RECLAMATION

Managing Water in the West

Report DSO-14-01

Construction Flood Case Histories

Dam Safety Technology Development Program

U.S. Department of the Interior
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Technical Service Center
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14. ABSTRACT
The purpose of this paper is to review case histories of significant floods that have occurred during construction of dam modifications, the meteorological and hydrologic conditions that occurred, and the response of the cofferdams or design features to the loading. The construction flood case histories include Jackson Lake Dam in 1986, Roosevelt Dam in 1993, and Glendo Dam in 2010 and 2011. Another case history is added (appendix A), which documents the Auburn cofferdam breach and the large flood event that caused the breach. The case histories are useful in reviewing the design floods selected, the cofferdam type and its performance, and in reinforcing the importance of evaluating risks during construction as presented in the Bureau of Reclamation’s Best Practices. The meteorological data (i.e., rain gauge observations, synoptic maps, National Center for Atmospheric Research/National Centers for Environmental Prediction reanalysis, and radar) were reviewed, and a summary of the meteorological conditions is presented.

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Dam Safety Technology Development Program

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Bureau of Reclamation
Dam Safety Technology Development Program
Technical Service Center

DSO-14-01

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Construction Flood Case Histories

I. Introduction

Flood conditions have occurred during construction at many of the Bureau of Reclamation's (Reclamation's) dams. As an example, in 1909, a large flood of record occurred at one of Reclamation's earliest projects that required heroic intervention. Inflows were 50 percent greater than previously recorded. By the middle of June, with the main dam partially completed (see figure 1), the reservoir continued to fill. It was concluded that the dike site was in danger of overtopping. Work to construct the dike was advertised, but all bids were rejected because the cost was considered too high. The use of Government forces was authorized, and two 10-hour shifts were used to construct a small, 4-foot-high "emergency" dike. The crisis ended on July 10, when the reservoir reached an elevation just 2 feet below the top of the emergency dike.

Figure 1. Partially completed masonry dam overtopping during construction.

In the past, risks during construction have primarily focused on damage to the work in progress. The contractor was given the historical flood inflow hydrographs and related data, and they designed the diversion capacity and cofferdam based on risks associated with damages to the construction. During the last 30 years very few new dams have been built by Reclamation, but many dams have been modified to address dam safety concerns. The modification of existing dams actually poses different and, in some cases, more significant challenges than those encountered during initial construction of projects because of the larger population centers that exist downstream of the dams.

The purpose of this report is to review case histories of significant floods that have occurred during construction by examining the meteorological and hydrologic conditions, the response of the cofferdams or design features to the loading, and any operational changes in response to the potential flood conditions. The case histories in this report include construction floods that occurred at Jackson Lake Dam in 1986, Theodore Roosevelt Dam in 1993, and Glendo Dam in 2010 and 2011. Appendix A presents the Auburn cofferdam case history, which documents the 1986 flood. The Auburn cofferdam case history is different than the others because construction was suspended for several years, and the contractor was no longer onsite when the cofferdam breached in 1986.

The case histories presented in this report are useful in reinforcing the need to evaluate risks during construction. Some additional considerations are presented in Reclamation's *Best Practices* [1] and Design Standards No. 14 - *Appurtenant Structures for Dams (Spillways and Outlet Works),* Chapter 2, "Hydrologic Considerations" [2]. The lessons learned from these case histories may help identify design considerations that can be used to mitigate potential risks during construction and to advance the development of the current methodology used to evaluate risks during construction.

Due to advances in meteorological forecasting capabilities, it is also important to understand the atmospheric and antecedent conditions that contributed to the flood conditions. This report includes selected meteorological data for each flood event. The meteorological data from rain gauge observations, synoptic maps, National Center for Atmospheric Research/National Centers for Environmental Prediction reanalysis, and radar, where available, were gathered and interpreted, and a summary of the meteorological conditions was produced.

II. Theodore Roosevelt Dam Modification

A. Background and Description

Theodore Roosevelt Dam is a concrete, gravity-arch structure on the Salt River, located approximately 76 miles northeast of Phoenix, Arizona, in Gila County. The original dam was a cyclopean masonry structure completed in 1911. Theodore Roosevelt Dam was identified as requiring dam safety modifications in 1978. The dam modification contract began in 1991 and was completed in 1996. The completed modifications to Theodore Roosevelt Dam increased the structural height of the dam by 77 feet with the construction of a mass concrete overlay on the downstream face and above the original dam crest. Modifications also included gated spillways (figure 2) constructed on both abutments within massive thrust blocks, and a new river outlet works constructed through the left abutment for reservoir evacuation. The original 36-megawatt powerplant was modified for operation under higher reservoir heads and tailwater, with a new penstock provided from the river outlet works tunnel.

