PAP-598

ALTERNATIVES FOR ENHANCING SPILLWAY CAPACITY CURRENTLY BEING PURSUED BY THE U.S. BUREAU OF RECLAMATION

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ABSTRACT

This paper will present an overview of Reclamation's experience and current practices in providing additional discharge capacity for existing dams when dam safety criteria require structural modifications. This overview includes selection criteria for labyrinth weirs, fuse plug spillways, increasing unit discharges over existing ogee crests, and overtopping protection systems for embankment and concrete dams. Emphasis will be placed on hydraulic and structural design considerations for each alternative, as well as case histories of recent dam safety modifications.

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INTRODUCTION

Reclamation (Bureau of Reclamation) a Federal agency with responsibilities to develop and manage water resources in the 17 Western United States. Consequently, Reclamation is responsible for operating and maintaining over 300 large dams. This responsibility includes enforcing the Reclamation Safety of Dams Act of 1978 (Public Law 95-578) and the amendments of 1984 (Public Law 98-404). As a result of these laws, Reclamation established the SEED (Safety Evaluation of Existing Dams)/SOD (Safety of Dams) Program. This program includes reviewing the design, construction, and performance history of all Reclamation dams, evaluating their structural and hydraulic integrity based on safety of dams criteria, and determining any need for remedial action. Periodic reviews are also conducted to ensure adherence to operation and maintenance guidelines.

When a potential hazard is identified, appropriate technologies are used to correct the deficiencies. If a structural modification is required, several options must be evaluated to determine the most cost effective engineering design.

Designing cost effective structures to pass large rare flood events up to the PMF (probable maximum flood) is being addressed by Reclamation. Designing traditional service spillways, even though the probability of their operation is low, may not be an effective option for providing additional discharge capacity. Both new and existing projects experience this problem. Dam safety of Reclamation's existing structures, some approaching 100 years of service, is evaluated based on flood magnitudes generated using modern hydrologic techniques. In many cases the PMF has been drastically increased over original estimates. For these cases and future projects, cost effective alternatives for emergency spillway structures must be found.

This paper presents an overview of Reclamation's design and construction experience and hydraulic research associated with enhancing spillway capacity using emergency or auxiliary spillways. This overview includes selection criteria for labyrinth weirs and fuse plug spillways, increased unit discharges over existing ogee crests, and overtopping protection systems for embankment and concrete dams. This paper will emphasize hydraulic and structural design considerations for each spillway alternative.

AUXILIARY AND EMERGENCY SPILLWAY SELECTION CRITERIA

In general, dams are designed with a service spillway to pass routine flows and an auxiliary or emergency spillway to pass extremely large flood events. Objectives in designing an auxiliary or emergency spillway include high discharge capacity and system reliability, combined with low maintenance and cost. Many considerations interplay in selecting an appropriate spillway for the site in question (table 1).

Spillway designs depend on several factors unique to the project location; therefore, a designer must consider the following site-specific details:

- Topographic, geologic, environmental, and aesthetic concerns
- Stability of existing structure and its foundation
- Space available for excavation

- Class and quantity of excavation material for disposal and reuse as construction material
- Downstream scour potential and tailwater influences
- Foundation permeability and potential erodibilty
- Stability of excavated slopes
- Hydraulic and structural influences on adjacent structures (intakes, stilling basins, etc.)

Table 1. - Spillway Selection Considerations

Functional Considerations	Safety Considerations
1. Adequate release capacity to accommodate the IDF (inflow design flood)	1. Hazard to downstream residents and property is adequately addressed
2. Compatible with type of dam and geologic conditions	2. Structurally adequate for full range of releases required to pass the IDF
3. Satisfies project's operational requirements	3. Releases are adequately controlled to ensure safety of dam
4. Selected spillway(s) option is economical	4. High operating reliability

This paper will examine only auxiliary and emergency spillway alternatives. The terms auxiliary spillway and emergency spillway are often used interchangeably. This paper defines these terms as follows:

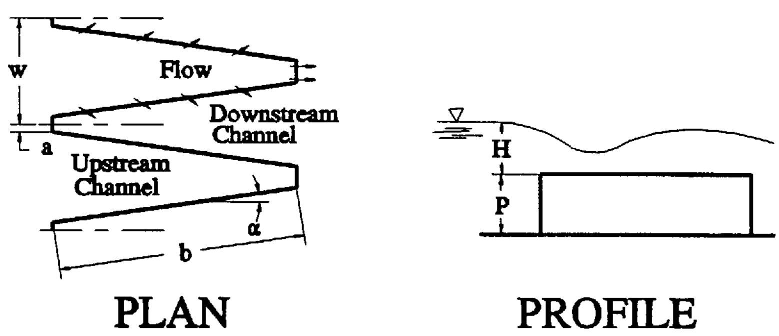
Auxiliary spillway - a backup spillway designed to pass floods (usually greater than the 100-year event) in a controlled channel past the dam. To minimize cost auxiliary spillways are generally uncontrolled (no gates) weir-type structures with the crest set above normal maximum reservoir elevation. To function as auxiliary spillways, these structures are designed to maximize discharge capacity as a function of depth above the weir.

Emergency spillway - a nonservice spillway intended to operate only when a flood endangers the structure. Emergency spillways are generally designed to pass floods with predicted recurrence intervals much greater than the design life of the dam. Typically, an emergency spillway consists of a control section with little or no formal conveyance channel downstream. Emergency spillways are least-cost alternatives and have a very low probability of operation.

LABYRINTH WEIR SPILLWAYS

The principle of a labyrinth weir spillway is to modify the plan shape of a linear, sharp-crested weir to increase the effective crest length. This modification provides an increased discharge capacity for the same channel width and operating head. Labyrinth weirs are ideally suited to meet the objectives of an auxiliary spillway. They are of particular value when site topography limits spillway width. Likewise, labyrinth spillways provide an efficient means of increasing spillway capacity without raising the dam's crest.

