DESIGN OF A CONTINUOUSLY REINFORCED CONCRETE SLAB
TO PROTECT A. R. BOWMAN DAM DURING OVERTOPPING

By
Robert K. McGovern[1] and Kathleen H. Frizell[2]

ABSTRACT

Reclamation's Safety of Dams review determined that large flood events would overtop A. R. Bowman Dam and cause dam failure due to erosion of the downstream face. The method chosen to pass the flood is to allow overtopping of the dam and provide protection in the form of a continuously reinforced concrete slab (CRCS) on the entire downstream face of the dam.

This paper will discuss the critical considerations associated with the design of the slab and how a hydraulic model study was used to provide data for each phase of the design.

INTRODUCTION

Arthur R. Bowman Dam was completed in 1961, forming Prineville Reservoir with a capacity of 154,700 acre-feet. The dam is located on the Crooked River about 20 miles upstream from Prineville, Oregon, figure 1.

The dam consists of a wide central core, narrow transitional zones, and rockfill shells. The dam has a structural height of 245 feet and a hydraulic height of 158.8 ft. The crest width is 35 feet, and the existing crest length is 800 feet at elevation 3264. The total volume of material in the dam is 1,424,000 yd^3. The existing service spillway, founded on rock, is located on the right abutment and consists of a 20-foot-wide uncontrolled ogee crest at elevation 3234.8, leading into a concrete chute and stilling basin. The design capacity of the spillway is 8120 ft^3/s at water surface elevation 3257.9. The outlet works, located in the right abutment, consists of a trashracked drop-inlet structure, a 11-foot diameter concrete-lined circular tunnel, a gate chamber containing two emergency gates and two regulating gates, and a concrete-lined horseshoe outlet tunnel. The outlet tunnel discharges into the spillway chute and stilling basin. The outlet works has a capacity of 3,300 ft^3/s at normal water surface elevation 3234.8.

The new probable maximum flood (PMF) developed in 1988, has a peak inflow of 263,000 ft^3/s and a 15-day volume of 964,000 acre-feet. Occurrence of the PMF results in overtopping of the dam for about 4.5 days with a maximum overtopping depth of 20 feet. Overtopping of the dam will occur

[1]Civil Engineer, USBR, P.O. Box 25007, Denver, CO 80225

[2]Hydraulic Engineer, USBR, P.O. Box 25007, Denver, CO 80225

Figure 1. - View of A. R. Bowman Dam (Spring 1990).

for floods exceeding 23% of the PMF with a return interval of about 500 years.

Failure of the dam during the PMF would result from rapid erosion caused by the overtopping flows. Dam failure would result in both loss of life and excessive property damage. While an early warning system could be used to reduce the potential for loss of life, the decision to modify the dam to safely pass the flood was based on the potential for large economic loss.

ALTERNATIVES

Several alternatives were investigated for modifying the dam. These alternatives included raising the dam to contain the entire flood, adding an auxiliary spillway, combining a partial dam raise with an auxiliary spillway, and providing overtopping protection for the downstream face of the dam. The overtopping protection methods were selected over the dam raise alternatives based on economic analyses, and environmental considerations.

Five different methods of overtopping protection were investigated. These included a reinforced rockfill blanket and four concrete overlay alternatives. Reinforced rockfill was the least cost alternative, but was not selected because the current technology could not be reasonably extrapolated for a dam of this height and depth of overtopping. The four concrete alternatives investigated were a roller-compacted concrete (RCC) overlay with steps, an RCC overlay without steps, a continuously reinforced concrete overlay with steps, and a smooth continuously reinforced concrete slab. Information on the alternatives investigated

for providing overtopping protection for A. R. Bowman Dam has been well documented (Frizell, et al., 1990, and Hensley and Hennig, 1991).

Construction of a smooth CRCS on the downstream face of the dam was selected to prevent erosion of the dam during overtopping based upon the following economic and technical considerations:

- Most economical of all technically feasible alternatives.
- Cracking, thus seepage, could be controlled.
- Reinforcement would provide continuity across entire surface.
- Abutment protection excellent.
- Ability to provide drainage through surface.
- Good condition of the existing embankment.
- Good condition of upstream CRCS on other rockfill dams.
- Capability to obtain data and verify designs with a hydraulic model.

PRIMARY DESIGN CONCEPTS FOR THE CRCS

The layout for the CRCS for A. R. Bowman Dam is shown on figure 2. The slab has a minimum thickness of 1 foot and covers the crest and entire downstream face of the dam. The slab follows the existing slopes of the dam with a 2:1 upper slope and a 4:1 lower slope. The slab thickness was increased in the transition area between the slopes to add more protection against impacts from flood debris. The slab is restrained at the top and bottom by crest and toe blocks. The crest blocks provide restraint for the top of the slab and act as a barrier to prevent seepage from the reservoir from entering the downstream shell. The toe block is designed to anchor the CRCS to the foundation rock at the toe of the dam. The left edge of the slab will be anchored to the left abutment rock and the right edge will be restrained by a concrete gravity wall placed against the left wall of the existing spillway.

The design of the CRCS required investigation of many design details. The feasibility design of the CRCS was constructed in a 1:48 scale hydraulic model to aid in the refinement of the final layout. The model was used to investigate flow over the crest and slab, abutment treatments, and increased flow in the existing service spillway. An overall view of the A. R. Bowman Dam model is shown on figure 3.

The main concerns with the CRCS design and how the model study contributed to the solution of these concerns are listed in table 1.