Figure 2. Photograph of Theodore Roosevelt Dam under construction, showing the cellular cofferdam, thrust block, and spillway on left abutment. This photograph shows the construction progress after the flood event.

B. Theodore Roosevelt Dam Flooding Event: January 6-19, 1993

A series of high precipitation events occurred over central Arizona between January 6 and January 19, 1993. These events corresponded to record rainfall and high inflows into Theodore Roosevelt Dam. During these events, construction of the left thrust block was ongoing at Theodore Roosevelt Dam.

C. Meteorological Setup for the Event

A long-lasting El Nino phase of the El Nino Southern Oscillation (ENSO) was in place from 1990 until 1996. El Nino typically causes the jet stream to transport tropical moisture from near the Equator northward into the Desert Southwest. Even though only a weak El Nino was present during the winter of 1992-93, it was the impetus for the flooding event that occurred. In December, a series of Pacific storms, occurring at a rate of nearly one storm per week, caused saturated soil conditions across much of central Arizona. As a result, antecedent conditions existed for the January 6 to19 flooding event at Theodore Roosevelt Dam, which included saturated soils and a slightly above average snowpack in the mountains above 6,000 feet.

The flooding event featured four main storms that caused the majority of the precipitation that occurred over the Theodore Roosevelt Dam watershed. All four storms are considered Atmospheric River events (narrow corridors of water vapor transport, usually a few thousand feet above the surface of the Earth). Atmospheric River events are characterized by intense, widespread rainfall that is often caused by the forced ascent of the water vapor band by a topographic boundary. On January 6, a large storm fueled by the subtropical jet stream moved onshore in California. The subtropical jet stream was able to carry ample amounts of moisture into the Desert Southwest of Arizona. As a result, the snow level rose above 8,000 feet from the initial 6,000 feet over much of Arizona, which resulted in a rain-on-snow event for the first portion of this heavy rain event. The Snow Telemetry (SNOTEL) site near Workman Creek, Arizona (elevation 6,900) recorded that, during this timeframe, all of the previous snow melted due to the rain [3]. Given the antecedent conditions of saturated soils, the rain-on-snow produced abundant and rapid runoff. Many locations received between 2 to 4 inches of rain with the first storm.

With the arrival of the next storm system, around January 10, the snow level dropped back down below 7,000 feet and remained there through the rest of the event, which caused the snowpack to begin building again. This storm system brought widespread rainfall amounts of 1 to 3 inches across much of Arizona below 7,000 feet. The third storm system arrived around January 13, and rainfall amounts generally less than 1.5 inches fell over the watersheds. The final storm system arrived around January 18 and lasted only about 24 hours. It produced rainfall amounts in the 1- to 3-inch range. The last three storm systems caused the snowpack, especially at elevations above 9,000 feet, to become quite large, with point rainfall return periods calculated as high as 50 to 60 years [4].

It is common for Arizona to receive relatively large amounts of precipitation during the winter (December, January, and February). On average, Theodore Roosevelt Dam receives nearly one-third of its annual precipitation during these 3 months. Only during the month of August in the Monsoon season does it receive more rain (figure 3) [5]. However, the January precipitation totals from 1993 were from 3 to 5 times the normal amount of precipitation that occurs in an average January (see figure 4) [6, 7, 8]. At Theodore Roosevelt Dam, the 14-day rainfall total was 10.81 inches, which corresponds to a return period of about 200 years [4].