Although nearly any geometric pattern could be used for a labyrinth weir, triangular and trapezoidal shapes are generally used for construction simplicity. Labyrinth weirs are defined by their geometric pattern, length magnification ratio, and vertical aspect ratio (fig. 1).



l = Developed length for one cycle = 4a + 2b
l/w = Length magnification ratio
w/P = Vertical aspect ratio
α = Angle of sidewall to flow direction

Figure 1. - Plan and profile of a trapezoidal labyrinth weir.

Reclamation first conducted research on labyrinth weirs to design an auxiliary spillway for Ute Dam near Logan, New Mexico (Houston, 1982). Subsequently, studies for Hyrum Dam, Utah (Houston, 1983) and Ritschard Dam, Colorado (Vermeyen, 1991) have been completed.

Hydraulic Design Considerations

The flow pattern over a labyrinth weir is complex when compared to flow over a straight weir. The convergent geometry of each cycle forces most of the flow to pass over the weir at an angle rather than normal to the crest. This also results in a variable head along the weir crest (similar to a side channel spillway). As the flow drops over the crest into the channel between weir cycles, the flow nappes interfere starting at the upstream apexes. Nappe interference and its severity are a function of both the length magnification (l/w) ratio and vertical aspect (w/P) ratio. At low heads (H/P $\leq$ 0.20), a labyrinth weir operates similarly to a linear weir, thus taking full advantage of the additional crest length.

As head on the weir increases several factors cause a steady decrease in discharge efficiency. The most notable factors are contraction effects upstream of the crest and nappe interference downstream of the crest. These effects on weir performance are best illustrated by laboratory data. Vermeyen (1991) collected data for a trapezoidal labyrinth weir designed for the proposed Ritschard Dam (fig. 2). Ritschard's labyrinth was designed with a quarter-round crest shape because research by Houston (1983) indicated this shape had better discharge characteristics than a sharp-crested weir and is easier to construct. Discharge coefficients are plotted against H/P for an extended range of heads (fig. 3). Discharge coefficients were calculated using $C=Q/(LH^{3/2})$, where L is the cumulative developed length for all labyrinth weir cycles. Notice the high discharge coefficients at small H/P values and a general decrease in performance for larger values of H/P. For H/P values less than 0.2 the discharge coefficients were greater than 3.33 because the nappe was not fully aerated. The small discontinuity in the curve (near H/P equal to 0.2) occurred when the nappe was aerated as it separated from the face of the downstream wall.

To achieve sound hydraulic performance at high heads, it is recommended that w/P should be 2.5 or greater, and l/w should be in the range of 3 to 5. These criteria facilitates a reasonable

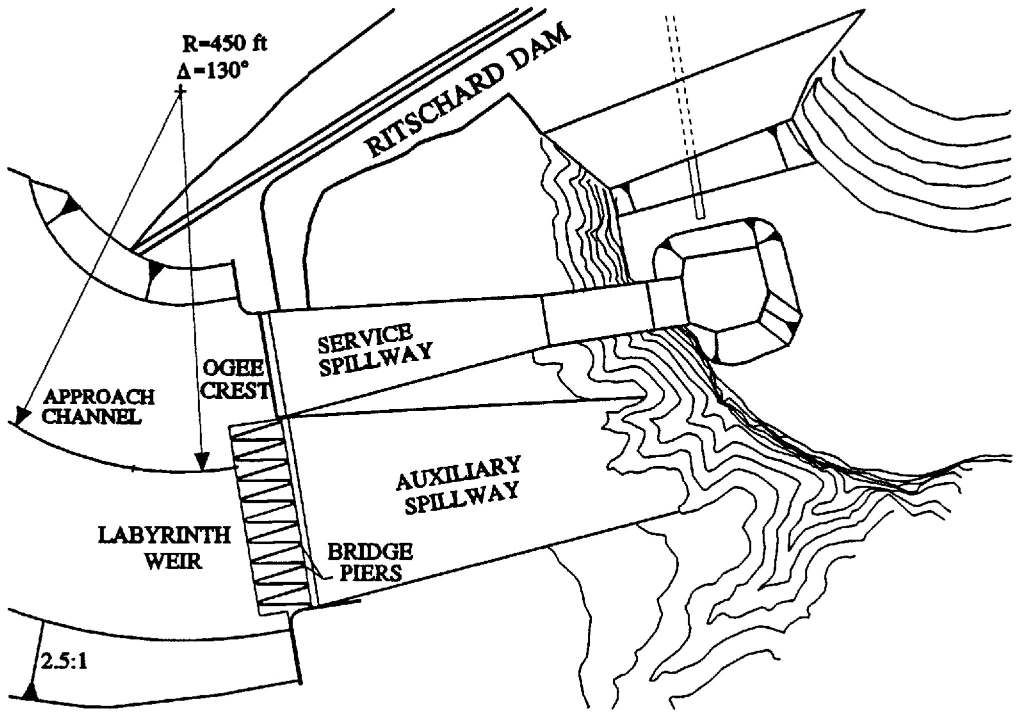


Figure 2. - Plan view of proposed Ritschard Dam spillways.

approach velocity and minimize nappe interference.

Spillway approach conditions affect overall hydraulic performance and are therefore important when designing a labyrinth weir. The approaching flow direction should be perpendicular to the spillway axis to ensure a uniform flow distribution. Likewise, the inlet structure and piers should be designed to minimize flow disturbances and head loss. If flow contractions cannot be avoided a significant portion of the labyrinth weir will have a reduced efficiency.

Reclamation research (Hinchliff, 1984) determined that labyrinth weirs perform better when the cycles are projected into the reservoir, rather than being contained within a spillway chute. Reduced approach velocities minimize flow contraction at the spillway entrance, which increases the efficiency of the cycles adjacent to the entrance.