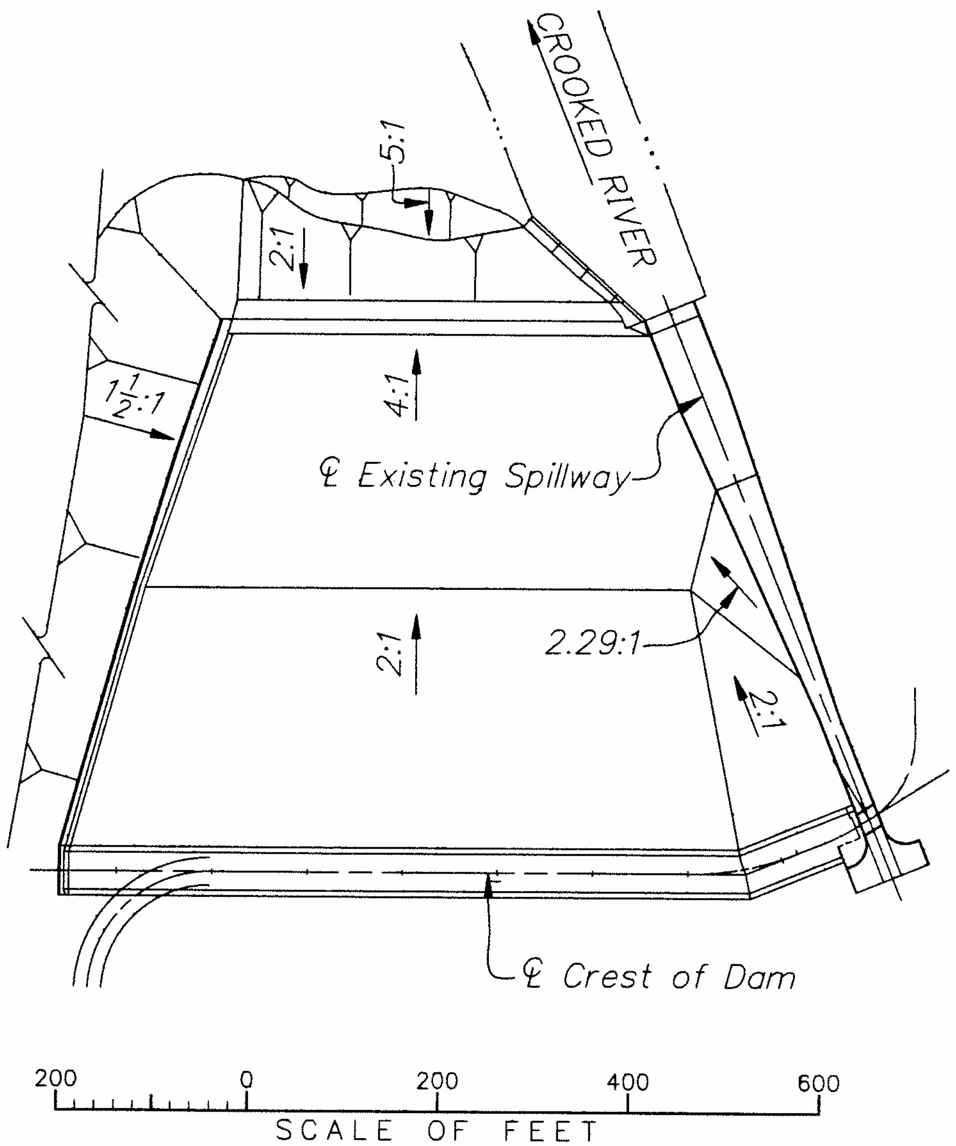

Figure 2. - Plan view of the CRCS on downstream face of A. R. Bowman Dam.

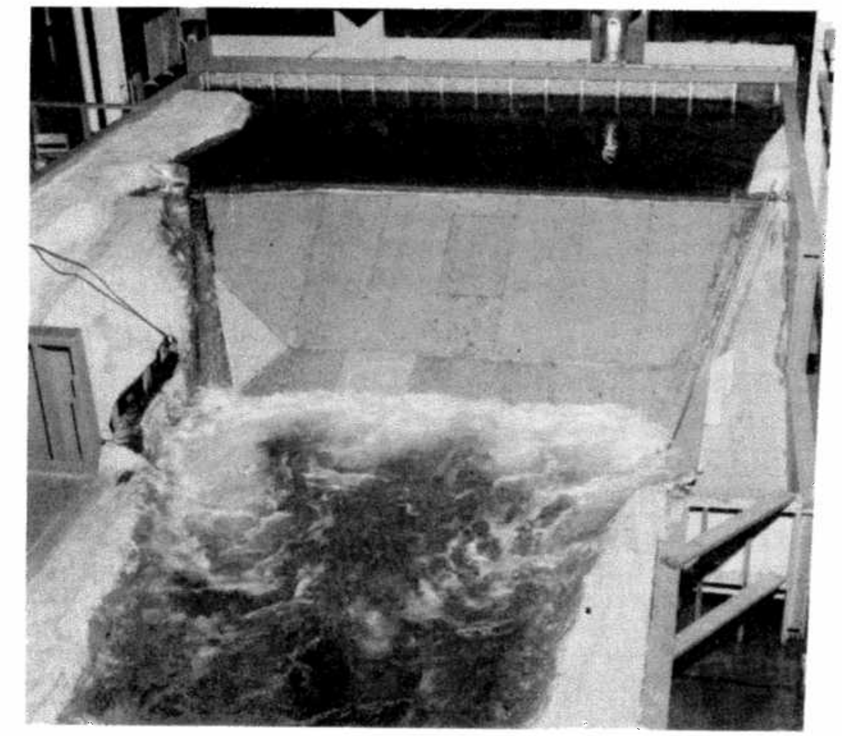

Figure 3. - Overall view of the 1:48 scale model of A. R. Bowman Dam with 15 feet of dam overtopping.