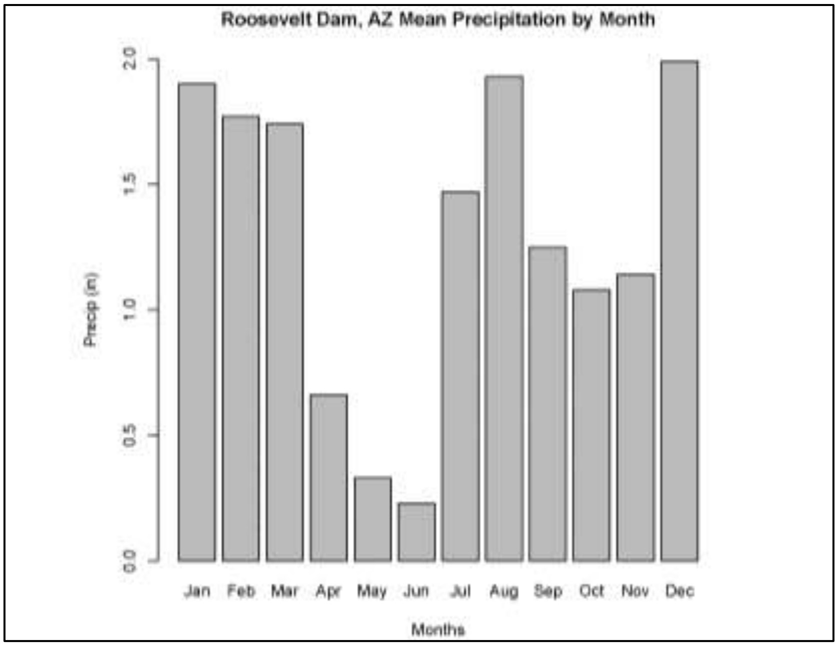

Figure 3. Average monthly precipitation for Theodore Roosevelt Dam, Arizona [5].

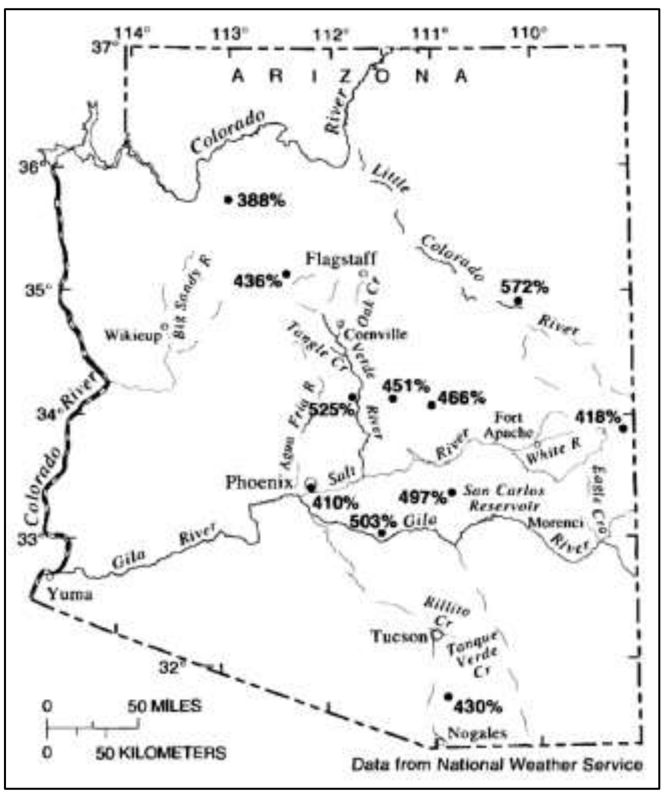

Figure 4. January 1993 precipitation as a percentage of normal January precipitation (1931-1960) [6].

D. Diversion Plan and Performance of Cofferdam

Record rainfall associated with storms that occurred in late December 1992 and January 1993 forced the operation of the eight available existing spillway gates on the right abutment. On January 8, 1993, a peak inflow of 135,800 ft^3/s occurred, which was the flood of record. The storm resulted in a maximum 5-day volume of 580,000 acre-feet [9]. A subsequent storm caused the reservoir to rise to elevation 2,139.1 feet on January 19, which was the highest reservoir elevation in the history of the dam.

The diversion plan for Theodore Roosevelt Dam consisted of staged construction, so that one spillway structure was available at all times to provide protection for up to a 25-year flood event; cellular cofferdams to provide protection for the spillway and thrust block under construction; a reservoir drawdown of 21 feet from the crest of the cofferdam to provide about 341,000 acre-feet of storage space; and provisions for diversion flows through a 129-inch-diameter, diversion pipeline extending downstream about 530 feet from the river outlet works control structure. The 25-year design flood had a peak inflow of 122,000 ft^3/s and a 5-day volume of 360,000 acre-feet. The staged construction of the thrust blocks and spillways allowed releases from the existing right spillway gates until the left thrust block and spillway were completed. The diversion pipeline capacity was 6,000 ft^3/s and permitted releases around the unwatered tailrace area during construction.