This concept was used on the Ritschard Dam project (fig. 2). The labyrinth crest was set at an elevation 5 ft above the ogee-crested service spillway. The 5-ft elevation differential was selected to prevent the auxiliary spillway from operating for flows less than the 100-year recurrence interval flood. Ritschard's labyrinth has a 485-ft-wide curved approach channel (R = 450 ft, Δ = 130°) and a 50-ft-long, straight reach just upstream of the weir. However, the approach velocity was minimized by excavating a deep approach channel and extending the labyrinth into the reservoir to minimize flow acceleration associated with channelizing the flow. Consequently, no measurable difference existed between the discharge coefficients measured using a 1:20 scale sectional model and using the 1:45 scale three-dimensional model. This labyrinth design included bridge piers on the downstream apexes, so developed length calculations did not include downstream apex lengths.

Labyrinth weir spillways are ideally suited to pass large flows with relatively small head rise within a reservoir. Due to flow complexity and design variations, limited design data are available. Drawing on the work of Hay and Taylor (1970), Houston (1982, 1983), and others, Lux (1989) used statistical correlations to generalize available design data for triangular and trapezoidal weir forms. Designer's tools have improved, but a need to investigate unique designs using hydraulic model studies still exists.

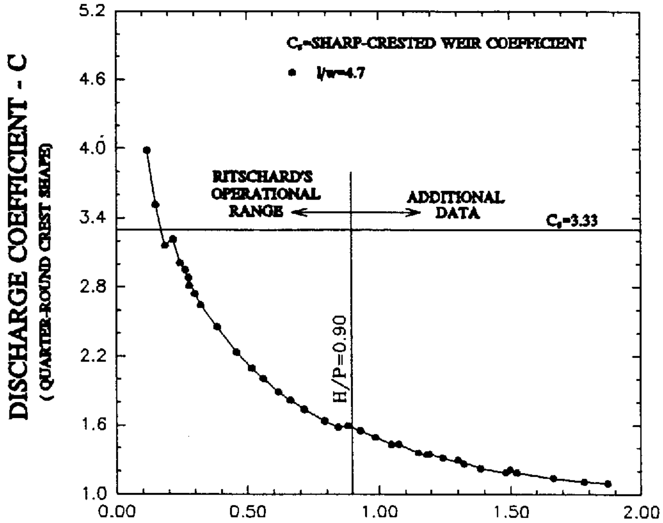


Figure 3. - Discharge characteristics, $C=Q/(LH^{3/2})$, for Ritschard Dam labyrinth weir.

In summary, labyrinth weirs have several advantages over other types of control structures when used in an auxiliary spillway, they include:

- High discharge capacity for low heads
- Can provide additional reservoir storage capacity
- Can be used to replace existing spillway structures

Structural Design Considerations

The structural analysis of a labyrinth weir begins with an examination of the foundation and overall structural stability. This examination includes the analysis of overturning, sliding, and foundation bearing capacity. The overall structural stability analysis includes an evaluation of uplift pressure and hydrostatic loads.

Generally, labyrinth spillway structures incorporates cantilever walls. Low walls with small hydraulic loadings can be designed using a simple wall analysis. High walls may require a three-dimensional analysis. Walls exceeding 30 to 40 ft in height may require gravity wall or counterforted wall designs. The structural analysis of the walls generally consists of a three-dimensional analysis evaluating 1/2 or 1 full cycle for hydrostatic loads, temperature loads, and earthquake loads when applicable.

The analysis of a labyrinth wall is different than a cantilever wall, in that the boundary conditions are different at the apex locations. Structural analysis for the Ute Dam labyrinth weir (Lux and Hinchliff, 1985) indicated that high stresses were concentrated near the apexes. High stresses result from extreme temperature loads. As a result, large quantities of reinforcement were required to resist the high bending moments, and tensile and shear stresses in the walls and base slab. The bending moment transferred in the base slab can be fairly complex in the area just upstream of the downstream apex. Resulting deflections of the base slab can produce local tension at the slab/foundation contact. These tensions could affect the uplift forces beneath the slab resulting in reduced overall stability. When three-dimensional analysis is required it should also be used to check overall structural stability.

From a structural standpoint, layout of the labyrinth spillway should consider an upstream apex at the abutments rather than a downstream apex. The upstream apex will provide a connection in compression rather than tension, as would be the case with a downstream apex.

FUSE PLUG SPILLWAYS

A fuse plug is an embankment section designed to erode in a predictable and controlled fashion when additional spillway capacity is necessary. A labyrinth or ogee crest spillway would meet the same objective, but would require a substantially wider crest length to pass an equal discharge, because a fuse plug spillway can develop a deep spillway section after the embankment section has washed downstream. Fuse plugs can also be used to block an existing auxiliary spillway, allowing additional reservoir storage while protecting the dam from overtopping during infrequent floods (similar to a gated structure without the need for mechanical or human operators).

One of the first applications of a fuse plug embankment spillway was for the Oxbow project on the Snake River between Idaho and Oregon (R. L. Albrook, 1959). A fuse plug embankment was used to replace a radial-gate-controlled spillway. Model studies were conducted to verify performance included 1:20 and 1:40 scale model tests in a laboratory and a 1:2 scale field test at the dam site, which are well documented. Fuse plug spillways were constructed by Reclamation in the early 1950s at Box Butte and Sumner Dams. The Box Butte auxiliary spillway design includes a series of nine embankment sections separated by divider or splitter walls. These sections are about 50 ft in length, and each embankment section is stepped six inches in elevation with the lowest section located in the middle.

