**.—Dams in predominantly rural or agricultural areas where failure may damage isolated homes, main highways or minor railroads, or interrupt service of relatively important public utilities.
- **High Hazard Potential**.—Dams where failure may cause loss of life or serious damage to homes, industrial and commercial buildings, important public utilities, main highways, or railroads.

Peak Breach Discharge Criteria

Use breach routings to help delineate the area potentially impacted by inundation should a dam fail and to aid dam hazard potential classification. Develop routings using topographic data and hydraulic methodologies mutually consistent in their accuracy and commensurate with the level of risk under evaluation. For hazard potential classification, evaluate probable downstream conditions that could exist for the failure mode being evaluated, and incorporate the condition that would represent the highest hazard into routings. Federal Emergency Management Agency (FEMA) 333, "Federal Guidelines for Dam Safety: Hazard Potential Classification System for Dams," requires the assignment of classification "based on the worst-case probable scenario of failure or misoperation of the dam," meaning assignment of hazard potential classification "based on failure consequences that will result in the assignment of the highest hazard potential classification of all probable failure and misoperation scenarios."

Evaluate dam failure with the water surface elevation of the reservoir at the dam crest or the peak reservoir stage resulting from the probable maximum flood (PMF). 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 $H_w \geq 103ft$

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

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

   $Q_{max} = 1100\ B_r^{1.35}$ 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—

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

   replace the equation, $Q_{max} = 65\ H_w^{1.85}$, in 1 and 2 above with—

   $$Q_{max} = 0.416LH_w^{1.5}$$

   Where—

   Q_{max} = peak breach discharge, cubic feet per second

   B_r = breach factor, for the equation, $B_r = \frac{V_s H_w}{A}$, acre

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

   H_w = depth of water at the dam at the time of failure; however, in the case of dam overtopping, not to exceed depth at the top of the dam, feet

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

$T =$ 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, feet

The peak discharge value determined by using principles of erosion, hydraulics, and sediment transport may be used in lieu of the peak discharge computed using the equations presented. Examples of acceptable, process-based models include the National Weather Service (NWS) BREACH model and NRCS WinDAM.

Cut Slope Stability

Plan and form excavated cut slopes 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. Part 4 of this TR discusses the requirements for a geotechnical investigation plan that may include the evaluation of natural slope stability. Part 5 of this TR discusses the stability evaluation of constructed slopes.

Reservoir Conservation Storage

Analyze reservoirs with water stored for conservation purposes using a water budget to determine a dependable water supply. For most purposes—

- NRCS defines a dependable water supply as one that is available at least 8 out of 10 years or has an 80-percent chance of occurring in any one year.
- A purpose such as municipal and industrial water supply may require a 95-percent chance of occurring 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.

Joint Use of Reservoir Capacity

Efficient use of a reservoir site occurs where hydrologic conditions permit joint use of storage capacity by floodwater and conservation storage. For joint-use storage dams, NRCS requires—

- Reasonable assurance of adequate water supply to meet project objectives.
- Satisfaction of flood protection objectives of the project.
- Spillway conditions that will enable the dam to perform safely.

NRCS may require special hydrologic studies to show compliance with the requirements listed above.

This may include hydrometeorologic instrumentation and analysis.

Hydraulic features must include an ungated spillway outlet at the top of the joint use pool. At the bottom of the joint use pool, provide a gated opening or other reliable means to remove water from the conservation storage to meet project objectives.

Make provisions 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 (O&M) 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 dam construction creates permanent pools, emphasis on water views can enhance the visual resource. A visual design objective must focus public views toward the permanent pool and reduce the visual focal effects of the structural elements.

Achieve visual focus on the lake 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 when the visual design keeps other elements subordinate.

Shape borrow areas to blend with the surrounding topography. Vegetate these areas with herbaceous and woody plants to provide a visual fit to the existing surrounding vegetation. As much as possible, locate fences parallel to the contour, behind existing vegetation as seen from the major viewpoints, and low in the landscape. Shape dams to blend with the natural topography to the extent feasible.

Safety and Protection

Many dams have features potentially 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. Design all dams 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 include safety fences, guardrails, or other safeguards necessary to protect the public and O&M personnel. Provide fences or other barriers where necessary to protect the dam from livestock, foot traffic, and vehicular traffic.

Water Supply Pipes

Water supply pipes or conduits for other purposes installed under any part of the embankment must—

- Provide durability, strength, and flexibility equivalent to the principal spillway.
- Include features to facilitate compaction for seepage control and filtered as required for principal spillway pipes.
- Be encased throughout the footprint of the embankment.
- Be watertight against anticipated pressures.
- Be adequate for their intended use.

- Have shutoff capability upstream and downstream of the embankment.

NRCS does not permit water supply pipes through previously constructed embankments unless there is no other technically viable alternative. Water supply pipes through previously constructed embankments must address all settlement, stability, seepage, and construction issues as other penetrations and must not damage or adversely affect any embankment zone or appurtenances.

Utility Cables and Pipelines

Dam sites frequently have existing pipelines, cables, and conduits of a wide variety of sizes, materials, and functions. These conduits are usually located at shallow depths in the floodplain. They constitute a hazard to the safety of the dam and require either relocation away from the dam, reconstruction, or modification. Reconstructed conduits must provide the durability, strength, and flexibility equal in all aspects to the principal spillway designed for the site, in accordance with NRCS criteria and procedures. Relocate or raise overhead cables or power lines as necessary to prevent damage or hazard to the public.

Designers should make every reasonable effort 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. NRCS only permits conduits to remain under an earth dam embankment as a last resort and under the limitations imposed.

Conduits remaining under any part of the embankment must have—

- Features to facilitate compaction for seepage control and filters as required for principal spillway pipes.
- Proper articulation on all yielding foundations.
- Encasement by concrete or otherwise treated to ensure durability and strength equal to that of the principal spillway.
- Watertight features preventing leakage either into or out of the casing and carrier pipe.

Acceptable enclosure of the conduit cable or pipeline requires enclosing the conduit within another conduit that meets the requirements of this section and positively sealing the upstream end to prevent seepage into the enclosing conduit. Such an enclosing conduit must extend the full distance through which the enclosed conduit, cable, or pipeline is beneath the embankment.

NRCS does not permit utilities through previously constructed embankments unless there is no other technically viable alternative. Penetrations for utilities through previously constructed embankments must address all settlement, stability, seepage, and construction issues as other penetrations and must not damage or adversely affect any embankment zone or appurtenances.

Streamflow Diversion during Construction

Determine requirements for diversion and care of streamflow during construction in the planning and design process and include diversion requirements in the specifications.

The construction of the permanent work and diversion features, such as cofferdams and ancillary structures, create a construction hazard from the time that structures obstruct natural streamflow until the dam and appurtenances are complete and are operational. The damage from overtopping or exceeding the capacity of the diversion works during construction can range from erosion repair and cleanup costs to a loss of the temporary diversion works, complete breach of the dam, or damage to the appurtenances.

Evaluate hazards during construction, including the costs of damage to the partially completed work, delay of function of the completed structure from extension of construction time, downstream hazard during different phases of construction, and environmental consequences, such as sedimentation from erosion of the diversion or permanent work.

During construction, a greater risk usually exists than the risk that exists after the dam is completed. The risk of damage or overtopping depends on—

- Hydrology.
- Watershed size.
- Upstream structures in the watershed.
- Construction duration.
- Individual project characteristics including—
    - New or rehabilitation construction.
    - Type of construction.
    - Embankment materials.
    - Site conditions.
- Diversion capacity.

Develop a suite of inflow floods spanning return periods of interest for evaluation of diversion requirements. Identify seasonal variability that may affect diversion requirements or construction schedule if applicable. Evaluate stage-duration-frequency information for a range of diversion conditions.

Select the tolerable level of construction risk and the diversion requirements to achieve a lower level of construction risk, based on the identified construction hazards. The allowable probability of inflow exceeding diversion capacity varies from a 20-percent chance in any 1 year for sites with limited hazard to less than a 1-percent chance in any 1 year for sites where there is extensive downstream hazard or there would be significant repair cost to the work.

Size the diversion to provide an acceptable level of risk during the construction period. As appropriate, specify cofferdam and diversion conveyance requirements, erosion protection, construction sequencing, timeframes and milestones required for diversion.

PART 2 – HYDROLOGY

Introduction

This part describes hydrologic criteria for determining spillway discharges and floodwater storage volumes. Title 210, National Engineering Handbook, Part 630, Chapter 21, "Design Hydrographs" (210-NEH-630-21), provides a detailed description of acceptable procedures and example problems for development of principal spillway, auxiliary spillway, and freeboard design hydrographs for TR 210-60, "Earth Dams and Reservoirs." 210-NEH-630-17, "Flood Routing," contains methods of flood routing hydrographs through reservoirs and spillway systems.

Special studies, as used in this text, refer to all site-specific studies undertaken with prior concurrence of the Director, Conservation Engineering Division.

Obtain precipitation data from the most recent National Oceanic and Atmospheric Administration (NOAA) National Weather Service (NWS) reference applicable to the area under study or from other special studies as appropriate. 210-NEH-630-21 contains a partial listing of references.

When computing runoff volumes using the NRCS runoff curve number procedure defined in 210-NEH-630-9, "Hydrologic Soil-Cover Complexes," and 210-NEH-630-10, "Estimation of Direct Runoff from Storm Rainfall," assume an antecedent runoff condition (ARC) II or greater for obtaining the runoff curve number. Apply the same runoff curve number throughout the entire storm.

Adjustments to runoff curve numbers and runoff volumes may be applicable in development of the design hydrographs. 210-NEH-630-21 includes examples of these optional adjustments. Make detailed evaluations to determine the appropriateness of these optional adjustments.

Principal Spillway Design Hydrographs

Use the runoff from a storm duration of not less than 10 days for sizing the principal spillway. The return period for design precipitation amounts depends on the dam's hazard potential classification, purpose, size, location, and type of auxiliary spillway. Figure 2–1 below shows minimum return period principal spillway hydrologic criteria by hazard potential classification.

Figure 2–1: Table of Minimum Principal Spillway Hydrologic Criteria

Class of dam	Purpose of dam	Product of storage (AC-FT) × effective height (FT)	Existing or planned upstream dams	Precipitation data for maximum frequency of use of auxiliary spillway types[1]	
				Earth	Vegetated
Low hazard	Single irrigation only[2]	less than 30,000	none	½ design life	½ design life
		greater than 30,000	none	¾ design life	¾ design life
	Single or multiple[4]	less than 30,000	none	P_{50}	P_{25}[3]
		greater than 30,000	none	P_{75}	P_{50}[3]
		all	Any[5]	P_{100}	P_{50}[3]
Significant hazard	Single or multiple	all	none or any	P_{100}	P_{50}
High hazard	Single or multiple	all	none or any	P_{100}	P_{100}

1. Precipitation amounts by return period in years. In some areas, NRCS allows the use of direct runoff amounts determined using procedures in 210-NEH-630, "Hydrology," in lieu of precipitation data.
2. Applies to irrigation dams on ephemeral streams in areas where the annual rainfall is less than 25 inches.
3. Increase the minimum earth spillway requirements to P_{100} for a ramp spillway.
4. Design low hazard potential dams involving industrial or municipal water with a minimum criteria equivalent to that of significant hazard potential.
5. Applies when the upstream dam is located so that its failure could endanger the lower dam.

Select the procedure for estimating runoff volumes that requires the higher auxiliary spillway crest elevation when routing the principal spillway hydrograph through the structure. 210-NEH-630-21 describes procedures used to estimate runoff volumes. The procedure for developing the 10-day principal spillway mass curve described in 210-NEH-630-21 uses both the 1-day and 10-day runoff volumes and includes procedures for making areal adjustments if appropriate.

In arid and semiarid climatic areas when the climatic index, as defined in 210-NEH-630-21, is less than one, NRCS allows the use of transmission losses to reduce the runoff. NRCS requires special studies when transmission losses appear to be significant, such as in cavernous areas, even though the climatic index is one or more.

Use procedures described in 210-NEH-630-16 and 630-21, and applicable national computer programs to develop the principal spillway hydrograph.

NRCS allows the use of streamflow records to develop the principal spillway hydrograph where a special study shows the adequacy of this procedure for this purpose.

Auxiliary Spillway and Freeboard Hydrographs

Figure 2–2 below, shows minimum auxiliary spillway hydrologic criteria by hazard potential classification.

Evaluate the discharge capacity, stability (surface erosion potential), and integrity (breaching potential) of the auxiliary as follows:

- Analyze both a 6- and 24-hour duration storm. If the drainage area is large and the time of concentration exceeds 24 hours, analyze longer duration storms to ensure evaluation of all significant storm events. Use the most critical results to check the discharge capacity and the integrity of the auxiliary spillway.
- For locations where NWS references provide estimates of local storm, general storm values, and other types of storm values, analyze all.
- Use the short-duration storm to check the stability of vegetated and earth auxiliary spillways.
- In areas without NWS references for spatial distribution, NRCS allows the use of minimum areal adjustment ratios as described in 210-NEH-630-21 for drainage areas greater than 10 square miles.
- In areas without NWS reference for temporal distribution, NRCS allows the use of the dimensionless auxiliary and freeboard storm distribution as described in 210-NEH-630-21, "Design Hydrographs." Alternately, develop the NRCS 24-hour five-point storm distribution by critically stacking incremental rainfall amounts of successive 6-, 12-, and 24-hour durations as described in NOAA Hydrometerological Report 52, "Application of Probable Maximum Precipitation Estimates - United States East of the 105th Meridian."