Table 1. - Summary of design concerns and study objectives.

DESIGN CONCERN	MODEL STUDY OBJECTIVES
Accuracy of discharge capacities.	Determine discharge curves for the existing spillway and dam overtopping.
Structural stability of existing left spillway wall.	Measure water surface profiles in the existing spillway.
Flow conditions on CRCS adjacent to existing spillway.	Determine flow conditions on the slab adjacent to the existing spillway.
Static water loads on the CRCS surface.	Measure water surface profiles over the downstream face of the CRCS.
Alignment and height of left abutment wall.	Measure water surface profiles and flow conditions along the left abutment wall.
Treatments needed to prevent flows over the right abutment.	Document flow conditions over the right abutment and methods to divert or contain flows.

DESIGN CONCERN	MODEL STUDY OBJECTIVES
Flow conditions with the bridge remaining during overtopping.	Determine effect of the roadway bridge on existing spillway and right abutment flows.
The eyebrow shape for the aspirating drains.	Measure effectiveness of several different drainage eyebrow shapes.
Location for drain outfalls on CRCS.	Define the location of the hydraulic jump on the downstream face of the CRCS.
Hydraulic loads on CRCS.	Measure dynamic pressures on the slab under the hydraulic jump.
Flow conditions of the existing spillway stilling basin wall and potential for erosion in the downstream river channel.	Measure wave heights and document flow conditions in the river channel downstream.
Erosion protection needed at toe of dam.	Observe flow conditions in tailwater at toe of dam.

The procedures that were used to design the CRCS for A. R. Bowman Dam can be applied to other dams. However, most of the results obtained were site specific to this project.

DESIGN OF THE CONTINUOUSLY REINFORCED CONCRETE SLAB

The main design concern with covering a rockfill dam with a concrete overlay is the ability the overlay to remain intact during the overtopping event. Several factors occur during overtopping which could result in failure of the CRCS. Seepage through cracks could accumulate under the slab and cause an uplift failure during the later parts of the flood if the tailwater levels drop below the water surface under the slab. Offsets in the slab caused by cracking, movement of the slab, or poor construction techniques, could cause failure by transferring the velocity head of the overtopping flow to stagnation pressures under the slab. Erosion of the abutments could undermine the CRCS and initiate failure.

The following sections will address the above items and the procedures or methods used to ensure passage of the PMF, minimize slab cracking, provide adequate drainage, and safely protect the abutments of the dam.

FLOW CONDITIONS AND DISCHARGE RATING

In order to design the CRCS, a better understanding of the flows over the dam and the hydraulic jump on the toe of the 4:1 slope was required. Flow conditions over the CRCS from the crest to the stilling basin were investigated using the hydraulic model. The flow converges down the entire face of the dam, bounded by the existing spillway on the right and

the contact with the rock on the left abutment. The crest length at the top of the dam from the existing spillway to the left abutment is 840 ft. The width at the toe of the dam is 380 ft.

Discharges were measured for each of the various flow areas: the existing spillway, the dam only, the existing spillway and the dam, and the existing spillway, dam, and right abutment combined. The rating curves for each of these areas are shown on figure 4. The discharge coefficient for dam overtopping flows was determined to be 3.08.

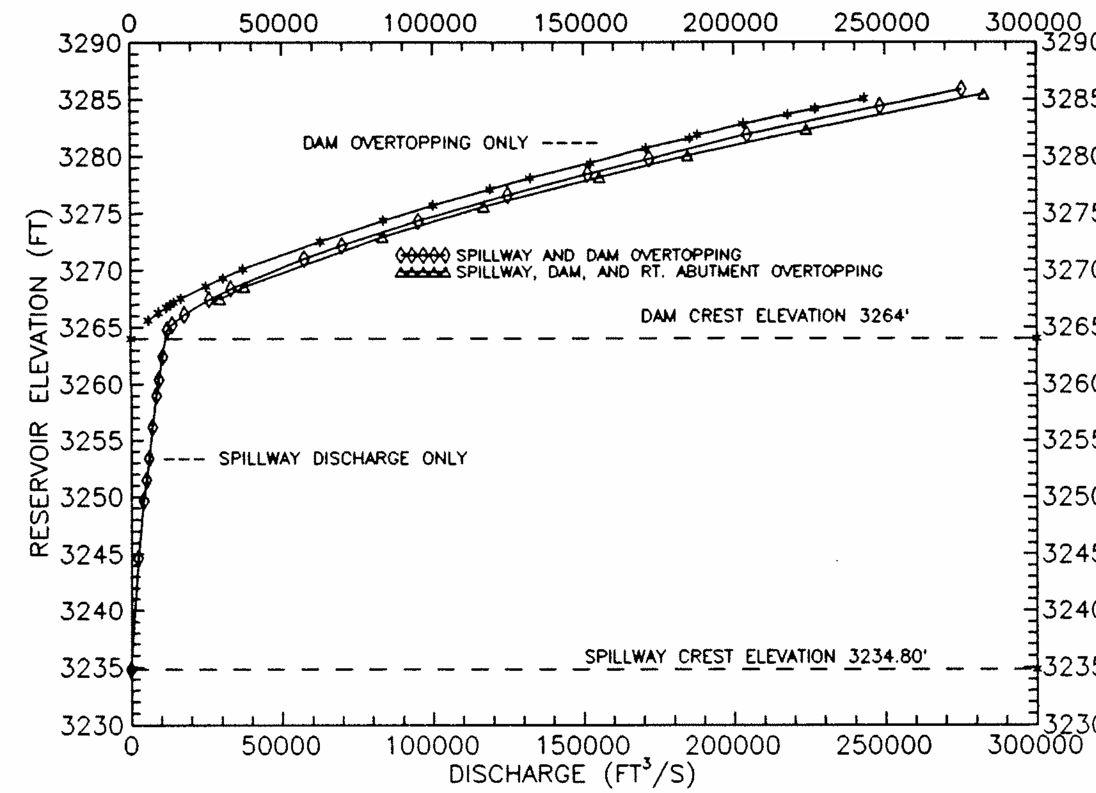

Figure 4. - Discharge curves for A. R. Bowman Dam.