The cellular cofferdam consisted of three cells and two connecting arcs with two connecting concrete gravity walls on each end for closure. The cells were 37.6 feet in diameter, and the arcs had a radius of 11.4 feet. The cellular cofferdam sheet piles were installed with a top elevation of 2138, and the sand fill was capped with a 1-foot-thick, concrete slab reinforced with welded wire fabric. The concrete cap was at elevation 2136 and was provided to prevent erosion of the sand within the cellular cofferdam in the event that overtopping of the cofferdam structures occurred. The cellular cofferdam incorporated a couple of features that were noteworthy. Concrete toe buttresses were constructed on the downstream sides of the cellular cofferdams to provide support of the cells and provide some protection for overtopping. Remote monitoring of piezometric levels within the cells was also performed to ensure that design assumptions were not exceeded during overtopping. The gravity walls were constructed to elevation 2136, with a 2-foot-high parapet wall on the upstream side.

In late December 1992, a winter storm caused the reservoir to rise to an elevation 28 feet below the crest of the cofferdam. This storm produced about 2.3 inches of rain, which fell on the 5,762-mi^2 drainage basin. This additional rain elevated the reservoir levels and saturated the soils in the drainage basin. On January 7, with the reservoir at elevation 2114, a second and more significant flood event occurred during construction of the left thrust block. The storm dumped about 6 inches of rain on the drainage basin over the next 5 days. Anticipating that the eight spillway gates on the right abutment would need to be opened, the downstream cofferdam crest was lowered by 6 feet, and all equipment and materials were removed from the area downstream of the dam.

Then, on January 8, 1993, the right spillway gates were opened with the reservoir at elevation 2125.7. On the left abutment, where the upstream cellular cofferdam had been constructed, the existing spillway had been removed, and mass concrete placements on the left abutment thrust block had already begun. Block 18 had been placed to elevation 2130; Block 19 had been placed to elevation 2076.85, which included the chute and flip bucket; Block 20 had been placed to elevation 2130; and Block 21 was placed to elevation 2110.

When the reservoir water surface exceeded elevation 2138 on January 19, overtopping of the left abutment concrete-capped cellular cofferdam and left thrust block began (see figures 5-7). The cofferdam overtopped for about 2 days and ended about midnight on January 21 [10]. The reservoir reached the highest elevation since the original construction of the dam. The flow over the cofferdam and partially constructed spillway was estimated to be about 700 ft^3/s.

Figure 5. Theodore Roosevelt Dam during 1993 flood showing partial construction of the left thrust block and overtopping of the cellular cofferdam.

Figure 6. Aerial view of cellular cofferdams being overtopped in 1993.

Figure 7. Aerial view of service spillway operating on the right abutment and flows from the cellular cofferdam overtopping on the left abutment during 1993 flood event.

The concrete-capped cofferdam performed fairly well for the approximately 1.5-foot depth of overtopping (see figure 6). The cellular cofferdam sustained minimal increase in deflection and bulging, none of the sheet piles went out of interlock, and the cone-shaped wire mesh inserted in the weep holes helped to minimize sand loss during the overtopping. Releases through the existing right spillway and outlet works continued until the reservoir was lowered. Releases from the right existing spillway gates were made again from February 12 until March 8, 1993, as flooding of the work site continued.

The flood caused a delay in construction and resulted in damages to the construction site. The dam construction was completed in 1996 (see figure 8).