NRCS allows consideration of special probable maximum precipitation (PMP) studies. NWS or other hydrometeorologists with experience in such work may conduct such studies. Consider special studies for large drainage areas, areas of significant variation in elevation, or areas located at the boundary of two studies where discontinuities occur. Compare the results of the special study with the base NRCS PMP and provide justification of using the special study results if less than the base NRCS PMP.

Use procedures in 210-NEH-630-16 and 630-21, and applicable national computer programs to develop stability design (auxiliary spillway) and freeboard hydrographs. NRCS allows methods outlined in FEMA 94, "Selecting and Accommodating Inflow Design Floods for Dams," as an alternate method to determine the freeboard hydrograph, provided downstream land use controls exist to prevent the voiding of incremental risk assumptions after dam construction.

Figure 2–2: Table of Minimum Auxiliary Spillway Hydrologic Criteria

Class of Dam	Product of storage (AC-FT) × effective height (FT	Existing or planned upstream dams	Precipitation data for[1]	
			Auxiliary spillway hydrograph	Freeboard hydrograph
Low hazard[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 hazard	All	none or any	$P_{100}+0.12(PMP–P_{100})$	$P_{100}+0.40(PMP–P_{100})$
High hazard	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. For dams involving industrial or municipal water, use minimum criteria equivalent to that of a significant hazard potential dam.
3. Applies when the upstream dam is located so that its failure could endanger the lower dam.

Dams in Series—Upper Dam

In a system of dams in series, if failure of the upper dam could contribute to failure of the lower dam, the hydrologic criteria and procedures for the design of the upper dam must be the same as, or more conservative than, those for the dam downstream.

Dams in Series—Lower Dam

In design of the lower dam, use the time of concentration (T_c) of the watershed above the upper dam to develop the hydrographs for the upper dam. Use the T_c of the uncontrolled area above the lower site to develop the uncontrolled area hydrographs. If the T_c for the total area exceeds the storm duration, increase the precipitation amounts for the auxiliary spillway and freeboard hydrographs by the values in NWS references.

In designing the lower dam, take the following steps:

Step 1—Using the hydrologic design criteria for the lower dam, develop the runoff hydrograph for the area controlled by the upper dam.

Step 2—Route the inflow hydrograph for the upper dam through the upper dam. If the upper dam is overtopped or its safety is questionable, it is considered breached or failed and the breach hydrograph from the upper dams is used as the outflow hydrograph for the design of the lower dam.

Step 3—Route the outflow hydrograph for the upper dam to the lower dam.

Step 4—Develop the runoff hydrograph for the intermediate uncontrolled drainage area of the lower (downstream) dam.

Step 5—Combine the routed outflow hydrograph from the upper dam with the hydrograph from the intermediate uncontrolled drainage area.

Use the combined hydrograph based on the principal spillway hydrologic criteria for the lower dam to determine the capacity of the principal spillway and the floodwater retarding storage requirement of the lower dam. Use the combined hydrograph based on the stability design (auxiliary spillway) hydrologic criteria for the lower dam to evaluate the stability (erosion resistance) of any vegetated or earth spillway at the lower site. Use the combined hydrograph based on the freeboard hydrologic criteria for the lower dam to determine top of dam and to evaluate the integrity of any vegetated or earth spillway at the lower dam.

NRCS allows the use of areal reduction factors, determined based on the total drainage area of the dam system, to reduce the minimum precipitation amounts for each of the required hydrographs.

Design of the lower dam should consider the effects of the breach hydrograph from the upper dam routed downstream to the lower dam, combined with the uncontrolled area hydrograph. To mitigate for a potential breach of the upper dam, designers of the lower dam should consider measures such as increased storage, increased spillway capacity, and overtopping protection.

If upon routing a hydrograph through the upper dam, the dam overtops without overtopping protection, or its safety is questionable, NRCS considers the dam highly vulnerable to a breach. NRCS requires increased storage and spillway capacity, without overtopping, for a lower dam below an upper dam highly vulnerable to breach.

Design of the lower dam should consider the potential for the removal of an upper dam or dams during the service life of the lower dam. Removal of an upper dam or dams may increase lower dam design flows. To mitigate for increased design flows associated with the removal of an upper dam, designers of the lower dam should consider measures such as increased storage, increased spillway capacity, or overtopping protection. Develop lower dam designs to accommodate upper dam removal as part of initial dam construction, dam rehabilitation, or as a future enhancement prior to removal of the of the upper dam.

Large Drainage Areas

When the area above a proposed dam approaches 50 square miles, it is desirable to divide the area into hydrologically homogeneous subbasins for developing the design hydrographs. Generally, the drainage area for a subbasin should not exceed 20 square miles. Use watershed modeling computer programs, such as WinTR–20 (the computer program for project formulation hydrology) or SITES (Water Resources Site Analysis Program) for inflow hydrograph development.

If the T_c for the entire drainage area is greater than 24 hours, test storm durations longer than the T_c to determine the duration that gives the maximum reservoir stage for the routed stability design (auxiliary spillway) and freeboard hydrographs.

Precipitation amounts may vary markedly within a large watershed due to topographical and meteorological parameters such as aspect, orientation, mean elevation of subbasin, and storm orientation. Consider the use of a special PMP study for large watersheds with drainage areas more than 100 square miles. Individual watershed PMP studies can consider orographic features typically smoothed in the generalized precipitation studies. Areas where significant snowmelt can occur during the design storms may warrant special studies.

NRCS encourages unit hydrograph development using watershed storage and timing effects, watershed model calibration, and similar studies using watershed stream flow records.

PART 3 – 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. Sediment accumulation in the reservoir can adversely influence operation of the low flow gates and principal spillway as well as reduce conservation and flood storage. Therefore, allocate storage capacity for the calculated sediment accumulation during the design life of the reservoir. Designers should also consider measures to address passing sediment through the dam, capturing sediment in the sediment basin, removal of sediment, or providing storage within the reservoir for sediment.

210-NEH-3, "Sedimentation" contains criteria and general procedures needed to determine the volume required for sediment accumulation and its allocation in the reservoir. It also includes procedures for determining—

- Sediment yield for present conditions and for the future after planned land treatment and the application of other measures in the drainage area of the dam.
- Trap efficiency of the reservoir.
- Distribution and types of sediment expected to accumulate.
- Proportion of continuously submerged sediment versus aerated sediment.
- Densities to which the sediment will become compacted.

If the amount of sediment accumulation calculated exceeds 2 watershed inches in 50 years for the uncontrolled drainage area of the dam, reevaluate the entire watershed to determine the feasibility and applicability of more economical methods of reducing sediment yield or trapping sediment.

When rehabilitating existing dams, use guidance in 210-NEH-3 to perform a sedimentation analysis. In addition, collect the following information on the existing structure:

- Sediment deposition amount and distribution
- Density of deposited sediment
- Proportion of continuously submerged sediment versus aerated sediment.
- Remaining sediment storage available
- Impacts on dam operation
- Projected future sediment deposition

PART 4 – GEOLOGIC AND GEOTECHNICAL CONSIDERATIONS

Investigate site geologic and geotechnical conditions in a manner that adequately examines embankments, spillways, abutments, borrow areas and foundations to enable adequate evaluation of all design conditions. Provide appropriate intensity and detail of these investigations for the class of dam, complexity of site geology, and the data needed for the dam design.

The design engineer, acting as team leader, geotechnical engineer, geologist, and other disciplines requiring geotechnical data, must jointly complete a geotechnical investigation plan prior to initiating geologic investigations. This plan must provide the basis for geologic investigations, and must include the sampling and testing required for—

- Addressing site geologic conditions.
- Specific requirements of the anticipated design at the stage of the subject investigation.
- General requirements of this TR.

Keep an updated testing program in alignment with the geotechnical investigation plan and provide the testing program with samples submitted for testing. Complete a draft report of geologic investigation and provide for use prior to preparing the soil mechanics report. At a minimum, the draft must include a description of key geotechnical and geologic issues at the site, the anticipated design, preliminary profiles and cross sections, and drill holes in stick figure format with field classifications and in-situ test results. Complete the final report of geologic investigation, including field and laboratory classifications, in final format for use in preparing the design.

210-NEM-531, “Geology,” and 210-NEH-631, “Geology,” establish the general requirements, procedures, and criteria for geologic investigations.

Site conditions and dam features that require special attention include, but are not limited to, the following geologic considerations.

Soils with Dispersive Clays

Soils containing dispersive clays are extremely erosive. Dams constructed using dispersive clays are vulnerable to internal erosion failures and require special design considerations. Geologic investigations for soils containing dispersive clays require special sampling procedures. This includes obtaining many discrete samples and preserving the samples at their natural water content.

Karst

Karst terrain requires detailed evaluation of potential subsidence, seepage, and leakage in the dam foundation and reservoir floor. Thoroughly evaluate these issues as they have significant impact on the design, construction, cost, performance, and safety of the structure. Multipurpose structures with permanent water storage in karst areas are especially critical. The geotechnical investigation must determine, and the report clearly state, if there are any foundation materials with dissolution potential.

Collapsible Soils

Evaluate the potential of moisture deficient, low density, unconsolidated materials to collapse on saturation or wetting. NRCS typically encounters collapsible soils in arid and semiarid areas.

Collapsible materials are often associated with deposits, such as alluvial fans, terraces, and aeolian soils. If the potential for collapsible soils exists, perform extensive site investigations and testing to provide quantitative information for design and construction. Obtain and test undisturbed samples that are representative of the collapsible material.

Liquefaction Susceptibility

Soil liquefaction typically occurs in recent deposits of loose sand and silty sand located below the water table; however, gravels and low plasticity silts may also liquefy. Assess groundwater conditions and the occurrence and extent of potentially liquefiable soils that could lose strength under the earthquake shaking considered at a site. Characterize other soils that could undergo loss of strength resulting from earthquake shaking, including sensitive clays, clays and plastic silts potentially susceptible to cyclic softening, and collapsible weakly cemented soils. When developing a subsurface investigation plan, the geologist and geotechnical engineer must consider identifying and locating all layers of potentially liquefiable soils to provide data adequate to analyze the potential significant loss of strength under earthquake loading.

Seismicity and Earthquake Loading

Consider the effects of earthquake loading for all dams. The level of investigation and analysis performed for site characterization and delineation of potential earthquake loading will depend on the potential consequences of seismic dam failure, the seismicity of the site, individual site characteristics that influence the performance of the dam under earthquake loading and the anticipated analysis and design methods.

Geologic investigations must document the existence or absence of active and capable faults at the site. Geologic investigation reports must include a map showing all magnitude 4 or intensity V (near epicenter), or greater earthquakes of record and any active or capable faults within a 100-kilometer (62 mile) radius of the site. Use moment magnitude (Mw) to define earthquake magnitude occurring after the 1970s.

Assess the existence and extent of potentially liquefiable soils and groundwater conditions at sites where the considered earthquake ground motions could cause liquefaction in susceptible materials.

As a minimum, use the peak ground acceleration (PGA) resulting from the maximum design earthquake loading (MDE) corresponding to the annual exceedance probability listed in figure 4–1 below. Obtain PGA and other earthquake characteristics associated with the exceedance probabilities listed in figure 4–1 from the U.S. Geological Survey (USGS), Earthquake Hazards Program. Ground motion values must be representative of the site foundation conditions. If foundation conditions representative of the site are not directly available from the USGS data, adjust the PGA values to represent the actual site conditions.

Figure 4–1: Table of Earthquake Loading – Minimum Peak Ground Acceleration for Seismic Evaluation of Dams and Appurtenances

Consequence of a seismic failure	Exceedance probability for peak ground acceleration (PGA)	Return period (years)	Approximate probability of exceedance in 50 years
Low consequence	1×10^{-3}	1,000	5%
Significant consequence	4×10^{-4}	2,500	2%
High consequence	1×10^{-4}	10,000	0.5%

Consequences from seismic failure based on a failure mode with the reservoir at the normal pool elevation are—

- **Low Consequence**.—Dams in rural or agricultural areas where seismic failure of the dam or appurtenance may result in damage to farm buildings, agricultural land, or township and country roads.
- **Significant Consequence**.—Dams in predominantly rural or agricultural areas where seismic failure of the dam or appurtenance may result in damage to isolated homes, main highways or minor railroads, and interrupt service of relatively important public utilities. For high hazard potential and significant hazard potential dams, significant consequence of a seismic failure of the dam or appurtenance may also include the loss of stored water from water supply dams where there is no other water supply source. For high hazard potential and significant hazard potential dams, significant consequence also includes the loss of the permanent pool where a seismic failure of the dam or appurtenance would have significant consequences such as the loss of important recreational facilities or environment damage.
- **High Consequence**.—Dams where seismic failure of the dam or appurtenance may cause loss of life or serious damage to homes, industrial and commercial buildings, important public utilities, main highways, or railroads.