The discharge rating curves obtained were used for the final flood routings. These routings indicated that the PMF would overtop the dam by 20 feet.

CRACK CONTROL ON THE CRCS

The ability to control cracks and offsets is critical to the design of the CRCS. Eliminating offsets prevents the buildup of stagnation pressures at joints and cracks and the potential for failure due to high pressures getting beneath the slab. Controlling crack size limits seepage through the slab during overtopping.

The CRCS is being designed using Continuously Reinforced Concrete Pavement

(CRCP) computer programs developed by the University of Texas (Ma, 1977). The programs model the response of the slab for various loadings based on the properties and dimensions of the concrete, on the gradation of the subgrade, and on limiting criteria for crack spacing, crack width, and steel stress.

To reduce the seepage through the slab, a target crack width of 0.003 inches at a slab temperature of 32 °F was selected. The 32 °F is considered the minimum slab temperature for computing crack width and seepage during the PMF.

Based on daily temperature data for Prineville, the minimum expected temperature is -33 °F. For a crack width of 0.003 inches at 32 °F, the corresponding crack width at -33 °F is 0.025 inches. This maximum crack width is a concern because of the potential for blowups in the slab which could occur if incompressible materials were to get into the cracks formed during these very low temperatures. In continuously reinforced concrete pavements (CRCP), this has only been a problem when crack widths exceed 0.1 inches (McCullough, 1991).

A limiting criterion for selecting the minimum crack width is the spacing of the cracks. For CRCP design, a minimum crack spacing is recommended to prevent punchout failures and to provide adequate development length between the cracks (McCullough, 1991). To meet the crack width criteria established for limiting seepage during the PMF at A. R. Bowman Dam, the minimum crack spacing was set at 2 feet.

Using these anticipated crack widths and spacings, the seepage volume through the slab during the PMF can be estimated assuming laminar flow through the cracks. The following equations were used to compute the seepage through the cracks (Amadei, et al., 1989):

$$\frac{Q}{w}=\xi\frac{gb^3}{12\nu C}\frac{\Delta h}{L}$$

$$C=1+8.8\left(\frac{k}{D_h}\right)^{1.5}$$

Q = discharge through the cracks (ft^3/s)
g = acceleration of gravity (ft/s^2)
b = aperture (width) of crack (ft)
ν = kinematic viscosity (ft^2/s)
Δh = difference in hydraulic head through crack (ft)
L = thickness of concrete (ft)
w = length of crack (ft)
ξ = degree of crack separation (varies between 0 and 1)
C = roughness coefficient
k = absolute roughness of crack wall surface (ft)
D_h = hydraulic diameter = 2b (ft)
k/D_h = relative roughness (varies between 0 and 0.5)
k/D_h = 0.5 was used because of the very small crack width relative to the size of the sand and aggregate in the concrete.

The above equations, along with water surface profiles obtained from the hydraulic model (figure 5) and downstream tailwater curves, are used to

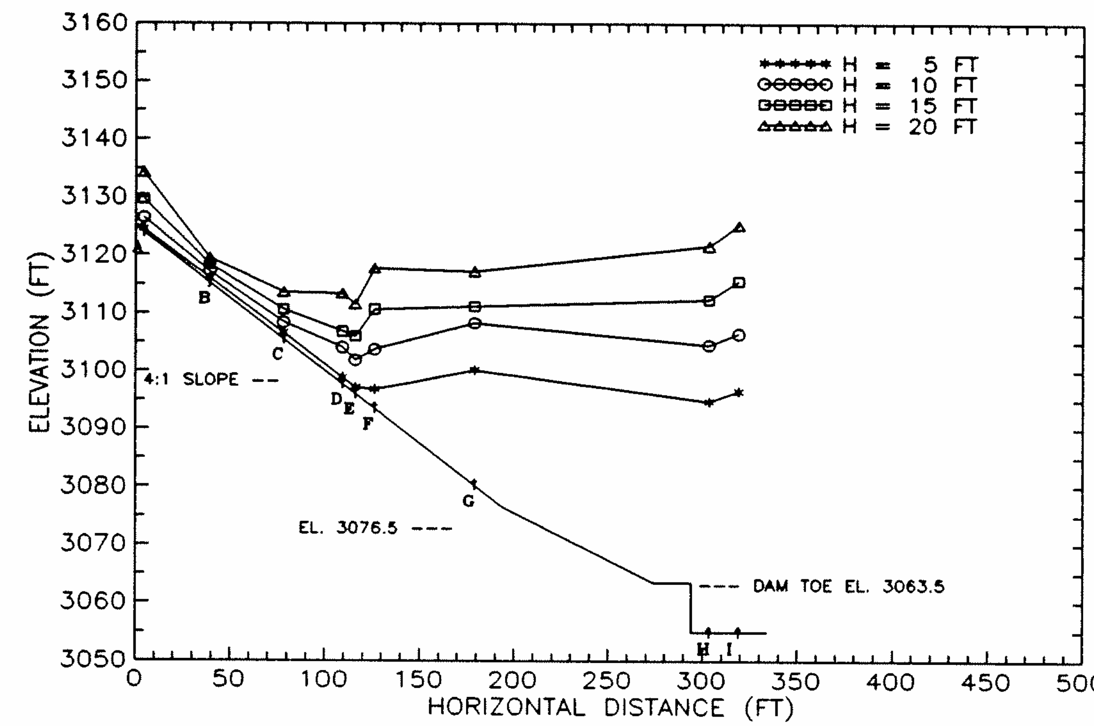

Figure 5. - Water surface profiles over the CRCS from the start of the 4:1 slope to the stilling basin.