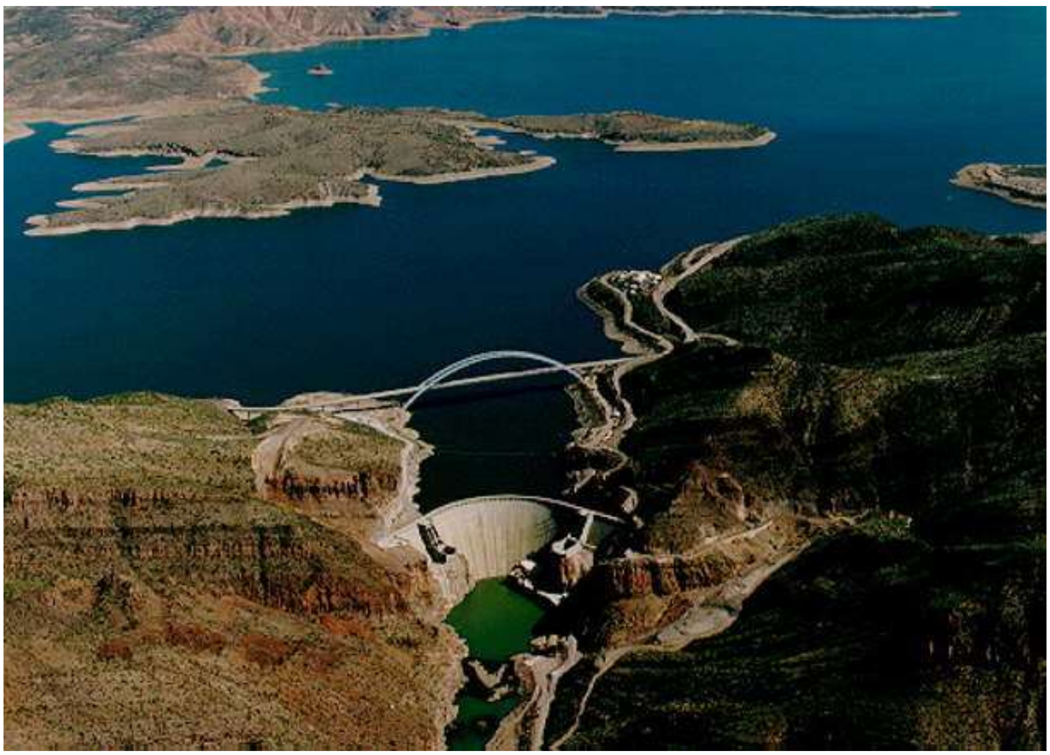

Figure 8. Aerial view of completed Theodore Roosevelt Dam.

III. Jackson Lake Dam Modification

A. Background and Description

Jackson Lake Dam is a composite earthfill embankment and concrete gravity structure on the Snake River in northwestern Wyoming, approximately 30 miles north of the town of Jackson. The dam was originally constructed as a temporary timber-crib structure between 1905 and 1907 to enlarge a natural lake. The temporary structure failed in 1910 and was replaced with a permanent dam between 1910 and 1911. Modifications to the dam were completed in 1916.

Safety concerns were identified at the dam in the mid-1970s, and the reservoir level at Jackson Lake was restricted to a lower than normal level from 1977 to 1989 because of concerns about potential dam failure during an earthquake. The dam was modified between 1986 and 1989. In its current configuration, it consists of a concrete gravity section flanked by a zoned earthfill embankment on its left (north) end and a short earthfill embankment on its right (south) end. The dam foundation was treated using a technique called dynamic compaction, and a grout curtain was installed below the foundation.

B. Jackson Lake Dam Flooding Event: June 1 to 14, 1986

A high water flow event occurred at Jackson Lake Dam in Wyoming between June 1 and June 24, 1986, while the dam was being modified. This event corresponded to rapid snowmelt due to warm temperatures around Grand Teton National Park and the higher elevations just south of Yellowstone National Park.

C. Meteorological Setup for the Event

A large snowpack had set up from the winter of 1985 into the spring of 1986 over the headwaters of the Snake River in northwest Wyoming. This was due, in part, to the ENSO that was in a La Nina phase (cooling waters off the coast of South America into the central Pacific Ocean). In response to La Nina, the jet stream moved farther south than normal and funneled moisture over the Pacific Northwest, which continued east into western Wyoming. The large snowpack persisted into June at elevations above 8,000 feet. The return period of the snowpack above 8,000 feet for the beginning of June was roughly 4 to 7 years [11].

The end of May and beginning of June 1986 featured a heat wave for portions of the inland northwest and northern Rocky Mountains. Many locations had high temperatures in the 90's, and a few even had 100-degree readings [12]. The higher elevations near Jackson Lake Dam had high temperatures in the 70's and 80's, with low temperatures staying mostly above freezing, which led to rapid melting of the large snowpack. A stationary boundary moved over northern Wyoming on June 4, which helped to abate the heat and bring isolated to scattered thunderstorms over the watershed. The stationary boundary caused a rain-on-snow condition above 8,000 feet, which lasted through June 15 in isolated locations at higher elevations. By June 24, the main threat of flooding had diminished because most of the snow had melted, even though isolated thunderstorms persisted. The heaviest recorded 4-day rain near the Jackson Lake

Dam basin was 1.99 inches, which is approximately a 2-year event [13]. This flood event was mostly predicated by rapid snowmelt, rather than heavy rainfall (see figures 9 and 10).