Designs may use a site-specific seismotectonic study conducted in accordance with FEMA 65, “Earthquake Analysis and Design of Dams,” for significant and high hazard potential dams in lieu of using the loadings shown in figure 4–1. If earthquake parameters for high hazard potential dams are determined from a deterministic seismic hazard assessment, base parameters on mean plus one standard deviation results. Documentation of site-specific study results will include comparison of the proposed earthquake loading resulting from using the most recent version of the USGS Interactive Deaggregation Program data at the annual exceedance probability listed in figure 4–1 and the rationale for selection.

Design requirements and the methods used to evaluate earthquake effects on the dam may require the use of additional earthquake parameters to evaluate earthquake effects on the dam. Additional parameters may include magnitude and distance of features contributing most of the seismic risk, recommended design earthquakes magnitude and distance, acceleration time histories, spectral response, amplification factors, or other parameters. Document all methods and assumptions incorporated into the development of the earthquake loading and details that are required to apply the results appropriately in the dam analysis.

As a minimum, use the peak ground acceleration (PGA) resulting from the operating basis earthquake (OBE) loading corresponding to the annual exceedance probability listed in figure 4–2 for post-earthquake operational evaluation of dams. Part 5 describes the requirements for this evaluation.

Figure 4–2: Table of Seismic Requirements for Postearthquake Operational Evaluation of Dams

Class of Dam	PGA annual exceedance probability	Return period (years)	Approximate probability of exceedance in 50 years
Low hazard potential	----	----	----
Significant hazard potential	4×10^{-3}	250	20%
High hazard potential	2×10^{-3}	500	10%

Evaluate the potential for earthquake-generated seiche wave to affect the dam or appurtenances.

Auxiliary Spillways

Evaluate all earth materials beneath a vegetated or earth auxiliary spillway down to the elevation of the downstream floodplain or valley floor in enough detail to determine excavation requirements, suitability of spillway excavation for embankment fill and to determine stability and integrity of the spillway. Highly erosive material, such as dispersive clays, silts and sands, and material that will require special processing for use as fill, require investigation that is more detailed. 210-NEH-628-52, "Field Procedures Guide for the Headcut Erodibility Index," includes guidance on evaluation of headcut erodibility.

Mass Movements

Evaluate landslides and landslide potential at dam and reservoir sites, especially those involving shale formations, clay layers, and those where unfavorable dip-slope or other adverse rock attitudes occur. Summarize the history of mass movement in the project area. Carefully evaluate the static and seismic stability of dam abutments, auxiliary spillway cuts and the reservoir rim. If sudden movements are possible, evaluate the effect of the potential landslide on displacement of reservoir water and resulting wave effects.

Subsidence

Consult the State geological survey and data sources to investigate the potential for surface subsidence due to past or future solid, liquid (including groundwater) or gaseous mineral extraction. 210-NEM-531 provides specific requirements.

Multipurpose and Water Retention Dams

Investigate and evaluate the groundwater regime and hydraulic characteristics of the entire reservoir area of water storage for potential leakage. Develop and analyze water budgets to assure the adequacy of the site to accomplish the intent of the project.

Other

Special studies and evaluations may be necessary where such conditions as highly fractured bedrock (such as basalt or rhyolites); compacted shales; some types of siliceous, calcareous, or pyritic shales; expansive soils; soluble salts; vertic soils; mirabilite; rebound or stress relief fractures (that may cause open fissures); sharp elevation changes in foundation materials; or artesian waters occur at a site.

PART 5 – EARTH EMBANKMENTS AND FOUNDATIONS

The earth embankment and its foundation must be stable and withstand all anticipated loads without movements leading to failure. Provide measures for adequate seepage control under all anticipated loads.

Height

Set the crest of an earth embankment at an elevation adequate to prevent overtopping during passage of the freeboard hydrograph. Evaluate the potential need for additional freeboard using the larger of the freeboard required for wave action or the freeboard required for frost conditions. The dam crest elevation must be above the higher of either auxiliary spillway hydrograph plus wave action or auxiliary spillway hydrograph plus frost conditions. Document the possible effects of wave action and frost conditions on the safe operation of the dam.

The dam crest evaluation must also allow for the minimum auxiliary spillway depth as described in part 7. Increase the dam crest elevation by the amount needed to compensate for settlement.

Top Width

Figure 5–1 shows the minimum embankment top width.

Increase the top width from the above minimums to—

- Meet State and local standards.
- Accommodate embankment zoning.
- Provide roadway access and traffic safety.
- Provide structural stability.

When raising an embankment for rehabilitation of previously constructed embankment dam, the top width may be no more than 10 percent less than listed in figure 5–1 if analysis and design for seepage, stability, constructability, access for maintenance, and long-term reliability justifies a narrower width.

When embankment top serves as a public roadway, use the minimum width of 16 feet for one-way traffic and 26 feet for two-way traffic. Use guardrails or other safety measures meeting the requirements of the responsible road authority.

Figure 5–1: Table of Minimum Top Width of Embankment

Overall height of embankment, H, (ft)	Minimum top width (ft)		
	All dams	Single-purpose floodwater retarding	Multipurpose or other purposes
<15	8	N/A	N/A
15<19.9	10	N/A	N/A
20<24.9	12	N/A	N/A
25<34.9	14	N/A	N/A

Overall height of embankment, H, (ft)	Minimum top width (ft)		
	All dams	Single-purpose floodwater retarding	Multipurpose or other purposes
35<94.9	N/A	14	(H+35)/5
95.0+	N/A	16	26

Embankment Slope Stability

Analyze the stability of embankment slopes using generally accepted methods based on sound engineering principles. Document all aspects of the analyses performed. Documentation should include analysis program and methods used, input data, and tabular and graphical output. Document shear strength parameters used for each zone of the embankment and each soil type or horizon in the foundation and the basis for these parameters. Document piezometric assumptions included in the analyses and any other variables that will influence stability, along with the basis for the values. The stability analysis should incorporate zones of saturation and seepage pressures consistent with the results of the seepage analysis. Identify and incorporate features necessary to achieve required safety factors in the design.

The extent and intent of the site investigation, soil testing program, and analyses should be consistent. The level of investigation, testing, and analysis should be commensurate with the complexity of the site and structure, along with the consequences of failure. Minimum required static stability safety factors are summarized in figure 5–3 for each condition analyzed.

This TR does not address all potential situations that may arise in the embankment dam stability analysis. When using approaches outside the scope of this TR, document references for all analysis methods used. Include justification for the methodology used, the basis for material characterization and pore pressure assumptions, and comparison of the analysis results with analysis methods included in this technical release.

Consider the following guidance in analyzing slope stability for all conditions:

- Evaluate the overall stability of the embankment and foundation. Determine the stability of critical slip zones (zones that encompass a significant portion of the embankment, pass through major portions of the embankment and foundation, and zones that include the crest), and identify slip surfaces with the overall minimum factors of safety. Document the location of slip surfaces evaluated relative to the location of comparatively weaker materials and areas of high pore pressure, to assure the evaluation of zones that have a propensity to control stability.
- Use effective stress parameters for soils that will drain as rapidly as load is applied. These parameters apply for all conditions of stability analyzed for these soils.
- Limit the use of infinite slope equations and associated safety factors listed in figure 5–3 to the stability of near surface failure surfaces in the exterior slope of embankments with cohesionless soils, when the critical failure is wholly within soils of this character, and analysis of deeper failures would result in higher factors of safety. Infinite slope equations should model the predicted seepage pattern in the slope under analysis. Different equations apply for no seepage, horizontal seepage paths, and seepage paths parallel to the slope face. If shallow slope failure

could progress and affect deeper embankment zones, use the higher safety factors listed in figure 5–3. If cohesionless zones occur with other soil types in analysis of a cross section, use other methods to analyze circular arc or wedge-shaped surfaces that intersect the soil zones with cohesion to locate the minimum safety factor.

- Use residual effective stress parameters for modeling slope stability analyses involving fissured clays or shales if preexisting movements have occurred. Drained shear strength tests provide the basis for these parameters. Use residual parameters for designing against shallow slope failures in desiccated clay embankments with previous movement. Designers can use fully softened shear strength with conservative pore pressure assumptions to evaluate the potential for first time shallow slope failures in compacted high plasticity clay embankments.

Static Stability

Analyze embankment stability for each of the following conditions in the structure life that are appropriate to the site. Clearly document the reasons for not analyzing a condition. Document the source of all shear strength parameters used. When using a shear strength parameter determined from an empirical correlation, include correlations to field performance and document the justification for use of the empirically based value rather than site-specific sampling and testing. Include discussion on the sensitivity of the outcome of the analysis to the variability of the parameter. Stability analyses must identify and incorporate any lower strength materials that may control stability.

Stability During Construction

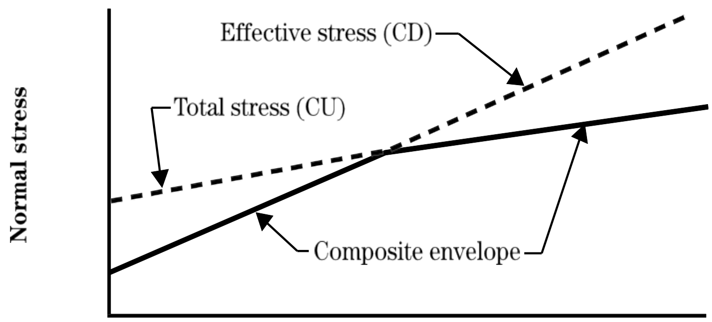
The construction sequence may control the configuration of the embankment and rate of construction when analysis predicts that embankment soil, foundation soils, or both will develop significant pore pressures during embankment construction. Evaluate site conditions, such as soft clay layers in the foundation that may require special designs. Consider flattened slopes and sequentially staged construction to achieve stable conditions. Factors determining the likelihood of this occurring include the overall 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 figure 5–3. Consider the highest likely placement water content of embankment soils in the shear-testing program. Alternatively, designers can use effective stress shear parameters to establish limits for pore pressure buildup during construction. The design should include procedures for monitoring pore pressures during construction to assure that actual pore pressures do not exceed limits established during design.

Construction stability analysis must include stability of excavation slopes that could have significant safety or construction consequences, such as excavation of existing embankment slopes during rehabilitation construction.

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 normal full reservoir level from which a phreatic line is likely to develop to the elevation of the lowest gated outlet. If the potential exists for saturation of zones in the upstream slope during temporary flood retention storage above the permanent normal pool, consider the effect of drawdown back to the normal pool on that saturated portion of the embankment. Designers may use transient seepage analysis to estimate the short-term saturation of the portion of the embankment above permanent normal pool. Select shear strength parameters for the analyses according to figure 5–3, as illustrated in figure 5–2.

Figure 5–2: Mohr-Coulomb Envelope for Upstream Drawdown



In cases where rapid drawdown slope stability is critical to embankment geometry or zoning, or for modifications to a previously constructed embankment, designers can also analyze rapid drawdown using procedures included in U.S. Army Corps of Engineers (USACE) Engineer Manual (EM) 1110-2-1902, Appendix G. Documentation should include the difference in analysis results between the methods and justification to use the selected results.

Steady Seepage

Analyze downstream slope stability of new or previously constructed dams under steady-state seepage conditions. Perform the analysis with the reservoir elevation at the highest normal pool level. Use shear parameters as listed in figure 5–3. Base the location of the steady-state phreatic surface on the highest normal reservoir pool elevation. Designers can develop the phreatic surface for the analyses using computer programs for seepage analysis, flow nets, or Casagrande procedures.

Existing embankments may not have achieved steady-state conditions. Thus, use the calculated long-term high-level steady-state seepage condition for both new and existing embankments. For existing embankments, compare the computed phreatic surface with field performance and field measurements, when available, to assure that the computed phreatic surface meets or exceeds that which has occurred over the life of the structure.

Include a separate foundation phreatic surface as appropriate at sites with limited foundation seepage cutoff, particularly in sites with confined seepage that results in uplift at the downstream toe.

If steady-state seepage stability is highly dependent on the success of internal drainage features to control the phreatic surface, check stability assuming partially functioning or plugged drainage systems.

For appropriate slopes, designers can use infinite slope equations and applicable safety factors listed in figure 5-3.

Flood Surcharge

Analyze downstream slope stability of new or previously constructed dams under flood detention conditions. Perform the analysis with the reservoir elevation at the freeboard hydrograph pool level. Use shear parameters as listed in figure 5–3.

Develop a phreatic surface resulting from the maximum reservoir elevation. Consider material properties and the potential for defects in the upper portion of the embankment when selecting the appropriate seepage analysis and evaluating seepage conditions that could develop. Also consider the potential for increase in pore pressures in the normally saturated portion of the foundation or embankment that may result from the higher reservoir loading. Evaluate a range of potential seepage conditions to determine the sensitivity of flood storage stability to potential seepage conditions.