As a result of recent developments in fuse plug design criteria (Pugh, 1984, 1985), Reclamation considers fuse plugs a cost effective alternative to providing additional spillway capacity. Fuse plug auxiliary spillways are currently proposed at both Horseshoe and Bartlett Dams on the Verde River near Phoenix, Arizona, to correct SOD deficiencies related to dam overtopping during the PMF.

Hydraulic Design Considerations

A fuse plug is designed as a dam, stable for all conditions of reservoir operation except for a flood that will cause overtopping. A breach in a fuse plug should begin at a preselected location, not at a random location dictated by construction techniques or settlement. A section of the fuse plug should be constructed at a lower elevation, commonly referred to as a pilot channel. Once the pilot channel has eroded the rest of the fuse plug is removed by erosion in a lateral direction (fig. 4).

Because fuse plugs are generally constructed on nearly flat channels, discharge coefficients for broad-crested weirs have been used in flood routing analyses. Depending on site-specific conditions, the hydraulic efficiency of the spillway can be improved by using an ogee crest structure.

One of the advantages of a fuse plug spillway is that it can regulate releases much like a gated spillway without costs associated with a gate structure. After it is completely breached, the fuse plug crest width is sized to discharge the design flood. Multiple sections with various crest elevations can be used to control the outflow as a function of reservoir rise. This minimizes the incremental discharge potential for smaller flood events.

Pilot channel elevations should be evaluated based on site-specific criteria such as:

- Rate of the reservoir rise versus erosion rate of the fuse plug section

- Approach channel characteristics (head losses) and potential localized drawdown on adjacent sections (for multiple section fuse plug embankments only)

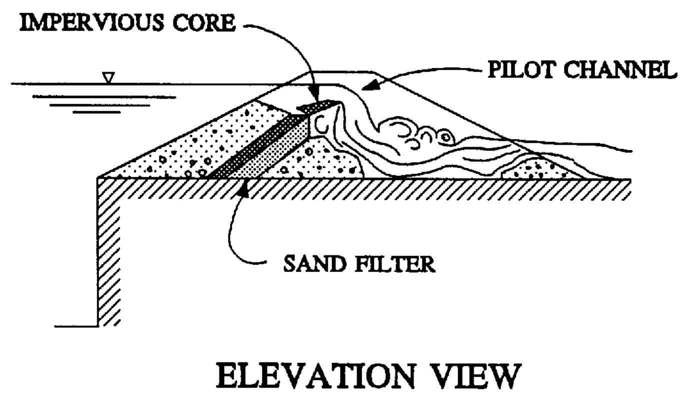


Figure 4. - Flow through a fuse plug pilot channel.

- Confidence in erosion rates and breach initiation

All of these criteria contribute to the selection of the pilot channel elevations. For multiple fuse plug embankments a sufficient elevation difference must exist between pilot channels, or the dam crest, to prevent premature overtopping of an adjacent fuse plug section or the dam itself. The rate of reservoir rise is an important factor in setting the pilot channel elevations. Using flood routing results, the rate of reservoir rise per hour can be compared with the time at which breaching of the fuse plug section is expected to occur. For example, if the average rate of reservoir rise is 3 ft during fuse plug operation, then a 3-ft or greater elevation differential should be provided between pilot channels. Because time and flow depth are not known exactly for the initial formation of fuse plug breach, sensitivity studies should be performed to determine if safety factors are needed in establishing pilot channel elevations.

If a fuse plug is divided into multiple sections, the potential for localized drawdown at an adjacent pilot channel should be analyzed. Analyses entail determining the water surface drawdown related to energy losses in the approach channel and localized increases in velocity head. Failure to account for drawdown may result in a pilot channel that may not breach as designed.

Erosion rate is another parameter in the design of a fuse plug that should be evaluated. Model studies conducted by Reclamation (Pugh 1984, 1985) have provided some guidance on the expected erosion rates as compared to height of the fuse plug embankment. These erosion rates are based on a specific fuse plug embankment design. Any deviation from this fuse plug section would generate different results.

Structural Design Guidelines

Because it is important to establish and maintain a hydraulic control section, a concrete crest or sill block with a base slab should be considered. A crest structure and base slab, with a cutoff wall, is needed to protect the structures from erosion and head cutting.

Multiple fuse plug embankments are separated by concrete splitter or divider walls. The contact between impermeable core and the splitter wall is considered a critical area because of the narrow core in a fuse plug embankment. Core sections as narrow as 5 ft are often used to maintain the predictable performance of breach initiation and erosion rate. A batter (slope) on the walls and possible flaring and thickening of the core in the wall area have been recommended, especially if the fuse plug will be designed to operate within the active conservation space of the reservoir. Training wall extensions on the splitter walls for both the upstream and downstream areas may be required to prevent erosion of an adjacent fuse plug section.

Structural design of splitter walls, which are essentially buried in the fuse plug embankment, should include loadings from both construction and operational conditions.

UNCONTROLLED OGEE SPILLWAYS

An ogee crest is a common control structure shape for service spillways, including morning glory inlets, side channel inlets, and controlled and uncontrolled overfall chutes. Consequently, the ogee crest has received much attention by researchers and its hydraulic characteristics are well understood.

The discharge over an uncontrolled ogee crest is influenced by a number of factors:

- Actual crest shape with respect to ideal nappe shape
- Ratio of actual head to design head
- Height of crest apex above the entrance channel invert
- Approaching flow velocity
- Downstream apron interference or tailwater submergence
- Upstream face slope

Thorough discussions of ogee crest design can be found in design manuals prepared by Reclamation (1987) and COE (Corps of Engineers) (1952).