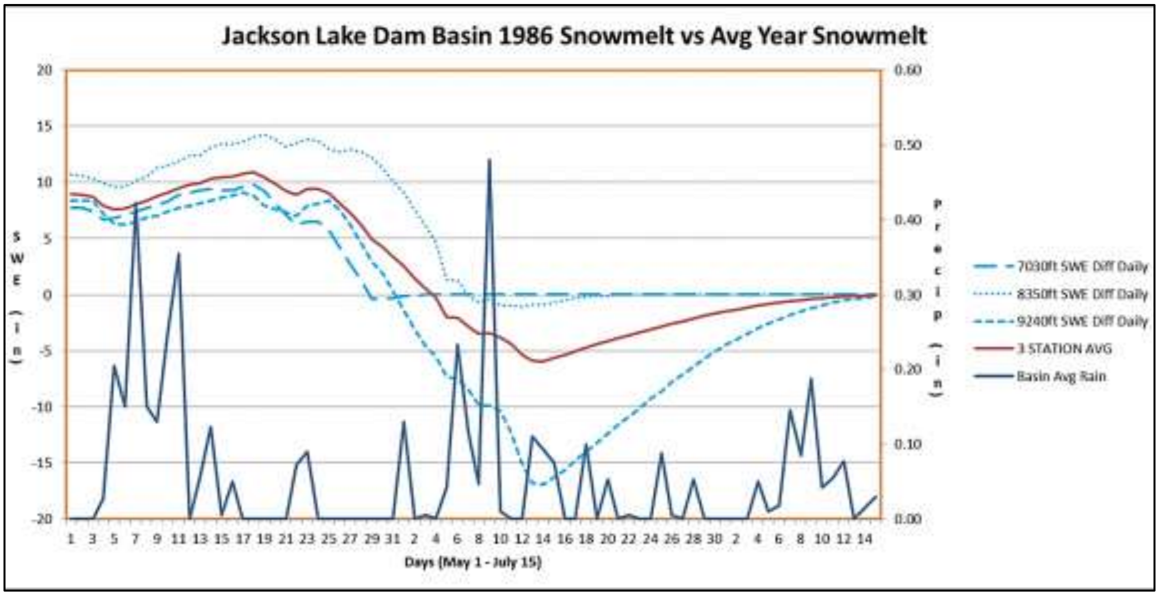

Figure 9. Graph showing 1986 snowmelt (light blue) for three stations with different elevations plotted against the average snowmelt for the year on the left axis for the May 1 to July 15 time period. The Snow Water Equivalent (SWE) above the 0 value represents greater than **normal snowpack, while below the 0 value, it represents less than normal snowpack. Basin average precipitation (dark blue) is plotted on the right axis for the same time period.**

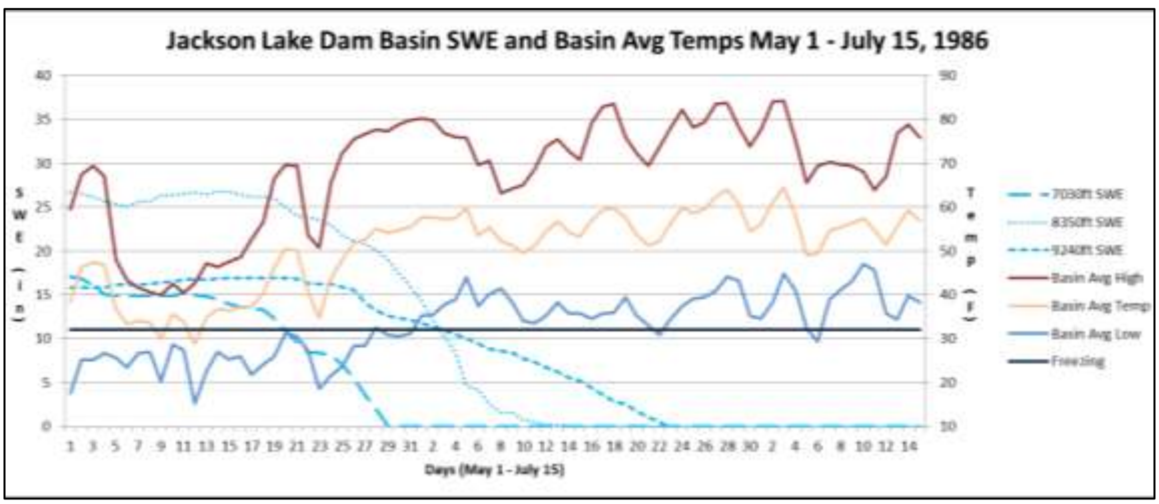

Figure 10. Graph showing the snowpack (light blue) in inches on the left axis for the May 1 to July 15 time period. High temperature, average temperature, low temperature, and 32 degrees are plotted on the right axis for the same time period. Notice how quickly the snowpack begins to melt once the average basin temperature rises above 32 degrees.