If short term flood storage stability is highly dependent on the success of internal drainage features to control the phreatic surface, check stability assuming partially functioning or plugged drainage systems.

For appropriate slopes, designers can use infinite slope equations and applicable safety factors listed in figure 5-3.

Figure 5–3: Table of Static Slope Stability Criteria

Design condition	Primary assumption	Remarks	Applicable shear strength parameters	Minimum safety factor
1. Construction Stability (upstream or downstream slope)	Zones of the embankment or layers of the foundation expected to develop significant pore pressures during construction	Low-permeability embankment soils should be tested at water contents that are as wet as likely during construction (usually wet of optimum). Saturated low permeable foundation soils not expected to consolidate fully during construction. Existing dams with additional fill placed above saturated low-permeability zones.	Unconsolidated; Total stress consistent with preconstruction stress state	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
	Embankment zones and/or strata not expected to develop significant pore pressures during construction	Embankment zones, foundation strata, or both comprised of material with a permeability high enough to drain as rapidly as they are loaded	Effective stress	
2. Rapid drawdown (upstream slope)	Drawdown from the highest normal pool to the lowest gated outlet	Consider failure surfaces both within the embankment and extending into the foundation Low-permeability embankment and foundation soils that will have limited drainage during reservoir drawdown Embankment zones, foundation strata, or both comprised of material with a permeability high enough to drain as the reservoir is drawn down	Lowest of effective stress or consolidated; total stress; consistent with predrawdown consolidation stresses (See fig. 5-1) Effective stress	1.2 1.1 for near surface (infinite slope) failure surfaces in cohesionless soils
3. Steady seepage	Reservoir water surface at highest normal pool. Phreatic surface developed from the highest normal pool; typically the principal spillway crest	Consider failure surfaces within both the embankment and extending into the foundation. Foundation analysis may require separate phreatic surface evaluation, particularly in sites with confined seepage that results in uplift at the downstream toe.	Effective stress	1.5 1.3 for near surface (infinite slope) failure surfaces in cohesionless soils
4. Flood surcharge	Reservoir at freeboard hydrograph level. Steady seepage phreatic surface incorporating increased pore water pressure that may occur from flood detention and pore water pressure from short term seepage resulting from reservoir surface above the normal pool elevation	Consider failure surfaces within both the embankment and extending into the foundation. Embankment zones, foundation strata, or both comprised of material with a permeability high enough to drain rapidly with changes in reservoir elevation Low-permeability embankment and foundation soils that will have limited drainage as the increased reservoir load is applied	Effective stress Lowest of effective stress or consolidated; total stress (See fig. 5-1)	1.4 1.2 for near surface (infinite slope) failure surfaces in cohesionless soils

Figure 5–4: Shear Strength Parameters, Associated Tests, and Nomenclature

Shear strength parameter	Types of shear tests	NRCS notation	Other common abbreviation	Notes	American Standard for Testing Materials (ASTM) test standard
Total stress; unconsolidated[1]	Unconsolidated undrained triaxial	UU	Q		D2850
	Unconfined-compression	qu		Suitable for limited types of soils[2]	D2166
	Field vane shear			Limited to saturated low strength (Su<2 tsf) clays and silts. Field vane shear test results must be corrected for plasticity index to determine undrained strength	D2573
Total stress; consolidated [1]	Consolidated undrained triaxial	CU	R	ASTM standard for cohesive soils	D4767
Effective stress	Consolidated undrained triaxial with pore pressure measurements	$\overline{CU}$	R	Determine effective stress parameters by subtracting measured pore pressures from total stresses.	D4767
	Consolidated drained triaxial[3]	CD	S		D7181
	Direct shear			Strain rate must be limited to the extent required to allow drainage	D3080
Residual effective stress	Ring shear			Residual shear strength of cohesive materials containing preexisting shear surfaces	D6467
				Fully softened shear strength of cohesive materials without preexisting shear surfaces.	D7608
	Repeated direct shear			No ASTM standard for soils	

1. Using total stress parameters in the analyses model generates pore pressures (positive or negative) in laboratory specimens that simulate the pore pressures that will develop in the field soils during loading. Total stress parameters used in developing composite shear strength envelopes should not consider large negative pore pressures that may result from shear testing. Take the stress condition at failure to correspond to either maximum deviator stress or maximum principal effective stress ratio to define total stress parameters. Only use the stress conditions at a target axial strain to define failure when more limiting than other criteria.
2. The test for unconfined compressive strength (qu) applies only to cohesive materials that will retain strength without confining pressure and will not expel water during testing. This test does not apply to dry, crumbly, fissured, or non-cohesive soils or other soils that will not hold their shape without confinement. Some soils will exhibit higher strengths when tested using the unconsolidated undrained (UU) test. See ASTM D2166.
3. NRCS generally performs consolidated undrained with pore pressure measurements ($\overline{CU}$) tests rather than consolidated drained (CD) tests, as a more rapid strain rate with the CU test reduces the time required to complete the test. With appropriately performed tests, effective shear strength parameters obtained from CU *bar* tests should be equivalent to those from CD tests.

Dynamic Stability

Evaluate the effects of earthquake loading for all dams.

Analysis of dams for earthquake loading must address all potential failure modes. Distress of embankment dams from earthquake loading primarily manifests itself in deformation causing crest settlement, lateral spreading, cracking and differential displacements between the embankment or foundation and appurtenant structures.

Limit embankment and foundation deformation resulting from the design earthquake to an amount that will not result in embankment overtopping from crest settlement, internal erosion from cracking or damage to appurtenances to the extent that could result in dam failure. Dam failure is instability or major damage that may lead to uncontrolled loss of the reservoir when subjected to the design earthquake loading.

Limit damage from the operating basis earthquake to the extent that, after the earthquake and prior to repair, the dam and appurtenances will pass the principal spillway flood without failure.

Design appurtenant structures such that earthquake-caused damage to structures will not lead to dam failure. Evaluation of appurtenances must consider damage that could result from embankment or foundation permanent deformation, in addition to transient seismic forces.

Evaluate the potential for fault movement in the dam foundation and the stability of the reservoir rim.

Do not locate new dams on active faults. Do not locate new significant or high hazard potential dams on capable faults, without specific design features that address potential fault movement.

Existing dams located on active faults must be able to withstand the potential fault offset that could result from the design earthquake, without failure.

Existing dams with high consequence from a seismic failure, located on capable faults, must withstand the potential fault offset resulting from the design earthquake, without failure.

Existing dams located on active faults must withstand the potential fault offset resulting from the operating basis earthquake and must, prior to repair following an earthquake, pass the principal spillway flood without failure.

Design and construct or rehabilitate embankment dams to resist earthquake loading with sound defensive measures representative of the current practice and commensurate with the seismicity of the site, site geology, and hazard of the structure.

Analysis

Seismic analyses should begin with the simplest and most conservative method to analyze the embankment and foundation. If these analyses show that the structure will perform adequately under the design earthquakes, then further analyses are unnecessary. If further analyses are needed, the analyses must be progressively more detailed and complex. Provide a determination of earthquake loading, site characterization and determination of material properties consistent and appropriate for the level of detail and complexity of the analyses.

Sites where the design peak horizontal ground acceleration (PGA) determined in accordance with part 4 of this TR is equal to or less than 0.07 g require no seismic analysis. This low level of earthquake loading should not significantly distress embankment dams, even with site conditions susceptible to damage from earthquake loading.

If the design ground motion exceeds 0.07 g, evaluate the potential for loss of shear strength due to liquefaction or cyclic failure under seismic loading.

For seismic analyses, assume that the reservoir is at the highest normal pool elevation. Base the extent of saturation of embankment and foundation materials on the steady-state seepage conditions prior to earthquake loading resulting from the same pool elevation. Initial embankment and foundation properties used in the analyses must represent existing conditions prior to earthquake loading. Consider both upstream and downstream failure.

The characteristics and extent of soils that could undergo an extensive loss of strength or liquefy in the foundation and embankment must be determined. Determine the characteristics and extent of sensitive clays, clays, and plastic silts that, under the considered earthquake loading, may be susceptible to cyclic softening, and collapsible weakly cemented soils at the site.

If these materials are or may be present at the site, characterize and analyze the site as a site with the potential for significant loss of strength under earthquake loading.

If these materials are not present, characterize and analyze the site as a site with limited loss of strength under earthquake loading.

Sites With Limited Loss of Strength Under Earthquake Loading

Well-built embankment dams on rock or dense soil—particularly clay—foundations generally perform well under earthquake loading. Well-built dams consist of well-compacted earth or earth and rock fill, with adequate static factors of safety, seepage control and freeboard, constructed under controlled conditions to ensure achievement of the design intent. For guidance, see Federal Emergency Management Agency (FEMA) 65, “Federal Guidelines for Dam Safety: Earthquake Analyses and Design of Dams.”

Embankments possessing these favorable characteristics, where the peak horizontal acceleration at the base of the embankment from the design earthquake is 0.20 g or less, do not require special seismic analysis.

Dams that do not have these characteristics, or where the design earthquake loading is higher, require additional analyses. Analyses should progressively increase in scope and complexity, based on data commensurate with the level of analysis. The analyses must proceed to the extent required to determine conclusively that the dam would not fail as defined above under earthquake loading. The analysis should document that the expected performance for new and existing embankments is adequate or that existing dams can achieve adequate performance by incorporating remedial measures. The analyses must include assessment of the vulnerability of any feature or appurtenance to damage from embankment deformation. This must include an evaluation of the potential of dam failure from a damaged feature or appurtenance. As a check on analysis results, compare estimated performance to observed performance of similar structures under equivalent earthquake loading.

Sites With the Potential for Significant Loss of Strength Under Earthquake Loading

The most substantial adverse effects of earthquakes on embankment dams have typically occurred in situations where there has been a significant reduction in strength in the embankment, foundation materials, or both due to liquefaction. The evaluation of sites with materials that will lose strength during earthquake loading requires—

- Determining the potential for the design earthquake to result in loss of strength of embankment or foundation materials.

- Evaluating the stability of the foundation and embankment with reduced strengths.
- Estimating the potential amount of deformation.

Evaluate the postearthquake static stability of the embankment and foundation under gravity loading if loss of strength of embankment or foundation material is expected. Use strengths characteristic of foundation and embankment materials that would exist after earthquake loading. Differentiate materials to the extent possible with available data; it is particularly important to evaluate the continuity of low-strength zones. Include analysis of upstream and downstream failure surfaces that involve the crest. Although these might not have the lowest factors of safety, they have the greatest potential for reservoir release. Evaluate the potential for progressive failure.

Conduct parametric studies to evaluate the sensitivity of the stability of the structure to assumptions regarding the extent of low strength materials and the strengths assigned. The required minimum factor of safety for critical surfaces, including surfaces that could lead to progressive failure, for postearthquake static stability is 1.2.

Achieving minimum factors of safety in the postearthquake static stability analysis does not necessarily indicate an acceptable level of deformation from earthquake accelerations. Evaluate deformation as discussed for sites with limited loss of strength. Use strengths of foundation and embankment materials as altered by earthquake loading if strength reduction is expected during earthquake shaking.

If postearthquake static stability or expected embankment deformation are not acceptable, remediate low-strength materials. Consider removal of susceptible material, densification of susceptible material, additional design elements to increase stability and reduce or accommodate deformation potential, relocation of the site, or other proven treatment methods. Document alternatives considered and the basis for any remediation alternative selected.

This TR does not address all potential situations that may arise in the analysis and improvement of embankment dam stability. If using approaches outside the scope of this TR, provide reference for all analysis methods used. In addition, provide justification for the methodology used, analysis inputs, results, and design of features incorporated because of the analysis.

Seepage

Evaluate the effects of seepage for all dams. The evaluation must address all potential embankment and foundation seepage related failure modes, including the potential for internal erosion, erosive flow along defects, internal instability, and uplift pressures to damage the embankment, its foundation, and appurtenant structures. The evaluation should be commensurate with the complexity, function, and hazard potential classification of the structure. Seepage control and management must be adequate to accomplish the intended reservoir function, provide a safely operating structure, and prevent damage to downstream property.

Design and construct or rehabilitate existing embankment dams with sound defensive measures to reduce, filter, collect, and discharge seepage that are representative of current practice.

Analysis

Seepage analyses should begin with the simplest and most conservative method to analyze the embankment and foundation. Perform progressively more detailed and complex analysis to the extent required, establishing that the design features will safely reduce, intercept, filter, and discharge seepage.

One or more of the following methods of analysis, listed in order of increasing complexity or other industry standard practices may be required to determine if the structure will safely accommodate seepage.