determine seepage rates through the slab during the PMF. These rates, along with the accumulative volume, are plotted against time during the PMF in figure 6.

Seepage rate information and estimates on the volume of voids in the downstream shell of the dam are being used to determine the uplift loads on the slab. Because no high capacity drain outlets are located under or downstream of the hydraulic jump, any seepage occurring in this area collects in the shell of the dam. This causes no problem while the hydraulic head above the slab is greater than the uplift. During the latter part of the PMF, the tailwater level begins to fall while the level of seepage water below the slab stays almost constant. The potential for an uplift failure of the slab occurs when the tailwater level is below the water level under the slab. A plot of tailwater elevation and underslab seepage elevation against time is shown on figure 7. The curves show that the potential for an uplift failure does not occur until near the end of overtopping. Once overtopping has ended, an uplift failure of the slab would not be a hazard to the safety of the dam. We are currently in the process of refining our estimates of the seepage rates through the slab and the storage capacity of the dam material below the slab. This will include estimating the amount of seepage through the 2:1 slope portion of the CRCS which is not collected and discharged through the upper drainage

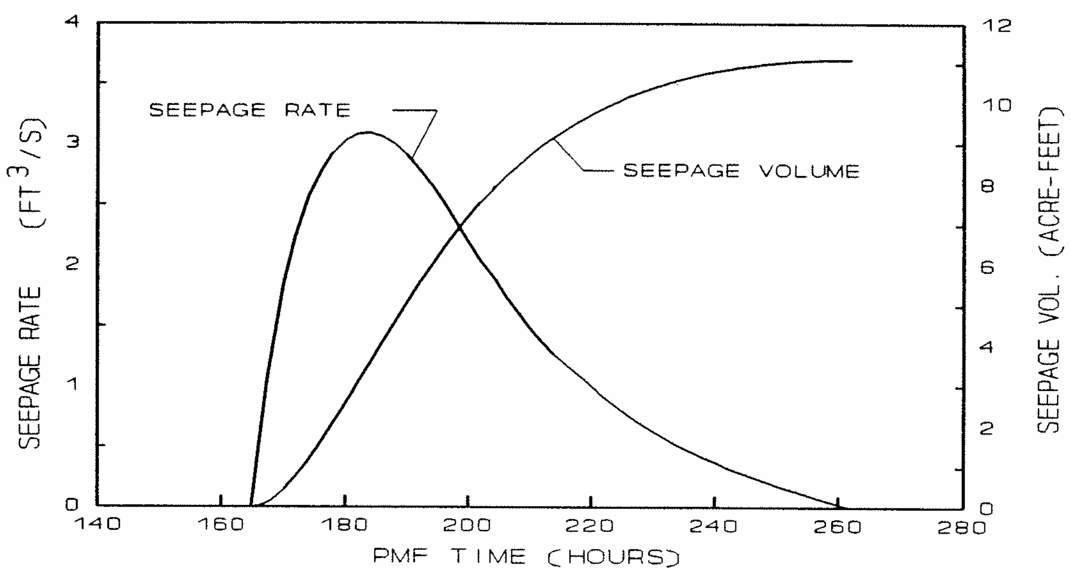

Figure 6. - Seepage through CRCS on 4:1 slope

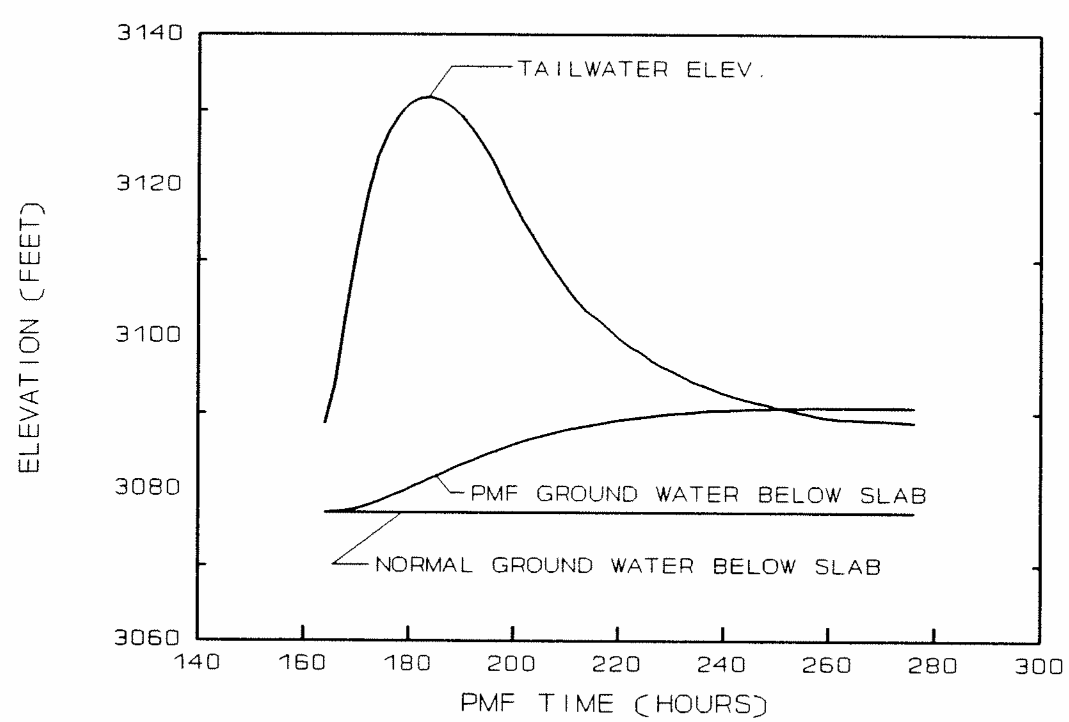

Figure 7. - Plot of tailwater elevation and groundwater elevation below slab during the PMF.

system and adding it to the seepage volume collecting under the slab. If the final analysis shows a potential uplift failure during overtopping, additional measures will be taken to protect the slab. These may consist of anchoring the slab to resist uplift, or treating the lower portion of the slab to reduce the seepage.

HYDRAULIC JUMP