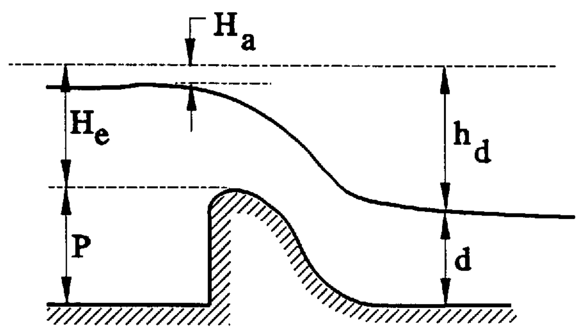
Hydraulic Design Considerations

Uncontrolled ogee spillway profiles are traditionally constructed to match the lower nappe surface produced by flow over a fully ventilated sharp-crested weir. Reclamation (1948) and many other researchers have measured lower nappe profiles and have developed design criteria for ogee crest geometry. A properly designed and constructed ogee crest shape will result in a discharge coefficient, $C=Q/(LH^{3/2})$, of 3.90 at design head, while atmospheric pressure is maintained on the spillway surface. However, for heads greater than the design head, subatmospheric pressure develops on the spillway crest, causing the discharge coefficients to increase.

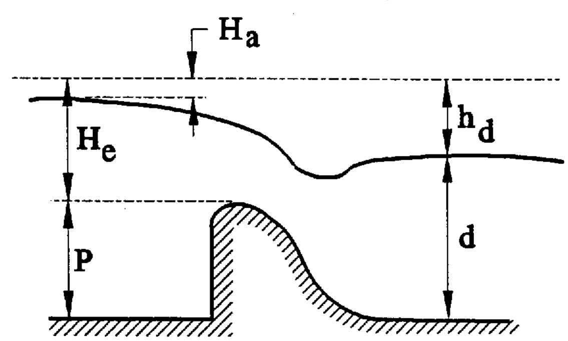
Understanding that greater spillway efficiency is possible by operating at heads greater than the design value has led Reclamation and COE to routinely "underdesign" ogee crests for heads equal to 75 percent of the maximum expected head. As a result, when an existing ogee crest is being evaluated for increased spillway capacity, care must be taken to determine the actual design head, which may not be the maximum head on the crest.

The upper limit of increasing discharge capacity is reached when the nappe springs free from the crest and becomes aerated. Research by Cassidy (1970) indicates that nappe separation can occur for heads greater than three times the design head. However, ongoing hydraulic model studies by Vermeyen indicate that under ideal entrance conditions (i.e. no contraction) discharge coefficients continue to increase for heads five times the design head. This condition is extremely unstable and nappe separation can occur from very small surface disturbances. Conditions such as a flow contraction, offset, or gate slot that will allow aeration may cause the nappe to prematurely spring free from the crest. If the air source is interrupted the nappe will reattach and may result in an oscillatory condition.

Another design consideration is the amount of free overfall to the downstream conveyance channel. Two conditions exist which can result in a reduced discharge coefficient: (1) the drop between the crest and downstream apron can be too small, and (2) tailwater can suppress the free overfall. Over many years, Reclamation's hydraulic research on ogee crests has resulted in design criteria for predicting reduced discharge coefficients related to apron elevation, tailwater depths, or a combination of these effects. These criteria can be found in Reclamation's manual *Design of Small Dams*, 1987. However, the apron design criteria can be summarized, using definitions in figure 5, as follows:



(a) APRON EFFECTS, $(h_d + d) \geq 1.7H_e$



(b) TAILWATER EFFECTS, $h_d \geq 0.7H_e$

Figure 5. - Design criteria for ogee crests with limited amounts of free overfall.

- Case (1) - To prevent back pressure on the crest because of insufficient free overfall, the downstream apron should be located such that the difference between the maximum reservoir and apron elevations, $h_d + d$, is at least 1.7 times the maximum reservoir head on the crest, H_e (including velocity head, H_a).

- Case (2) - To prevent tailwater suppression of flow over the crest, the difference between the maximum reservoir and maximum tailwater elevations, h_d, should be greater than 70 percent of the maximum reservoir head on the crest, H_e (including velocity head, H_a).

If neither of the above design criteria are achievable, then procedures outlined in *Design of Small Dams* are effective in estimating the discharge coefficient.

For heads greatly exceeding the design head, negative surface pressures develop on the crest. Negative pressures increases the cavitation potential, especially at joints, offsets, and surface irregularities. While cavitation can cause damage to a spillway surface, seldom does this result in structural failure. However, prolonged spillway operation during a major flood can result in loss of large quantities of surface material and costly repairs.

Structural Design Considerations

Structural design procedures for ogee crest structures are similar to most mass concrete structures and must be evaluated for instability caused by hydrostatic and uplift forces. This entails examining overturning potential - making sure to include negative surface pressures on the ogee crest. Likewise, sliding potential at high heads should be evaluated.

Anchoring and/or structural modifications can enhance stability of existing ogee crest structures that are unstable at high heads. These modifications include changing the crest shape, changing inlet or pier configuration, or using post-tensioned anchors for an additional stabilizing component. For instance, Reclamation has recently completed a similar application at Stewart Mountain Dam in Arizona (Bruce, 1991), in which over 80 post-tensioned tendons were used to stabilize the dam in the event of maximum credible earthquake loadings.

OVERTOPPING DAMS

Overtopping of both concrete and embankment dams is being evaluated by Reclamation as an emergency spillway alternative. In many cases, rehabilitating a dam or designing a new dam to withstand overtopping during extreme flood events is a cost-effective option. Overtopping an embankment or rockfill dam requires that erosion protection systems be designed to protect the crest and downstream slope. Such methods are currently the subject of an intensive research study by Reclamation.

Overtopping Embankment Dams

Currently the most widely used overtopping embankment dam protection method is RCC (roller-compacted concrete). RCC slope protection has been chosen for numerous small (< 150 ft high) embankment dams within the United States.