- **Qualitative Methods**.**—**For some simple structures with favorable conditions, the seepage performance of the structure can be satisfactory predicted with limited quantitative analysis. The seepage evaluation for these structures should address the same factors as for more complex analyses. Designers can develop seepage control features for these structures using standard practices based on experience and history of satisfactory performance with similar conditions. The analysis should clearly convey the rational used in the analysis and the justification for the seepage control features incorporated into the design.
- **Analytical Methods**.**—**Designers may use closed-form solutions or approximate solutions for simpler seepage conditions associated with dams.
- **Graphical Methods**.**—**Designers may draw flow net analyses by hand or may use adaptations from numerical solutions. Designers may use flow nets to estimate seepage quantities, evaluate the exit gradient for embankments and foundations and to compute uplift pressures acting on appurtenant concrete structures or soil blankets.
- **Numerical Methods**.**—**Use computer based numerical methods to analyze seepage in cases where graphical solutions are not practical, there is complex stratigraphy or there is a need to account for saturated and unsaturated or transient flow. Designers can use these methods to estimate seepage quantities and pore water pressures and evaluate the exit gradient for embankments and foundations, as well as uplift pressures acting on appurtenant concrete structures or soil blankets.

Material Properties

A seepage evaluation’s replication of actual conditions depends on how well the geometry and material properties assumed reflect actual conditions. Embankment zones and foundation strata should be included in the analysis to the extent required to represent seepage conditions.

Use the geologic interpretation of foundation conditions to determine which foundation strata to consider in analyses. Base the foundation strata on the geologic interpretation of the foundation conditions. Evaluate and consider discontinuities in foundation materials and preferential flow paths in the analyses. In design, evaluate and address potential failure modes accounting for erodible or soluble materials.

Determine hydraulic properties, such as saturated hydraulic conductivity, anisotropy, water content and unsaturated hydraulic conductivity functions, for each zone from specific testing, empirical relationships based on index properties or from industry standard practice for similar materials.

Analysis documentation should clearly convey the geometry and material properties assumed in the analyses and the basis for these assumptions.

Design Requirements

Determine seepage reduction, filtration, collection and drainage measures to address seepage based on sound practice of defensive dam design, the potential for unknown foundation conditions or deficient construction and, where applicable, minimum factors of safety. Compute safety factors using an assumed reservoir head set at the freeboard hydrograph reservoir water surface. Provide measures to the extent required to ensure a safe, functional structure.

Design seepage reduction measures to limit seepage and embankment saturation as necessary to address seepage failure modes, provide adequate static and dynamic stability and limit water loss to the extent required by project function.

Provide seepage integrity for all reservoir stages up to the freeboard hydrograph water surface. The impervious core of zoned embankments should typically extend to the elevation of the reservoir water surface during passage of the freeboard hydrograph. As a minimum, the impervious core of zoned embankments must extend above the above the elevation of the stability hydrograph reservoir water surface, accounting for settlement. Document that embankment zones, filters, or other measures included in the upper portion of the embankment adequately address anticipated reservoir storage and associated seepage conditions.

Filtration and drainage features must be included in all significant and high hazard potential embankments. Filters and drains must include chimney filters and drains that extend high enough in the embankment to intercept seepage from usual and unusual loading conditions and potential defects that could result from faulty construction, cracking resulting from differential settlement, foundation settlement upon wetting, hydraulic fracturing, desiccation, frost heave, or separation that could occur at embankment-appurtenance contacts. Provide filtration and drainage for embankment zones adjacent to foundation strata that are not filter compatible and for foundation materials susceptible to seepage failure. Provide filter and drainage measures at locations where seepage exits to a free face

Determine gradation of filter and drain material using criteria presented in 210-NEH-633-26, “Gradation Design of Sand and Gravel Filters.”

Designers can establish rationale for lesser filter and drain protection for rehabilitation of existing embankments, based on favorable site conditions and a satisfactory seepage history. Most existing structures have not experienced the maximum design reservoir elevation; thus, designers must support assumptions that seepage performance to date represents probable performance at higher water surface elevations in the reservoir. The designer must provide well-documented justification for reduction in filter protection, and resultant level of defensive design.

Do not reduce filter requirements for rehabilitation of existing embankments for the following:

- Embankments with a history of high phreatic surface or seepage exiting the downstream face of the embankment
- Embankments with a history of seepage exiting the foundation and abutments in the immediate area of the embankment under circumstances that could adversely affect internal stability of foundation materials or the embankment foundation contact
- Embankment or surrounding soils containing dispersive clays, highly erodible soils such as silts,

sands, lean clays, and mixtures that have a PI less than 15 or soils that are broadly graded or gap graded, and potentially internally unstable

- Embankments with a residual potential for settlement of foundation materials upon wetting
- Embankments in arid environments susceptible to desiccation cracking
- Embankments in areas susceptible to impervious zone damage from frost penetration
- Embankments where earthquake induced cracking is an identified failure mode

For embankments of all hazard potential classifications with penetrations not protected by comprehensive filter and drainage features, include a filter drainage diaphragm around any structure that extends through the embankment to the downstream slope. Design the filter diaphragm in accordance with 210-NEH-628-45, “Filter Diaphragms.”

Seepage collection and discharge systems, including structural drainage systems, must include access points. Construct these systems in a manner that permits inspection and cleanout of drainpipes. Collection systems should include sediment traps and seepage measurement devices such as weirs or flumes, unless excluding these features is justified in design documentation.

Document that all personnel must follow current Occupational Safety and Health Agency (OSHA) rules and regulations when accessing any feature intended for personnel entry.

Limit pipes in embankment drain systems to locations near the toe of the embankment that allow for required repair or replacement. Limit toe drain pipe to types recommended in report U.S. Bureau of Reclamation (USBR) Dam Safety Office (DSO)-09-01, “Physical Properties of Plastic Pipe Used in Reclamation Toe Drains” (September 2009).

Evaluate heave at sites where vertical exit gradients in cohesionless soils exist downstream of the dam toe. These soils require a minimum safety factor of 4.0 for vertical exit gradients. In designs where this condition exists and that do not meet this minimal factor of safety, include filter drainage measures and filter berms.

Evaluate uplift at sites where a blanket-aquifer condition exists. Minimum acceptable factors of safety are 3.0 for a blanket-aquifer condition in soil when computed by the effective stress method or 1.5 when calculated by the total stress method. Natural blankets are typically not uniform or homogeneous. If a natural blanket is relied on to meet minimum factors of safety, thoroughly investigate the integrity of the blanket and document blanket conditions. In designs where this condition exists and that do not meet the minimal factor of safety, include drainage measures and berms if necessary. For guidance, see USBR Reclamation Design Standards No.13, “Embankment Dams,” Chapter 8, “Seepage” (October 2011).

This TR does not address all potential situations that may be arise in the analysis and treatment of seepage in embankment dams. If using approaches outside the scope of this TR, provide reference for all analysis methods used. In addition, provide justification for the methodology used, analysis inputs, results, and design of features incorporated because of the analysis.

Geosynthetics

NRCS restricts the use of geosynthetics in embankment dams to applications where alternatives for dam rehabilitation are limited or where geosynthetics provide significant cost or constructability benefits over the use of natural materials. Do not use geosynthetics as an individual element to perform functions critical to the safety or satisfactory performance of embankments or foundations.

Designers must document the reasoning for geosynthetic use in lieu of natural materials. Design documentation should include an assessment of the outcome of geosynthetic failure on the performance of the structure should this occur. Geosynthetic design should consider the potential for material defects, construction damage, and consider the long-term consistency of fundamental properties. Instructions to the engineer in the design summary for projects incorporating geosynthetics must include key vulnerabilities of the materials used and construction methods required to reduce potential installation damage to that assumed in the design.

Typically, do not use geosynthetics in locations that are not accessible for inspection and replacement without removing substantial and critical parts of the structure. Designers can establish rationale for use of geosynthetics in areas lacking technically or economically viable alternatives if analysis documents adequate performance of the system for the design life of the structure. Such analysis should evaluate all foreseeable material defects, the potential for construction damage, and effects of long-term deterioration of the geosynthetic.

Potential functions of geosynthetics include filtration of soil particles, planar drainage (transmission) reinforcement, separation of dissimilar materials, providing a water barrier, surface stabilization, and earthfill reinforcement.

Use granular filters, drains, and pipe envelopes designed in accordance with 210-NEH-633-26 for filtration and drainage of seepage water. Designers can use geosynthetics as a component of choke filters where drainage is not a design parameter and permeability is not of concern. Do not use geosynthetics as a filter medium individually or as a component of filter-drain systems requiring both filtration and drainage.

In dam rehabilitation, designers can use geosynthetics as a system component along with granular filter material in functions requiring transmission of seepage flow within the plane of the geosynthetic. Such designs require adequate capacity of the system under normal loading without the geosynthetic and mineral material filtering of all water entering the geosynthetic. Do not use geosynthetics in new dam construction in functions requiring transmission of seepage flow within the plane of the geosynthetic.

Designers can use geosynthetics to improve slope stability in embankment rehabilitation if factors of safety without the geosynthetic are $\geq$1.1 and are within 10 percent of the required factors of safety included in figure 5-3. Factors of safety with the geosynthetic must meet or exceed the requirements of figure 5-3. Do not use geosynthetics as reinforcement for new dam construction.

Designers can use geosynthetics to provide a separator function in the construction of embankment components where flow across the geotextile is not required, and where accessible for repair or replacement. Designers can use geosynthetics for riprap bedding under rock riprap used for wave or water erosion protection on upstream slopes, downstream channels, or stilling basins. Do not use geosynthetics for separation of dissimilar materials within the body of embankments or to encapsulate coarse fill in foundation excavations that lack filter compatibility with surrounding material.

In dam rehabilitation, designers may use geosynthetics as a component of the seepage barrier provided the geosynthetic does not serve as a sole impervious component. Do not use geosynthetics as a seepage barrier in new dams.

Designers can use geosynthetics as a component of embankment surface erosion protection if the design and O&M plan conveys the associated surface-use restrictions and maintenance requirements.

Zoning

Zone embankments to obtain a stable structure with the most economical use of available materials to control seepage in a safe manner or to reduce the uncertainties of material strengths and resultant stability. To the extent possible, increase the permeability of each zone toward the outer slopes. Include transition zones between zones of dissimilar materials so that materials from one zone do not migrate into the voids of adjoining zones by steady-state seepage, rapid drawdown seepage forces, or surface infiltration.

Design embankment zones a minimum of 10 feet wide, except for filters and drains with specified and controlled gradation. Design drains and filters wide enough to allow efficient and reliable construction, provide required filtration and drainage, and to provide resiliency from potential movement. Drains and filters must meet the requirements contained in 210-NEH-633-26.

Use soil materials that exhibit significant shrinkage, swell, or dispersion with extreme care. If possible, do not use these materials for embankment construction. To minimize detrimental effects on the dam, place these materials in zones less susceptible to failure from an operational incident or maintenance problems. When there is no alternative to their use, use soil amendments to treat them to improve their performance. Use filters, drains, and self-healing transition zones to protect these materials.

Surface Protection

Protect embankment surfaces against surface erosion from reservoir wave action or precipitation on the surface of the embankment and associated runoff. Protection may be vegetative, gravel, rock riprap, soil cement, structural, or similar treatment of durable quality and proven satisfactory performance.

Vegetative Protection

Use vegetative protection on surfaces meeting the following conditions:

- Inundation of the surfaces is of such frequency that will not inhibit vegetative growth.
- Vigorous growth sustainable under average climatic conditions by normal maintenance without irrigation.
- Stable protection designed according to the procedures in TR 210-56, “A Guide for Design and Layout of Vegetative Wave Protection for Earth Dam Embankments.”

Structural Protection

Provide protection against wave erosion by riprap or other structural measures for—

- Dams where vegetation will not provide effective control.
- Multiple-purpose dams.
- 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. Base the upper limit on an analysis of anticipated wave height and runup.

When using structural measures, provide a quality of riprap and other structural protection consistent with the anticipated life of the dam and provide a structurally stable design. For guidance on design of riprap for this application, refer to TR 210-69, “Riprap for Slope Protection against Wave Action.”

Observation and Instrumentation

Include requirements for visual observation of all dams in O&M plans. Include instrumentation for significant and high hazard potential dams that allow for comparison of performance to design assumptions and for evaluating dam safety. As a minimum, include instrumentation to monitor surface movement of embankment and structures and to measure pore pressures. Include instrumentation to the extent adequate to represent the entire structure. Include additional instrumentation to monitor other parameters as appropriate to the individual site. Include specific requirements and responsibilities for instrumentation data acquisition, reduction and evaluation in O&M plans.

Parapet Walls

NRCS only allows the use of parapet walls to prevent overtopping from wave runup and reservoir setup above the PMF water surface for rehabilitation of previously constructed embankment dams. Do not use parapet walls in the design of new dams or for temporary retention of flood surcharge during passage of the freeboard hydrograph.

Parapet walls must not exceed 10 percent of the overall embankment height without the parapet or a maximum height of 5 feet. Design parapet walls for internal and local stability under the maximum design loading. Design the overall stability of the parapet and embankment to meet the applicable requirements for the overall design of the dam. Provide downstream erosion protection to prevent wall undermining and embankment erosion. Design the parapet wall to remain watertight for the life of the structure. Parapet wall design must control seepage by adequate connection with an impervious embankment zone, appropriate tie in with abutments and appurtenant structures, and provision of appropriate filter and drainage elements. Parapet wall design must address crest drainage and access requirements, and potential future settlement. Sheet piles will not meet overall parapet wall requirements without special provisions. Designs must not allow driving sheet pile into the impervious embankment zone. Consider the impacts to maintenance requirements when using parapet walls.