Many stilling basins or chute slabs have failed due to pressure fluctuations being transmitted underneath the concrete surface either through poorly placed drains or offsets into the flow. The theory of the CRCS is to not allow pressure fluctuations or differentials to be transferred or built up underneath the slab. To ensure the integrity of the slab, flush mounted pressure cells were located in the model in the area of expected maximum pressure fluctuations underneath the hydraulic jump (Bowers, 1989). These were located on 4-ft centers (prototype) to determine the areal extent or possible periodic tendencies of the pressure fluctuations.

Simultaneous time series plots of the data show no consistent relationship between the two cells that would indicate anything more than instantaneous pressure spikes of short duration and over a small area, figure 8. There

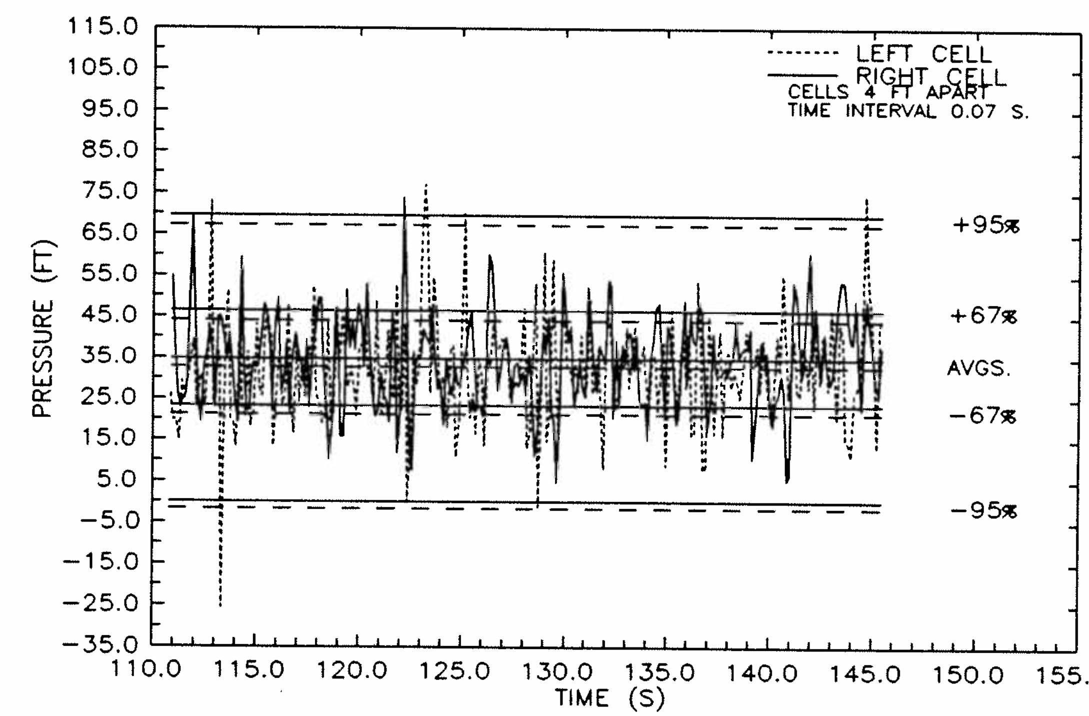

Figure 8. - Time tracing of the pressure fluctuations underneath the hydraulic jump for 20 ft of overtopping head.

were no cases where the maximum and minimum values were consistently occurring together nor cases where the extreme pressure spikes were more than instantaneous. Very few instantaneous pressures exceeded 3 standard deviations (95%) of the mean. The plot also indicates that except for a few random spikes, the load is in a downward direction and therefore would not contribute to an uplift failure of the slab. Further analyses will use these data to develop a response spectrum from the dynamic loads to be compared with the range of natural period of vibration expected for the slab.

Observations of flow conditions at the toe of the dam were used to determine areas that will require additional erosion protection. Due to the converging flow down the dam, greater flow depths and more turbulence is concentrated along the sides of the canyon. On the right, the existing stilling basin wall will need to be modified to withstand the hydraulic jump from overtopping flows. The left canyon wall will experience some higher flow velocities during the PMF.