D. Hydrologic Loading, Diversion Plan, and Performance of Cofferdam During Construction

The diversion plan during construction was to have a cofferdam that could be installed in the wet, allow for staged construction of the left and right sides of the concrete gravity section, and allow for diversion of flows by utilizing the existing outlet works without restricting the approach channel area and the required discharge capacity. Due to the space limitations and high localized velocities, a cellular cofferdam was selected with embankment tie-in sections. The diversion plan with the cellular cofferdam provided definite advantages associated with the construction schedule during stage 2 construction. The contractor elected to use a double sheet pile wall to tie into the cellular cofferdam. The center cofferdam consisted of three 40-foot-diameter cells and two 11-foot-radius connecting arcs with a connecting concrete gravity wall for closure.

Flood routings were performed for both spring runoff and thunderstorm flood conditions to evaluate flood control operations, to determine specification requirements for operation of the existing outlet works, and to determine the level of flood protection and height of the cofferdam [14]. It was concluded that dam operations needed to be similar to normal operations, except that the reservoir would be operated at a lower reservoir level. The diversion plan included 200,000 acre-feet of flood storage to account for spring runoff conditions and to allow some flexibility in managing releases to allow downstream tributary flows to pass prior to increasing releases from Jackson Lake Dam. It was necessary for the outlet works discharge capacity to be at least 9,000 ft^3/s during the spring flood operating season between May 1 and July 10.

Spring runoff floods with return periods of 5 years to 100 years were routed. The 100-year spring runoff flood (based on the 1918 flood) had a peak inflow of about 15,000 ft^3/s and a 15-day volume of 388,000 acre-feet. The flood routing for the 100-year spring runoff flood resulted in a freeboard of a little less than 5 feet. The results of the spring runoff flood routings indicated that the difference in maximum reservoir elevation between the 25-year flood and the 100-year flood was only 1 foot, which supported the decision to increase the flood protection level to a 100-year return period because the increased cost would be insignificant compared to the increased flood protection.

The 100-year thunderstorm flood routings did not include any flood storage space prior to the flood since thunderstorms could occur after July 10, which is historically the end of the spring runoff time period. As a result, the 100-year thunderstorm flood controlled the design of the cofferdam crest elevation, and the estimated freeboard for this flood was 2.4 feet.

For diversion during stage 1 of construction, the existing outlets were made available for spring runoff floodflows, and the specifications required that the tie-in cofferdam, which would restrict the number of outlets available, could not be constructed until after July 10 or be removed by May 1 of the following year (see figure 11). In addition, consideration would be given to breaching the tie-in cofferdam if more discharge capacity was needed after the July 10 date.

Figure 11. Aerial view of cellular cofferdam upstream of Jackson Lake Dam during the first stage of construction after July 10.

For diversion during Stage 2 of construction, 11 outlets were available, which satisfied the 9,000-ft^3/s discharge capacity requirement (see figure 12).

The maximum inflow during the 1986 to 1989 modifications was 17,800 ft^3/s, which occurred on June 2, 1986. Based on the peak inflow data, this flood runoff was estimated to have a return period of about 500 years [15]. Snowpack conditions were well known in advance of the peak runoff and additional storage space was created in the reservoir in anticipation of the large runoff in addition to the planned 200,000 acre-feet of flood storage. Releases from Jackson Lake Dam were also successfully managed to allow downstream tributary flows to pass. The highest reservoir water surface elevation reached during construction resulted in about 2.6 feet of freeboard remaining on the cofferdam. The diversion plan worked well in allowing staged construction, while still providing protection for the flood of record. The cellular cofferdam was inspected during construction and performed as designed throughout construction, including the flood conditions in 1986.

Figure 12. Aerial view of cellular cofferdam upstream of Jackson Lake Dam during the second stage of construction.