PART 6 – PRINCIPAL SPILLWAYS

The structural design and detailing of principal spillways must conform to the recommendations of 210-NEM-636, “Structural Design,” and NRCS standard drawings. All components of principal spillways except replaceable parts, such as gates and trash racks, must be equally durable. NRCS does not allow the use of a vegetated or earth open channel as a principal spillway.

Capacity of Principal Spillways

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.
- 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 empty at least 85 percent of the principal spillway hydrograph (PSH) routed through the retarding pool in 10 days or less. The 10-day drawdown begins when attaining the maximum water surface elevation during the passage of the PSH. The drawdown routing must include storm runoff inflow, quick return flow, upstream releases, and outflow. If more than 15 percent of the retarding storage volume remains after 10 days, raise the elevation of the crest of the auxiliary spillway by adding the volume remaining after 10 days to the initial retarding storage volume to determine the raised auxiliary spillway crest elevation.

Elevation of Principal Spillways

Single-purpose floodwater-retarding dams

Set the crest of the principal spillway or the low-stage inlet of a two-stage principal spillway at the submerged sediment pool elevation or higher. For dry dams, establish the elevation of the principal spillway inlet as described above and make provisions to drain the reservoir in a reasonable time and, thus, satisfy the functional or legal requirements of the dam.

Other dams

When providing conservation storage, determine the elevation of the crest of the lowest ungated inlet of the principal spillway by the volume, area, or depth of water required for the planned purpose or purposes and the required sediment storage. The lowest crest of the low-stage inlet, single-stage inlet, or an open spillway may serve as the crest.

Routing of Principal Spillway Hydrographs

The reservoir stage-storage curve used for routing should reflect the anticipated accumulation of sediment expected in the reservoir during its design life. The initial reservoir stage for principal spillway hydrograph routing must correspond with 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—

- For dams with significant base flow, start the principal spillway hydrograph routings at an elevation no lower than the water surface elevation associated with the base flow. Significant base flow is an average annual or seasonal flow that produces at least 0.5 feet of head over the lowest principal spillway inlet immediately prior to a flood or occupies more than 10 percent of the floodwater storage capacity.

- For dams with joint-use storage capacity, when one of the uses is floodwater detention, start routing of the principal spillway hydrograph at the elevation where the flood detention volume is empty, but the joint-use volume is full.

- For single-purpose low hazard class irrigation dams with gated outlets and vegetated or earth auxiliary spillways located on ephemeral streams in areas where the average annual precipitation is less than 25 inches, begin routing at the elevation where the reservoir contains both 100 percent of the anticipated sediment storage volume and 30 percent of the available nonsediment storage volume.

Design of Principal Spillways

Hydraulics

Design the principal spillway to carry the planned flow for expected headwater and tail water conditions. Use TR 210-29, “Hydraulics of Two-Way Covered Risers”; Technical Note (TN) 210, Design Note (DN) 8, “Drop Inlet Entrance Losses”; 210-NEH-5, “Hydraulics”; 210-NEH-650, “Engineering Field Handbook”; and other appropriate references for hydraulic design.

Risers

Drop inlet principal spillways require risers to maintain the reservoir pool level at or near the inlet crest elevation during low-flow periods; establish full pipe flow using as low a head over the crest as practical; and 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. NRCS standard covered risers have an inside width equal to the width diameter (D) of the conduit and an inside length equal to three times the width D of the conduit (D × 3D cross section).

Design risers 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 × 3D covered risers tend to line up longer pieces of trash and facilitate their passage into and through the conduit. In most cases, use covered risers with standard skirted or baffle inlets because they effectively exclude trash without clogging. Skirted inlets, having a cover with skirts extending below the weir crest elevation, are applicable where backfill or settlement levels must be at least two times the conduit width (diameter) below the crest. Use baffle inlets for a riser backfilled to the crest elevation or with expected sediment build up to the crest elevation.

Structurally design risers to withstand all anticipated water, earth, ice, and earthquake loads. Provide articulation to allow movement of the riser with respect to the conduit.

For structures that retain a normal pool, provide low-stage outlet capacity to evacuate the normal pool. Provide risers with low-stage inlets at or near the bottom with concrete aprons to prevent erosion of soil and undermining of the riser footing by high-velocity flow approaching the inlet.

NRCS encourages the use of standard covered risers for drop inlet principal spillways on low hazard potential dams with an effective height of more than 35 feet and for all significant and high hazard potential dams. New non-standard risers or non-standard risers in existing dams require documentation of structural adequacy, adequate hydraulic performance and reliable debris protection. For low hazard potential dams not more than 35 feet in effective height, NRCS permits prefabricated pipe risers where hydraulically and structurally adequate. The riser pipe must be of the same material as the conduit and at least one standard pipe size larger than the conduit pipe.

Use special riser designs for spillways having maximum conduit velocities more than 30 feet per second and for spillways having conduits larger than 48 inches in width or diameter. Generally, while like standard risers, these designs require a special elbow and transition at the junction of the riser and conduit and may require special design of the inlet. Consider hydraulic model testing if the maximum total head on the spillway is more than 75 feet or the conduit velocity exceeds 50 feet per second.

Conduits

Where possible, design the conduit straight in alignment when viewed in plan. If required, modify alignment using watertight angle changes at joints or by special elbows having a radius equal to or greater than the conduit diameter or width. Provide thrust blocks of adequate strength when using special pipe elbows. Design thrust blocks to distribute the thrust due to change in direction for the maximum possible discharge. Install drop inlet conduits 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. 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.

Design principal spillway conduits under earth dams to support fill heights greater than the original constructed height if there is a reasonable possibility of raising the embankment height later due to a change in dam classification or to incorporate additional storage for approved beneficial use.

Design rigid principal spillway conduits as positive projecting conduits in accordance with the principles and procedures given in TR 210-5, “Structural Design of Underground Conduits.”

Principal spillway conduits must consist of the types shown below and must meet the requirements and conditions listed for the specific type used.

Cast-in-place reinforced concrete conduits

Design cast-in-place rectangular reinforced concrete conduits in accordance with principles and procedures in TR 210-42, “Single Cell Rectangular Conduits Criteria and Procedures for Structural Design”; TR 210-45, “Twin Cell Rectangular Conduits-Criteria and Procedures for Structural Design”; or other appropriate design aids.

Reinforced concrete pressure pipe conduits

For pipe meeting American Water Works Association (AWWA) C301, “Standard for Prestressed Concrete Pressure Pipe, Steel-Cylinder Type,” use the 3-edge bearing strength at the first 0.001-inch crack with a safety factor of at least 1.0.

For pipe meeting AWWA-C300, “Standard for Reinforced Concrete Pressure Pipe, Steel-Cylinder Type,” AWWA-C302, “Standard for Reinforced Concrete Pressure Pipe, Noncylinder Type,” and for other types of reinforced concrete pipe, use the 3-edge bearing strength at the first 0.01-inch crack with a safety factor of at least 1.33.

Do not use elliptical or other systems of reinforcement requiring special orientation of pipe sections in spillway conduits. Design reinforced concrete pipe to support at least 12 feet of earthfill above the pipe at all points along the conduit.

Conduit diameter

Provide adequate diameter to meet hydraulic requirements, facilitate inspection, cleaning, repair, and to reduce plugging potential. Use a 36-inch minimum inside diameter for designs accommodating personnel entry.

Minimum conduit diameters on yielding foundations are as follows:

- Low Hazard Potential Dams.—The minimum inside diameter of the principal spillway conduit is 30 inches, unless a joint extension safety margin of at least 1.5 inches is used, in which case the minimum diameter is 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 Potential Dams.—The minimum inside diameter of the principal spillway conduit is 30 inches, unless a joint extension safety margin of 1.5 inches is used, in which case the minimum diameter is 24 inches.
- High Hazard Potential Dams.—The minimum inside diameter of the principal spillway conduit is 30 inches.

Minimum conduit diameters on nonyielding foundations are as follows:

The minimum inside diameter of the principal spillway conduit for low hazard potential dams is 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 potential 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 does not require articulation.

Consider the potential for hazard potential classification upgrade during the life of the structure when selecting the conduit diameter.

Previously constructed dams may retain conduits with diameter less than the minimum described above for the following:

- Conduits with satisfactory service performance, projected performance, and service life
- Conduits receiving repairs that reduce the diameter of an existing conduit below the specified minimum that will meet performance and service life requirements

Designs using conduits with diameters less than the minimums described above must—

- Provide trash racks designed to assure no debris will accumulate in the conduit.
- Use the actual inside pipe dimensions to develop the hydraulic performance characteristics of the principal spillway when establishing the auxiliary spillway crest elevation described in part 7 of this TR.

Corrugated steel pipe or welded steel pipe conduits

Designers may use corrugated steel or welded steel pipe principal spillway conduits for single-purpose low hazard potential dams with the product of storage times effective height of dam less than 10,000 acre-ft^2.

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
- Pipe watertight

Corrugated steel pipe must be polymer-coated with watertight connecting bands. Design the minimum gage based on 35 feet of fill over the pipe.

Design welded steel pipe conduits as rigid pipe. When placing joints between lengths of welded steel pipe, provide a joint extension safety margin of 1.5 inches for conduits on a yielding foundation. Protect welded steel pipe by an exterior coating as defined in National Conservation Practice Standard (CPS) Irrigation Pipeline (Code 430), or by an exterior coating of coal tar-epoxy paint conforming to Paint System F in 210-NEH-642-2, Construction Specification 82, “Painting Metalwork.”

Joints between lengths of corrugated steel or welded steel pipe, other than welded joints, must be electrically bridged on the outside of the pipe with insulated copper wire, number 6 American wire gage or larger, securely attached to the uncoated pipe metal at both sides of the joint. The wire should have a tough, waterproof insulation designed for direct burial, with a rating of at least 600 volts. Thoroughly coat bare wire and exposed pipe metal at the points of connection with a coating equivalent to the original pipe coating to prevent the entry of moisture.

Designs for the use of steel pipe require soil investigations for resistivity and pH of the subgrade and backfill materials adjacent to the conduit. Measure the resistivity using saturated samples of soil and backfill materials.

Provide cathodic protection for welded steel pipe conduits according to the criteria in NRCS TN 210-DN-12, "Control of Underground Corrosion." Provide cathodic protection meeting the above requirements for corrugated steel pipe in soil with resistivity in a saturated condition less than 4,000 ohms centimeter (ohms-cm) or with pH lower than 5.0.

For pipes not requiring cathodic protection according to the above criteria and installed without anodes, measure pipe-to-soil potentials within the first 2 years after construction. Preferably, measure the potentials after stabilization of the water level above the dam and following the development of normal postconstruction moisture content in the soil around the conduit. Immediately install cathodic protection if such measurements indicate a need.

NRCS allows the use of welded steel pipe for other low, significant, and high hazard dams if fully encased in concrete. Design the concrete encasement to withstand all external and internal loads and prevent leakage independent of the steel pipe. Design the steel pipe to withstand all internal and external hydrostatic loads without benefit of the concrete encasement. Corrosion protection for the steel pipe must meet the requirements listed above. If joints are included, provide compatible joint extension and rotation capability between the steel and concrete and meet the requirements below.

Joints

Conduit joints must 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 210-18, "Computation of Joint Extensibility Requirements," 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.

Use a margin of safety of not less than 0.5 inch. 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 principal spillway conduit, and the potential for embankment or foundation deformation under seismic loading. For significant and high hazard potential dams, estimate consolidation from adequate foundation borings, foundation samples, soil mechanics laboratory tests and engineering analysis. For low hazard potential dams that lack undisturbed foundation samples, designs may incorporate approximate procedures based on soil classification and experience estimating foundation consolidation.

For precast concrete pipe conduits, use joints incorporating a round rubber gasket set in a positive groove that will prevent its displacement from either internal or external pressure under the required joint extensibility. Precast concrete pipe conduits must have steel joint rings providing rubber-to-steel contact in the joint.

Articulate the conduit (freedom for required rotation) at each joint in the conduit, at the junction of the conduit with the riser and at the junction of any outlet structure.

Place conduits for significant and high hazard dams on concrete cradles with tapered sides that extend, as a minimum, to the spring line of the pipe to facilitate compaction around the conduit. Articulate cradles placed on yielding foundations. Concrete bedding for pipe conduits does not require articulation.

Replacement or installation of conduits in existing embankments

Installing penetrations through existing dams disrupts the integrity of the embankment. Designs should avoid penetrations unless other economically or technically feasible alternatives are unavailable.

If necessary, install or replace conduits in existing embankments using cut and cover or boring and jacking techniques.