DRAINAGE SYSTEM

An extensive drainage system will be designed for relieving possible uplift pressures due to seepage buildup underneath the slab. Before placing the slab, 5 feet will be excavated off the downstream face of the dam and a 3-foot drainage blanket placed. The drainage blanket will be used to collect seepage and direct it to drain outlets. The drains will be vented on the surface of the slab by the pressure differential created due to flow over the ramp or eyebrow located over the drain outlets. The pressure differential at the eyebrows also ensures that the outlets will not act as a source for additional water under the slab. The drains will vent to the surface just upstream or above the toe of the hydraulic jump. The location of the hydraulic jump was determined in the model study and is below elevation 3110.4 ft for all flow conditions.

In the lower portion of the slab, outlets are only located in the toe block. These drains are provided only for the steady state seepage through the dam. Flap valves on the outlets prevent backward flow from high tailwater during the flood. As an additional precaution, a header system will not be used for the lower drains. If the flap valves were to fail, the drains would act as point sources, allowing only minimal flow under the slab.

CRCS TIE-IN WITH EXISTING SPILLWAY

Another critical area of the design is the edge of the CRCS along the left wall of the existing spillway. The slab will be connected to a large gravity wall placed against the left wall of the existing spillway. The gravity wall will be used because it can be designed to withstand the restraint forces generated in the slab from temperature loads. The existing spillway walls are not strong enough to take either the restraint loads or the increased loads due to higher fill heights. In addition, the joints in the existing walls are not waterstopped and would provide an easy seepage path to the underside of the slab.

Flow conditions in the existing service spillway were investigated using the hydraulic model to ensure the integrity of the existing spillway when subjected to discharges far exceeding the original design capacity. The discharge in the existing spillway is increased from 8,120 ft^3/s at El. 3257.9 to 21,900 ft^3/s at El. 3284 or maximum dam overtopping. A pocket of air is trapped underneath the bridge that spans the spillway when overtopping begins. As overtopping increases, the bridge divides the flow and maintains the void underneath the bridge.

The model study showed that spillway flows will exceed the wall heights

and would quickly erode the backfill behind the right wall. Once the backfill is gone, the flows could start to undermine the spillway. Failure of the spillway foundation could extend to the dam and cause failure of the overtopping slab. To prevent this, the backfill behind the spillway wall will be removed and replaced with mass concrete. A protective layer of shotcrete will be added above the mass concrete to protect against flows coming down the abutment.

The model study also showed that flow from the CRCS impinged on the left wall of the spillway chute and created deeper flows on the slab along the wall. This problem was eliminated by the addition of a fillet on the slab adjacent to the existing spillway wall. The fillet spans the break in slope between 2:1 and 4:1, figure 2.

RIGHT AND LEFT ABUTMENT PROTECTION

The initial design of the CRCS had no method to protect the right abutment from overtopping flows. The left abutment wall was placed using the best estimate of the rock foundation underneath the alluvium. The model study provided the information required to design a method to protect both abutments from erosional damage which could undermine the CRCS and impact the safety of the dam.

Model operation showed that 7,100 ft^3/s would flow over the existing spillway bridge and the adjacent right abutment area during the PMF. This water would flow down both the existing state highway and over the rock abutment into the spillway chute. Because of the potential for erosion of the right abutment and the highway it was decided to divert flows from this area into the spillway chute. Model operation showed that the right abutment overtopping was most severe when it was assumed that the bridge spanning the existing spillway did not fail during overtopping. Therefore, this assumption was used in the design. A permanent structure across the crest of the dam could be used to divert all flows away from the abutment and into the spillway. However, a requirement to maintain access across the dam on the existing state highway eliminated that as a possible solution.

Several alternatives for diverting the right abutment flows were evaluated using the hydraulic model. The preferred alternative consists of the following two features:

- Construction of a wall following the shape of the existing rounded entrance on the right side of the spillway.

- Construction of a catchment basin or trough immediately adjacent to the spillway bridge. An erodible fuse plug embankment will be placed into the trough to support the highway.

In the model, the wall diverted the majority of the flow across the bridge and into the spillway. However, diagonal flow from the reservoir across the bridge still traveled down the highway and spread across the right abutment. To contain this portion of the flow, a catch basin was constructed. This trapped the remainder of the right abutment flows and

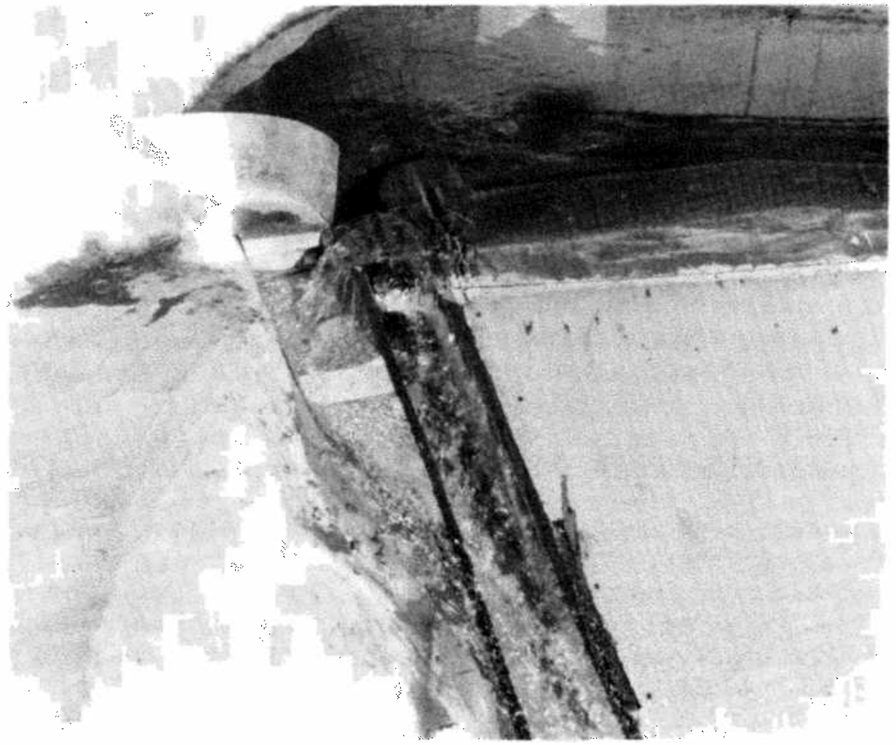

Figure 9. - View of right abutment wall and catchment basin containing flows associated with 10 feet of dam overtopping.