Although RCC is a proven embankment dam protection method, certain factors may discourage its use, especially on larger embankment dams. An engineer must ensure that the protective overlay will remain stable during overtopping. Any type of overtopping protection design employing an impervious material placed on the embankment must account for embankment drainage and prevention of uplift pressures. In the case of conventional RCC, both uncontrolled cracking of RCC and placement of drains require special attention. During overtopping, water passing through the embankment or through cracks in the RCC layer must be carried around or back through the impervious protection to prevent buildup of hydrostatic pressure. Obviously, venting drains directly through the protection will minimize expensive drainage collection and conveyance systems, which requires open drains underneath the overtopping flow. To prevent the drains from passing water into the embankment, a localized subatmospheric pressure zone must be created and sustained within the flow at the drain intake. As might be expected, a stepped geometry typical of RCC placement does create reduced pressure zones where the flow separates from the downstream step edge (fig. 6).

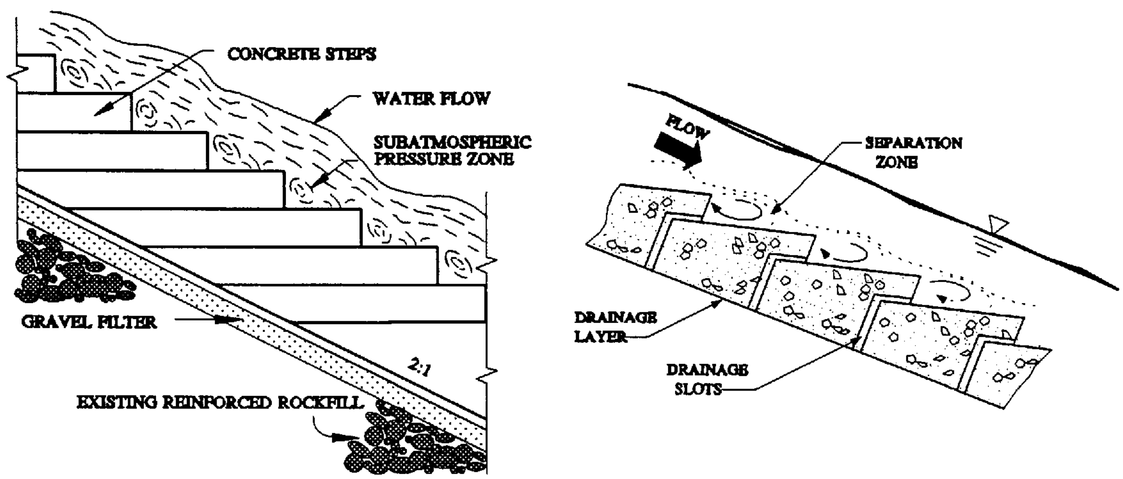


Figure 6. - RCC embankment overtopping protection and wedge-shaped revetment block design developed by Pravdivets.

However, recent investigations by Reclamation (Frizell, 1990, 1991) of horizontal steps on a 2:1 slope embankment show minimum surface pressures remain positive. The relatively flat angle at which the flow impinges on each horizontal step forces a strong component of the flow upstream into the eddy zone. This flow component limits the reduction in pressure that can be achieved on the step surface. Sloping the step surface in the direction of flow reduces the return flow component and thus provides a greater reduction in the surface pressure at the back edge of each step.

Pravdivets (1989) used this principle in the development of wedge-shaped revetment blocks. These blocks incorporated both a sloping step surface and subsurface drains to enhance block stability (fig. 6). Although the wedge block design has seen limited prototype testing in Russia and model testing by CIRIA (Construction Industry Research and Information Association, 1989), little formal development of hydraulic data covering flow over sloping steps has occurred.

Reclamation Research on Stepped Overlay Protection

Many problems associated with the use of RCC for overtopping protection are currently undergoing intensive investigation at Reclamation's Hydraulic Research Laboratory. The research objectives are to develop hydraulic design criteria for step tread slope in relation to embankment slope, overtopping depth, flow surface pressure, energy dissipation, and step stability. The research is not bound by the limitations of any single construction method. Methods such as slip forming, concrete paving, or a modified RCC may prove to be the most cost effective for constructing stepped overlays.

Laboratory Test Facility

Reclamation's laboratory work is being conducted using a 1.5-ft-wide Plexiglas-walled flume. To simulate embankment dam slopes, the flume can be set at any slope between 2:1 and 4:1. The facility allows investigation of model unit discharges up to 14 $ft^3/s/ft$ under reservoir heads up to 2.8 ft. The total drop from the reservoir to the controlled tailwater is 15.5 ft. Step surface pressure profiles and velocity profiles are measured as a function of step tread slope, overtopping depth, distance down the slope and embankment slope. Step surface pressure measurements are used to determine overlay stability and drain placement. Velocity profiles are measured to determine the dissipation of kinetic energy that occurs on the relatively rough stepped surface. All velocity profiles are measured, using a laser doppler anemometer, by traversing normal to the embankment starting at the downstream edge of step treads.