The cut and cover excavation for conduit represents a closure section in the embankment. Excavation side slopes in the embankment must be 3:1 or flatter. Establish rationale for steepening excavation slopes up to 2:1 for sites with favorable conditions and materials. Do not steepen side slopes at sites where the compacted soils in the excavation may have considerably different stress and strain properties than the existing embankment or the excavated foundation horizons. Similarly, do not steepen side slopes of foundations or embankments consisting of dispersive clays or materials susceptible to internal erosion. Design the bottom width of the excavation wide enough to accommodate conduit and backfill construction. Specify required treatment for the excavation invert and slopes prior to backfill placement, and any material or placement requirements specific to the backfill. Extend filter diaphragms beyond the excavated surface in accordance with 210-NEH-628-45, “Filter Diaphragms.”

Do not install conduits by boring and jacking in embankments with materials that will not retain the as-excavated annular space between the embankment and casing pipe, prior to grouting, because of deformation, fallout, or raveling. Do not install conduits by boring and jacking at sites where the conduit would intersect existing filters and drains. Installations of conduits in embankments must include casing and carrier pipes. The casing pipe must be new material heavy enough to resist deformation and deflection during advancement. Do not use fluids in conjunction with installation of the casing pipe. The carrier pipe must meet requirements listed under part 6, section “Conduit” of this TR. Specify alignment and grade tolerances as required to meet carrier pipe installation, hydraulic and maintenance requirements.

Perform boring and jacking installation in a manner that will not crack or damage the embankment. Perform boring and jacking installations in a manner that will minimize the annular space between the embankment and casing pipe. Grout the annular spaces in a manner that will completely fill the space with minimal shrinkage to maintain volume stability. Annular spaces requiring grout include the space between the embankment and casing pipe, as well as the space between the carrier pipe and casing pipe. Place grout in a manner that will not damage the embankment or deform the pipe.

Rehabilitate existing conduits by lining, if the lined conduit meets hydraulic performance requirements, if the condition of the existing pipe is suitable and access is available. Conduct inspections to determine the structural adequacy of the pipe for the intended design life of the rehabilitation. When designing a lining for an existing conduit, consider—

- Designing the lining and connections to maintain as much flow capacity as possible.
- Designing the lining to be watertight.
- Designing the lining to carry external soil loads when the existing pipe lacks structural adequacy for the intended design life of the rehabilitation.
- Designing the lining to withstand internal and external hydrostatic loads.
- Selecting a liner capable of resisting all applicable environmental exposure.
- Installing the lining by clearing, cleaning, and filling of any annular space to provide complete contact with the existing conduit.

Conduit abandonment

Conduit abandonment in existing embankments requires backfilling and filtering the conduit. Clean the remnant conduit and completely backfill with cellular concrete or other nonpermeable material known to have volume stability so that the space remains completely filled and watertight. Remove upstream and downstream portions of the conduit as required to provide a uniform upstream slope and adequate distance from the downstream face to construct a filter with cover at the downstream end.

Conduit filters

Provide filters for all conduit installations as required in part 5, section “Seepage” of this TR.

Outlets

Base the choice of outlet on a careful consideration of all site and flow conditions that may affect operation and energy dissipation.

Design may include cantilever outlet and plunge pools where their use—

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

Design plunge pools to dissipate the energy and for stability. Unless constructing the pool in bedrock or highly erosion-resistant materials, the design can include riprap to ensure stability. Use NRCS TN 210-DN-6, “Riprap Lined Plunge Pool for Cantilever Outlet” (Second Edition), to design plunge pools.

Support cantilever outlets on bents or piers. Extend outlets a minimum of 8 feet beyond the bents or piers. Locate the bents 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 of excavation or potential erosion expected in the plunge pool. Set the invert of the cantilever outlet at least 1 foot above the tail water elevation at maximum discharge.

Use St. Anthony Falls (SAF) basins when adequate control of tail water exists. Use TR 210-54, “Structural Design of SAF Stilling Basins,” for structural design and 210-NEH-14, “Chute Spillways,” for hydraulic design.

Use impact basins when taking positive measures to prevent large debris from entering the conduit. Use TR 210-49, “Impact Basins Associated with Full Flow in Pipe Conduits,” for hydraulic design.

Trash Racks

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

If using a low-level reservoir outlet with a trash rack or a ported concrete riser 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.

Provide trash racks on drain outlets to protect against trash that may reduce capacity or impede gate operation.

Antivortex Devices

All closed-conduit spillways designed for pressure flow must have adequate antivortex devices.

PART 7 – AUXILIARY SPILLWAYS

Auxiliary spillways convey excess water past a dam. NRCS typically constructs auxiliary spillways by excavating open channels in natural earth, earthfill, or rock. NRCS also constructs roller-compacted or reinforced concrete structural spillways. NRCS does not design for flow passing over a dam without a designed structural auxiliary spillway.

Closed-Conduit Auxiliary Spillways

Where a design does not include an open channel auxiliary spillway, provide a closed-conduit auxiliary spillway. Closed spillways must pass the freeboard hydrograph without overtopping the dam, clogging the riser inlet, or restricting the passage of trash through the conduit elbow. Closed-conduit spillways must also meet the following requirements:

- For low hazard potential dams with a product of storage times the effective height of the dam of less than 10,000 acre-ft^2, the closed-conduit cross-sectional area must be 12 ft^2 or more in cross sectional area.
- For dams with drainage areas of 10 square miles or less (except those covered above), the closed-conduit cross-sectional area must be 20 square feet or more.
- For dams with drainage areas greater than 10 square miles (except those covered above), drop inlet spillways with an NRCS standard two-way-covered top inlet must have a minimum unobstructed cross-sectional area of each opening of the conduit of 40 square feet. Any 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 must be between 0.75 and 1.33.

A closed-conduit auxiliary spillway may also serve a dual purpose as a principal spillway.

Spillway Requirements

<u>Capacity of auxiliary spillways</u>

Auxiliary spillways must pass the auxiliary spillway hydrograph at a safe velocity determined for the site. They must have enough capacity to pass the freeboard hydrograph with the water surface in the reservoir at or below the elevation of the designed top of the dam as described in part 5 of this TR. The minimum required auxiliary spillway capacity is the greater of 200 ft^3/s or 237 DA$^{0.493}$, where DA is the drainage area in square miles. Provide a minimum of 3 feet difference in elevation between the crest of the auxiliary spillway and the settled top of the dam. State law may establish a greater minimum capacity or elevation difference between the auxiliary spillway crest and top of dam.

For vegetated and earth auxiliary spillways on high and significant hazard potential dams, NRCS will allow up to a 10-percent reduction in the precipitation values in figure 2-2 used for generating the auxiliary spillway hydrograph if—

- The designer provides computations documenting the recurrence interval associated with the reduced precipitation values and the recurrence interval associated with the precipitation values in figure 2-2.
- Computations demonstrate that the recurrence interval associated with the reduced precipitation values is not less than 1,000 years for significant hazard potential dams and 2,000 years for high

hazard potential dams.

- The design includes documentation that the dam owner understands that the auxiliary spillway may require additional maintenance due to the potential for more frequent activation of the auxiliary spillway.

Elevation of the crest of the auxiliary spillway

Figure 2–1 provides the minimum principal spillway hydrologic criteria for use when designing vegetated and earth auxiliary spillways. When routing the principal spillway hydrograph, provide adequate retarding storage and the associated principal spillway discharge to meet the 10-day drawdown requirement described in part 6 of this TR and allow no discharge through the auxiliary spillway. Raise the crest elevation of the auxiliary spillway as needed to meet both conditions. An alternative strategy may include adjustment of the principal spillway as described in part 6 of this TR.

For sites with earth spillways, figure 2-1 criteria apply where peak flows of short duration may be expected and where erosion-resistant soils and moderate slopes exist. Sites with vegetated spillways must have these same characteristics and in addition, be capable of maintaining vigorous vegetation without irrigation. For sites with erodible soils or sites unfavorable to maintaining vigorous vegetation, design the crest of the auxiliary spillway using precipitation values greater than indicated by figure 2-1.

For vegetated and armored earth auxiliary spillways, use the same precipitation data for maximum frequency of use of auxiliary used for vegetated auxiliary spillways in figure 2-1.

Auxiliary Spillway Routings

Route the stability design and the freeboard hydrographs through the reservoir starting with the water surface at the highest of the—

- Elevation of the lowest ungated principal spillway inlet.
- Anticipated elevation of the sediment storage.
- Elevation of the water surface associated with significant base flow.
- Pool elevation after 10 days of drawdown from the maximum stage attained when routing the principal spillway hydrograph.

Exceptions to these elevations include—

- Dams with gated spillways and joint use storage capacity. Start stability design and freeboard hydrograph routings 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 potential irrigation dams. Start stability design and freeboard hydrograph routings at or above the water surface elevation of the irrigation storage.

Hydraulic Design

For open channel auxiliary spillways with an inlet channel, a level crest, and an exit channel, subcritical flow exists in the inlet channel. When flow at the entrance of the exit channel is supercritical, the subcritical flow in the level crest passes through critical depth providing hydraulic control. When flow in the exit channel is subcritical, the level crest does not function as a control section.

Evaluate the relationship between the water surface elevation in the reservoir and the discharge through the auxiliary spillway to establish a stage discharge relationship for the auxiliary spillway. Compute the water surface profile from the reservoir pool through the full length of the spillway, including the inlet channel, a level crest, and exit channel. Use flow resistance values appropriate for the type of auxiliary spillway.

When the level crest functions as a hydraulic control causing the exit channel maximum discharge flow to become supercritical, the exit channel grade must 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.

When flow in the exit channel is subcritical for the maximum discharge during passage of the freeboard hydrograph, compute the water surface profile through the exit channel, level crest, and into the reservoir for an accurate stage-discharge rating of the auxiliary spillway.

Structural Stability

Adequately investigate, analyze, design, and construct the spillway 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 that are stable against sliding. Evaluate the stability of the cut slope 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 auxiliary spillways typically have trapezoidal cross-sections and a grass cover to protect them from damaging erosion. They adapt well to sites capable of sustaining vigorous grass growth by normal maintenance without irrigation.

Designs may use earth auxiliary spillways at sites incapable of maintaining vegetative growth. While like vegetated spillways, NRCS typically designs earth spillways for lower velocities, lower stresses, and less frequent use. They typically require more maintenance than vegetated spillways after a flow occurs.

No damage should occur to vegetated or earth spillways during passage of all flows up to the auxiliary spillway hydrograph. Although vegetated and earth auxiliary spillways may experience minor surface erosion during passage of larger infrequent storms, the spillway must not breach during passage of the freeboard storm.

Designers can use hydraulic data in TR 210-39, “Hydraulics of Broad Crested Spillways,” in the design of vegetated and earth auxiliary spillways. Use a minimum vegetal retardance curve index of 5.6, as defined in Agricultural Research Service (ARS) Agriculture Handbook 667 (AH–667), “Stability Design of Grass-Lined Open Channels,” to determine hydraulic capacity and vegetal stress in vegetated spillways. Use a minimum Manning’s roughness coefficient, *n*, of 0.02 for earth spillway design. Base the actual hydraulic capacity of the spillway on an appraisal of the expected roughness condition at the site.

Maintenance for auxiliary spillways increases as the frequency and duration of flow increase. Good designs balance maintenance cost against the cost of modifying the other elements of the dam to reduce the flow frequency. Strategies to economize the design of auxiliary spillways include raising the crest elevation, increasing the capacity of the principal spillway, adding a structural primary auxiliary spillway, providing erosion protection, or any combination thereof.

Maintaining the effectiveness of vegetated and earth spillways requires a systematic and regular maintenance program. If damage to a spillway occurs during a flow event, complete a repair of the spillway as soon as possible following the flow event.

The O&M plan for the dam should describe the expected maintenance and repair for conditions such as vehicle ruts created during wet conditions, degradation from burning surface vegetation, and impact of mower blades on components above the soil surface.

Layout

210-NEH-628-50, "Earth Spillway Design," provides guidance on the layout of auxiliary spillways. Locate spillways away from the dam whenever possible. The layout and profile of vegetated and earth spillways should provide safety against breaching of the spillway during the passage of the freeboard hydrograph. The designer should consider extending the length and flattening the grade of the exit channel to delay or prevent headcut formation, as well as maximizing material bulk to contain any headcut.

The exit channel, including cross section, slope, alignment, and enclosing cut slopes or dikes must contain the discharge for the freeboard hydrograph and discharge flow without impinging on the toe of the embankment or groins. Evaluation of this aspect of the design must consider the expected erosion within the spillway channel.

Designers may curve the inlet channel centerline, but it must be tangent to the centerline of the level crest. The level crest must be at least 30 feet long, the same width as the exit channel, with its centerline straight and coincident with the centerline of the exit channel.

In plan view, the upstream portion of the exit channel must align parallel to the direction of flow through the level crest. This straight section of the exit channel must extend beyond the farthest downstream toe of the embankment. The channel may curve beyond the toe of the dam if flowing water will not impinge on the toe should the exit channel fail at or near the curve. Designs should include support computations verifying that flows will not impinge on the dam toe.

Designers may terminate the exit channel at some point above the maximum tail water elevation or may extend the exit channel 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 flows produce stresses exceeding allowable stresses. Consider land rights in making the decision on how to handle the return flow to the natural or constructed stream channel downstream from the dam, and where flows will deposit eroded materials.