diverted them parallel to the existing spillway. The area parallel to the right wall of the spillway chute which would be susceptible to this flow is easily protected with shotcrete and the mass concrete which will replace the existing backfill behind the wall. These right abutment modifications, and how they function, are shown on figure 9.

The slab connection on the left abutment is much more straightforward. A wall will be placed along the left abutment for erosion protection. Several locations for the left wall were modeled. The final layout of the wall was determined by refining the estimated location of the contact between the abutment rock and the dam slope and attempting to minimize the change in the wall angle. Flow runup occurred due to the convergence of the wall and the change in angle of the wall at the break in slope. The height of the wall is based on the water surface profiles measured during the model study. The top of the wall is above the maximum water surface of the high velocity flows. A large block of mass concrete forms the base of the wall and anchors the CRCS. The block allows for the direct transfer of the restraint forces into the rock abutment.

CONSTRUCTION

Construction of the modifications is planned for two stages. In the first stage, a diversion wall will be constructed directly downstream of the left wall of the stilling basin. The wall is designed to allow normal

operation and passage of up to a 25-year flood during construction work at the toe of the dam. During Stage I, a majority of the earth work will be performed. This includes excavation of the face of the dam, stripping of the left abutment, and excavation of the rock at the toe of the dam. Exposure of the rock on the left abutment is a critical part of Stage I. Once the abutment rock is exposed, a final decision for its treatment will be made. This may include lowering the height of the left abutment wall. Additional Stage I work includes dam excavation and filter placement in the contact area between the dam core material and the downstream abutments. The filter is designed to prevent piping of the material originally placed between the zone 1 core material and the abutments. The need for this work was identified during the Safety of Dams review.

Stage II construction will include all the concrete placements for overtopping protection. The sequence during Stage II will be to construct the spillway gravity wall, left abutment wall, and dam toe block. Once these are complete, slipforming of the CRCS will be started. The CRCS will be placed from toe to crest in alternating panels. This will allow the initial expansion and contraction of the panels to take place before adjacent panels are placed. Placement of the final closure panel will be scheduled to produce minimal temperature stresses in the rest of the slab. Once the slab is placed, the crest blocks and crest roadway will be placed. A 20-foot high parapet wall on the right side of the spillway crest structure just upstream of the outlet works control house will be constructed. The spillway stilling basin will be modified during Stage II to correct a nitrogen supersaturation problem in flows exiting the existing basin. Concrete will be placed in the basin to raise the floor by 15 feet.

CONCLUSIONS

The use of a CRCS at A. R. Bowman Dam is an economical solution for safely passing large flood events by allowing overtopping of the dam.

The ability to control cracks and offsets was critical in the selection of the CRCS. Eliminating offsets will prevent the buildup of stagnation pressures at joints and the potential for failure due to high pressures getting beneath the slab. Controlling crack size will limit seepage through the slab during overtopping and the potential for an uplift failure due to high water levels under the CRCS.

The hydraulic model studies provided useful design information and were critical in determining the flow characteristics along the abutments and toe of the dam. The abutment treatments and crest and toe blocks ensure the integrity of the perimeter of the CRCS.

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STAGE 1- CONSTRUCTION

1993

Innovative Aspects of the Santa Cruz Dam Modification

Megan Metcalf[1], Timothy P. Dolen[2], and Paul A. Hendricks[3]

Abstract

Santa Cruz Dam underwent safety modifications in 1989. It is now capable of passing the probable maximum flood and withstanding the maximum credible earthquake. Modifications included construction of an arched roller compacted concrete buttress on the downstream face -- the first to be constructed in the United States. This paper will discuss three unique features of the modification: an inflatable vinyl "balloon" to form a drainage gallery within the buttress, the forming system used on the arched stepped spillway, and the inclusion of an air entraining admixture for freeze-thaw protection.

Introduction

Santa Cruz Dam is an arch/gravity structure located 26 miles northeast of Santa Fe, New Mexico. It was constructed in 1929 for irrigation control. The original dam (Figure 1) was 151 feet high and 50 feet wide at the base, with a 500-foot crest curved on a continuous radius. The overflow spillway near the center of the structure was 50 feet wide and had a capacity of only 1,450 ft^3/s before overtopping of the dam parapet occurred. The original outlet works consisted of a series of two 24-inch diameter gate valves housed on the downstream face of the dam.

The drainage basin for Santa Cruz Reservoir is located on the western slopes of the Sangre de Cristo Mountains and is 99 mi^2. The area is characterized by rough, rolling country that is covered with sparse pine forest in the upper

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[1] Civil Engineer, Concrete Dams Branch, Bureau of Reclamation, Denver, CO.

[2] Research Civil Engineer, Materials Engineering Branch, Bureau of Reclamation, Denver, CO.

[3] Chief, Construction and Engineering Services Branch, Bureau of Reclamation, Boulder City, NV.