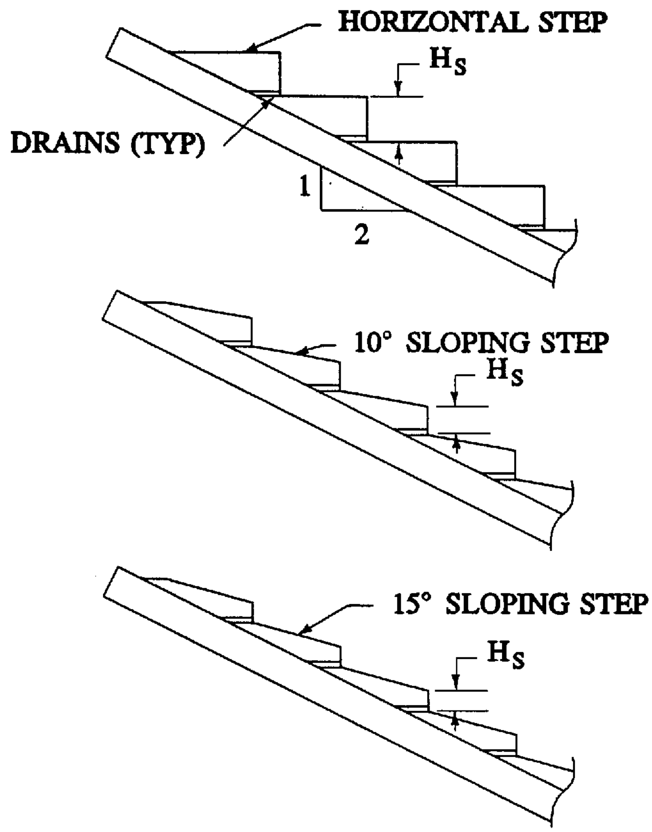


Figure 7. - Step geometries tested by Reclamation.

Results of Laboratory Tests for a 2:1 Embankment Slope

Although Reclamation's research on overtopping protection for embankment dams is far from complete, the results are encouraging. Investigations to define the hydraulics of flow over steps with different tread slopes for a 2:1 embankment slope are complete. Flume tests were conducted on horizontal steps, steps sloped 10° below horizontal, and 15° below horizontal (fig. 7). The results provide design data for drain placement, stability, and energy dissipation.

Step Surface Pressures, 2:1 Embankment Slope

Step surface pressures depend on step geometry, overtopping head, and distance down the slope. For all three step shapes, increasing the ratio of overtopping head (H defined as the total head measured at the dam crest) to step height (H_s defined as the vertical offset of each step) reduces the pressure drop in the separation zone. As shown in figure 8, surface pressures measured on horizontal steps remain positive. Therefore, the horizontal step geometry does not produce active aspiration of flow surface drains. Sloping the step tread 10° downward increases the pressure reduction. Under low overtopping flows, subatmospheric pressures develop just a few steps down the embankment slope. The number of steps or slope distance required increases proportionally with the overtopping head. Sloping the tread to 15° further enhances the development of subatmospheric pressure on the step surface. The steeper angle reduces the slope distance required prior to the occurrence of subatmospheric pressures. Steepening the step tread thus increases both the embankment coverage and overtopping head for which subsurface drains could be installed through an embankment overlay.

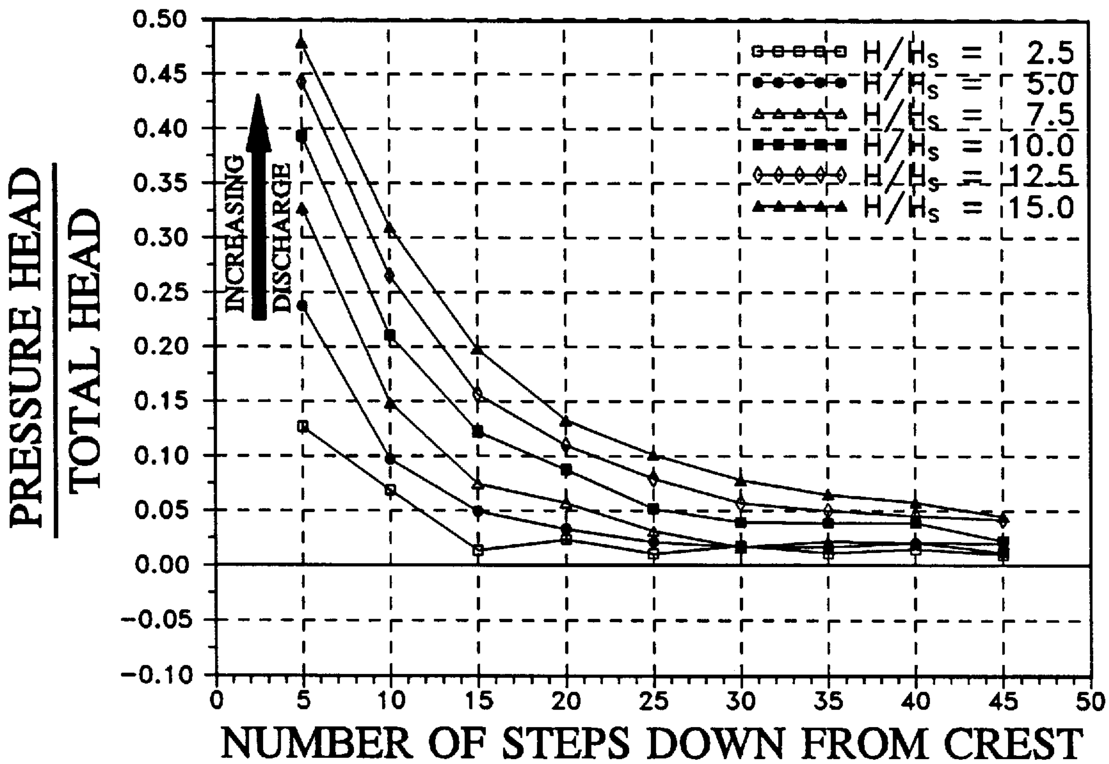


Figure 8. - Surface pressures on horizontal steps for several overtopping heads on a 2:1 slope

Steps are large surface roughnesses creating both areas of adverse pressure gradient and large scale turbulence. These conditions promote rapid self aeration of the flow profile from the surface. Air bubbles become entrained in the flow separation zones downstream of the steps when subatmospheric pressures exist. This strong entrainment mechanism limits the negative pressure drop that occurs. As air concentration increases, minimum step pressures approach atmospheric pressure.