NRCS does not allow the use of ramp spillways for significant or high hazard potential dams. For low hazard potential dams, provide justification for placing the spillway on the dam, and for using a ramp spillway in lieu of erosion protection or a structural spillway. Design ramp spillways no steeper than 10 percent. Construct ramp spillways with the same compaction procedures and quality control as the earth embankment. Provide 1 foot of topsoil on the surface of a vegetated ramp spillway.

Stability design of vegetated and earth spillways

Vegetated and earth spillways must be stable and not erode when routing the auxiliary spillway hydrograph through the dam. The design stress must not exceed the maximum stress limitations for vegetated and earth spillways in any reach of the constructed exit channel. Base the stress limitations on uniform flow conditions in the exit channel during the maximum discharge when routing the auxiliary spillway hydrograph.

For spillways with anticipated use more frequent than once in 50 years, determine the allowable stress for a vegetated or earth spillway in accordance with AH–667. The allowable values may be increased 20 percent when the anticipated average use is once in 50 years or increased 50 percent when the anticipated average use is once in 100 years. Determine the allowable stress for the actual vegetal cover conditions reasonably expected to exist at the time of the flow. For grasses or mixtures not included in AH–667, determine values by comparing site characteristics with those described in AH-667. In lieu of values shown in AH-667, designers may use values determined from special studies or investigations for a species, soil, and condition.

Integrity design of vegetated and earth spillways

Evaluate the spillway for headcut development and advancement during passage of the freeboard storm using the procedures in 210-NEH-628-51, "Earth Spillway Erosion Model," and 628-52, "Field Procedures Guide for the Headcut Erodibility Index." The auxiliary spillway must not breach (i.e., headcut will not advance beyond the upstream edge of the level crest) during passage of the freeboard storm.

Evaluate headcut development for the center and both sides of the spillway channel.

Special precautions for high hazard potential dams

Give special consideration to the layout of spillways on high hazard potential dams to ensure the spillway will not breach under the most extreme flow conditions. Increase the length of the exit channel to the maximum extent possible to keep the area most susceptible to erosion a considerable distance from the dam. Within the limitations of the site, provide maximum material bulk in the profile of the spillway.

Use exit channel levees when necessary. When used, design exit channel levees high enough to contain the peak flow of the routed freeboard hydrograph. Construct levees of erosion-resistant materials compacted to develop maximum resistance. They must have a top width not less than 12 feet, and, if not protected with riprap, have side slopes 3 horizontal to 1 vertical, or flatter, on the side where water flows. When constructing levees on a foundation subject to piping or undermining, key the levees into the foundation with a compacted core having a bottom width not less than the top width of the levee and of enough depth to reach sound material, or to a depth equal to the overall height of the levee, whichever is less. Exit channel levees and cuts must be stable with the depth of headcut erosion estimated at the inside (valley side) of the spillway channel.

Where soils are highly erodible, provide crest control structures to maintain a uniform surface during onsite runoff and very low flows through the spillway. Increase the effective bulk length by installing barriers that will effectively stop a gully advancing through the spillway. Consider 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. As an alternate or complementary procedure, increase the capacity of the principal spillway by means of a two-stage inlet of enough size to carry an appreciable amount of the outflow hydrograph.

Barriers to stop headcut progression in earth spillways

Designs can use barriers, such as cutoff walls, to stop headcut progression. Barriers used to stop headcut progression include measures such as reinforced concrete walls, sheet pile walls, and secant walls.

Design barriers to function with the maximum projected headcut. Barriers must retain structural stability following the passing of all design flow events and resulting soil and rock erosion.

Armored earth auxiliary spillways

Designs can incorporate hard armor erosion control products such as articulated concrete blocks (ACBs) or riprap to protect earth auxiliary spillways. Design articulated concrete block protection for auxiliary spillways using guidance contained in 210-NEH-628-54, “Articulated Concrete Block Armored Spillways.”

Do not use erosion control products, such as turf reinforcement mats, to increase design values for auxiliary spillway erosion resistance due to the limited durability, longevity, and damage susceptibility of such erosion control products.

The design should clearly state the expected functions of the erosion control components regarding spillway performance. The design of surface erosion protection should reference laboratory test results or prototype performance under conditions equal to or exceeding the design conditions for the following hydraulic design parameters:

- Shear stress
- Depth of flow
- Surface slope
- Flow duration
- Debris in flow

Address the permanence of erosion protection in the design, including expected design life of the armor materials. For protection from routed freeboard hydrograph flows, the design must include terminal features required in conjunction with the erosion protection. For protection from routed auxiliary hydrograph flows, the design must include terminal features unless the design clearly documents that the design will not fail without terminal features, and that the installation would not compromise integrity protection. Base the design of terminal features on applicable laboratory tests or prototype performance.

The designer should understand the capabilities and limits of erosion estimation software when analyzing the performance of erosion protection and erosion cutoff features. For example, while SITES and WinDAM software have the capability to model the integrity of erosion-resistant headcut barriers in the auxiliary spillway, they do not have the capability to model the performance of surface erosion protection measures such as ACBs.

Design erosion protection components to meet stability requirements must include evaluation of effects on project repair costs from loading exceeding the stability design loading. The evaluation must include estimating the extent and cost of repair of the erosion protection from floods intermediate to the stability and integrity hydrographs. The design must identify the required O&M and potential repair requirements for the spillway.

The plans and specifications should provide enough detail for installing erosion protection features, including the foundation and drainage requirements, area of coverage, proper overlap, and anchoring the edges. The construction inspection plan should provide for inspectors with appropriate expertise to monitor installation of these structural features.

In-Situ Rock Auxiliary Spillways

Some of the principles used for the layout of earth auxiliary spillways apply to auxiliary spillways excavated into rock. Determine allowable average frequency of use and permissible velocities 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 Manning's roughness coefficient, *n*. Rock auxiliary spillway designs must not breach during passage of the freeboard storm.

Structural Auxiliary Spillways

NRCS typically uses reinforced concrete or roller compacted concrete to construct structural auxiliary spillways. Some applications may require the use of other materials, such as Portland cement grouts, nonreinforced concrete, or mass concrete. A structural auxiliary spillway may also serve a dual purpose as a principal spillway.

Structural auxiliary spillway designs must pass the freeboard hydrograph without causing damage to the dam or the structures themselves. The structural spillway configuration must be compatible with foundation conditions at the site, channel stability downstream from the spillway, possible range of tailwater depth, and proximity of the spillway to the embankment. The inlet portion of a chute spillway must consist of a straight inlet, a box drop inlet, an ogee crest, or other appropriate hydraulic structure that will produce critical flow at the crest and result in a determinate stage-discharge relationship.

Use principles such as those contained in 210-NEH-5, "Hydraulics"; 210-NEH-11, "Drop Spillways"; 210-NEH-14, "Chute Spillways"; and U.S. Department of Interior Bureau of Reclamation (USDI-BOR) publications for the hydraulic design of structural auxiliary spillways. When considering the effects of air entrainment by water traveling at supercritical velocities, base the design on appropriate model studies.

Design structural auxiliary spillways to contain all flows up the maximum freeboard discharge. When the physical size of a structural auxiliary spillway exceeds that of structures commonly designed by NRCS, perform model studies or other special studies.

The outlet section of concrete chute spillways must consist of a hydraulic jump basin, such as an SAF, deflector bucket, roller bucket, or other appropriate hydraulic structure, that will dissipate the energy of the high velocity discharge. Design structural auxiliary spillways to withstand lateral earth pressures, uplift, seepage and other hydrostatic and hydrodynamic pressures. Use safety factors for structural design for the full maximum freeboard discharge with uplift and sliding safety factors greater than or equal to 1.1.

Structural spillways over embankments must include drainage under the full footprint of the spillway chute adequate to filter and discharge seepage through the embankment under normal and flood surcharge reservoir elevations, and leakage through the spillway during operation. Address differential settlement of the spillway due to varying foundation conditions and potential embankment or foundation deformation under seismic loading.

GLOSSARY

Active fault.—A fault on which tectonic movements have occurred within the Holocene epoch (approximately the last 10,000 years).

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 flow persisting after depleting storm runoff and associated quick return flow. Base flow usually originates from groundwater discharge or gradual snow or ice melt over extended periods, but need not be continuous flow. (Often 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.

Capable fault.—A fault with no known evidence of Holocene displacement, but which has experienced tectonic movements within the last 35,000 years or lacks adequate data to preclude tectonic movements within the last 35,000 years.

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

Consequence of seismic failure.—Damages downstream of the dam from seismic failure with the reservoir at the normal pool elevation. The consequences low, significant, and high correspond to the consequences used for determining hazard potential classification per 210-NEM-520-C, "Dams."

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 with the capacity to impound or divert water.

Design life.—A period of time during which designers expect a dam to perform its assigned functions satisfactorily.

Dry dam.—A dam that has an ungated outlet positioned that allows the drainage of essentially all water from the reservoir by gravity. The reservoir will normally remain 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 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 that conveys the flow to a point where flow release will not jeopardize 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. Designers also use the freeboard hydrograph to evaluate the integrity (breaching resistance) of a vegetated or earth auxiliary spillway.

General storm.—General storms are major synoptic events that produce precipitation over areas more than 500 square miles over durations often much longer than 6 hours.

Ground motion.—A general term including all aspects of ground motion that can result at a site from earthquake shaking. Often this term is in relation to particle acceleration, velocity, or displacement.

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

Joint extensibility.—The amount a pipe joint extension from the fully engaged position without losing strength or water tightness. In case of rubber gasket joints, measure joint extensibility 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 that serves two or more purposes; for instance, conservation storage and floodwater storage.

Local storm.—Local storms have durations up to 6 hours and cover areas up 500 square miles.

Maximum design earthquake (MDE).—The earthquake selected for design or evaluation of the dam. Designers typically select this earthquake based on a probabilistic assessment of the ground motions that may occur near the dam. The hazard potential classification and consequences of seismic failure of the dam provides the basis for the required recurrence interval of ground motions resulting from this earthquake. These ground motions do not necessarily represent the most severe earthquake ground motions considered possible at the site. In application, designers may define the maximum earthquake as several events to determine which generate the most critical ground motions for evaluation of the seismic performance of the dam. Designers must demonstrate that the dam can withstand the level of earthquake shaking resulting from the MDE without release of water from the reservoir.

Multipurpose dam.—A dam whose conservation and flood pool serve multiple purposes. Commonly, a multipurpose dam involves additional water supply outlets for local uses.

Normal pool.—The portion of the reservoir and associated reservoir elevation that exists when not retarding flood inflows. This pool would typically be at the elevation of the lowest ungated principal spillway inlet.

Operating basis earthquake (OBE).—The design earthquake generating the most critical ground motions for evaluation of a postearthquake scenario. The postearthquake evaluation must demonstrate that the dam can withstand the level of earthquake shaking resulting from the OBE and have the capability to pass the principal spillway flood without failure.

Overall height of dam or embankment.—The difference in elevation, in feet, between the top of dam and the lowest elevation at the downstream toe.

Peak ground acceleration (PGA).—The maximum ground acceleration at a site experienced during the ground motion resulting from earthquake shaking. The USGS typically expresses earthquake acceleration as a percentage of the acceleration due to gravity (g).

Postearthquake operating evaluation.—The minimum earthquake a dam must withstand, with damage limited to the extent that the dam retains partial function prior to repair of the dam. The hazard potential classification of the dam provides the basis for the required recurrence interval of ground motions resulting from this earthquake. Designers must demonstrate that the dam and appurtenances subjected to OBE ground motions can pass the principal spillway flood without release of stored water from the reservoir.

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. Designers also use the principal spillway hydrograph 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 minimum hydrograph safely passed through a vegetated or earth spillway without initiating surface erosion.

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

Valley floor.— Term used in geologic investigation guidance, often synonymous with a floodplain.

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.

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R. NRCS 210-NEH-630, “Hydrology.”

S. NRCS 210-NEH-630-9, “Hydrologic Soil-Cover Complexes.”

T. NRCS 210-NEH-630-10, “Estimation of Direct Runoff from Storm Rainfall.”

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Y. NRCS 210-NEH-633-26, “Gradation Design of Sand and Gravel Filters.”

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EE. NRCS TN-210-DN-6, “Riprap Lined Plunge Pool for Cantilever Outlet.”

FF. NRCS TN-210-DN-8, “Drop Inlet Entrance Losses.”

GG. NRCS TN-210-DN-12, “Control of Underground Corrosion.”

HH. NRCS TR-210-5, “Structural Design of Underground Conduits.”

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KK. NRCS TR-210-29, “Hydraulics of Two-Way Covered Risers.”

LL. NRCS TR-210-39, “Hydraulics of Broad Crested Spillways.”

MM. NRCS TR-210-42, “Single Cell Rectangular Conduits Criteria and Procedures for Structural Design.”

NN. NRCS TR-210-45, “Twin Cell Rectangular Conduits-Criteria and Procedures for Structural Design.”

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UU. USBR Reclamation Design Standards No.13 (2011), “Embankment Dams,” Chapter 8, “Seepage”

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