Energy Dissipation on Stepped Overlay, 2:1 Embankment Slope

The energy dissipation of overtopping flow on an embankment slope must be considered in a spillway design. Energy dissipation may need to be minimized, as in the case of marginally stable structures where additional loading is undesirable. Consequently, a smooth deck-type protective overlay may be the only overtopping protection option. For highly stable structures, maximizing energy dissipation on the spillway may offer added benefits, like minimizing erosion at the embankment toe, thereby reducing stilling basin costs. Reducing the kinetic energy developed in flow down a steep embankment also reduces the need for downstream erosion protection.

The influence of a stepped geometry on the kinetic energy of the flow can be visually illustrated by comparing flow velocity profiles for different surface roughnesses under similar flows as in figure 9. The stepped surfaces sharply reduce near-surface flow velocities as compared to those on a smooth surface; horizontal steps show greater dissipation than 15° sloping steps.

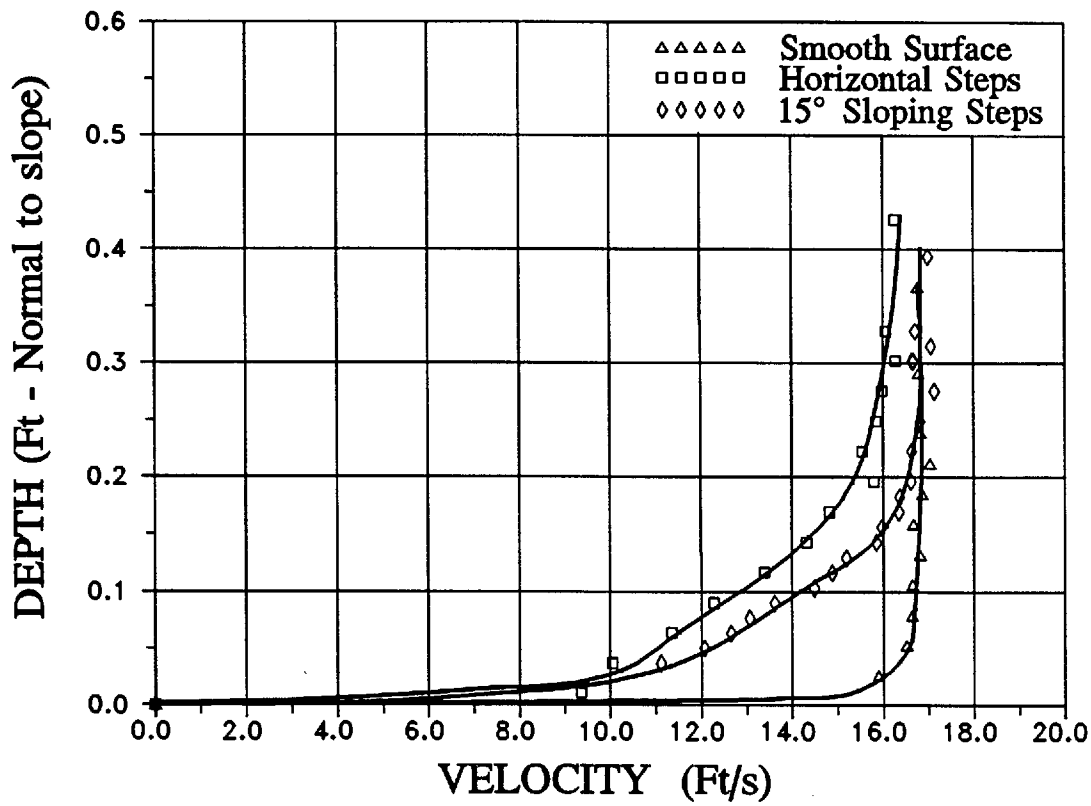


Figure 9. - Effect of stepped surface roughness on velocity profiles.

Overtopping of Concrete Dams

For large floods, the crest of a concrete dam can be used as an auxiliary spillway. If the depths, durations, and energy related to the overtopping is judged not to endanger dam stability, several methods can accommodate overtopping, such as, armoring the foundation rock, utilization of tailwater, and allowing a portion of the crest to overtop.

Armoring an abutment with conventionally placed concrete anchored into rock was used at Gibson Dam to provide overtopping protection. At Santa Cruz Dam in New Mexico, RCC was used to construct a gravity buttress and abutment armoring.

In general, it is advisable to restrict overtopping to areas where sufficient tailwater is available to provide adequate energy dissipation, especially where the foundation and abutment rock is erodible. The tailwater depth requirements can be evaluated in a manner similar to a plunge pool design. Raising tailwater by using weirs in the downstream channel has also been considered by Reclamation.

Restricting the location of overtopping can be accomplished using parapet walls to raise a portion of the dam crest. For example, Stony Gorge Dam was modified to provide additional discharge capacity by allowing overtopping on a portion of the dam and raising the parapet wall on the rest of the dam by 12 ft. The portion of the dam subjected to overtopping was armored with concrete.

SUMMARY

The problem of designing spillways to pass rare, extremely large flood events is receiving an increasing number of solutions. Unique and innovative approaches for designing and constructing limited-use spillways such as the labyrinth weir, fuse plugs, or overtopping protection are providing lower cost solutions. Auxiliary and emergency spillway costs no longer directly reflect spillway capacity. A labyrinth auxiliary spillway can save costs by reducing spillway width requirements and providing high discharge capacity under small increases in reservoir head. Existing ogee crest spillways may be required to operate at heads greater than design head. If so, structural stability and hydraulic performance for high heads must be carefully evaluated.

Emergency spillway options are typically the least-cost alternatives. If a portion of a dam can be designed as an emergency spillway, such as a fuse plug, the need to construct additional hydraulic structures and flood conveyance channels is eliminated. Research results show that stepped-type overlay protection for embankment dams offers additional advantages in drainage and energy dissipation of the overtopping flow. Step tread slope can be chosen to customize both drain aspiration and energy dissipation to meet the design needs of specific structures.

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