IV. Glendo Dam Modification

A. Background and Description

Glendo Dam and Reservoir are located on the North Platte River about 4-1/4 miles southeast of Glendo, Wyoming. The reservoir is used for flood control, irrigation, and fish and wildlife enhancement. The construction of the dam, dikes, and appurtenances was completed in 1959. Glendo Dam is a zoned earthfill structure. Three dikes are located in the low areas between the hills of the reservoir's south shore. In 1989, a seepage berm was constructed on the downstream toe of the dikes.

The contract for construction of a new uncontrolled ogee spillway at Glendo Dam was awarded in 2009, and the contractor had already begun some of the Safety of Dams construction work in 2010. In advance of construction, a risk during construction report and an interim construction Emergency Action Plan (EAP) were prepared. The interim construction EAP was prepared to supplement the existing EAP, to identify potential failure modes specific to construction, and to include the line of communication in the event of an emergency. In addition, decisionmakers participated in a conference call every 2 weeks to coordinate construction issues, which facilitated discussions and decisions related to concerns about the potential for rising reservoir levels.

B. The Floods of 2010 and 2011

A high precipitation event occurred over the North Platte River watershed west of Glendo Dam, Wyoming, between June 10 and June 20, 2010. This event corresponded to high discharge flows from Seminoe and Pathfinder Dams, as well as Glendo Dam. During this event, construction was ongoing at Glendo Dam. In 2011, a record snowpack developed in the basin upstream of Glendo Dam.

1. Meteorological Setup for the Event

During the summer of 2010, the ENSO phase was transitioning from an El Nino phase (warm ocean temperatures off the coast of South America into the central Pacific Ocean) to a La Nina phase (cool ocean temperatures off the coast of South America into the central Pacific Ocean; see figure 13) [16]. The El Nino phase during the winter and spring allowed a large snowpack to build in the Rockies of Colorado and Wyoming and also allowed cooler than normal temperatures to continue into the summer. The cooler than normal temperatures led to a snowpack above 9,500 feet, which persisted into June. As the ENSO phase transitioned from El Nino to La Nina during the early summer, the jet stream (relatively strong winds concentrated within a narrow stream in the atmosphere typically above 30,000 feet) transitioned and funneled more Pacific moisture over the State of Wyoming. As such, the soil for the North Platte River basin was saturated when the high precipitation event occurred from June 10 to June 20, leading to substantial amounts of runoff, including a rain-on-snow component above 9,500 feet.

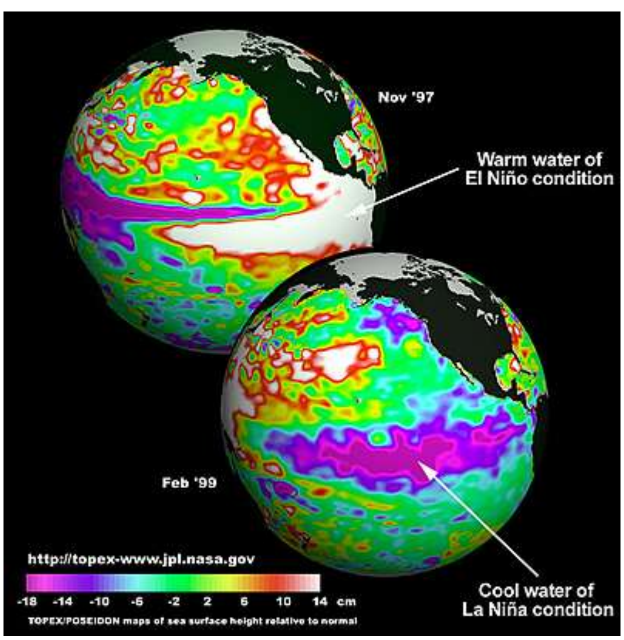

Figure 13. El Nino and La Nina conditions.

A series of upper level lows crossed northern Colorado and Wyoming starting on June 7 and continuing until nearly the end of the month. Due to these upper level lows, stratiform and convective precipitation occurred almost daily over some portion of the North Platte River basin west of Glendo Dam from June 7 until June 17. The heaviest precipitation that fell near Glendo Dam occurred approximately 25 miles south, near Wheatland, Wyoming, when 2.27 inches of rain fell within a 48-hour period from June 12 to 13. In the more intense convective cells, precipitation amounts between 2 and 3 inches generally occurred (based upon precipitation estimates derived from radar), but none of these events were captured by a rain gauge. Given a 2.3-inch rainfall over a time period of 48 hours near Glendo Dam, and a 2.5-inch rainfall over a 4-day period near Jeffrey City, Wyoming, the return period for the precipitation is estimated to be 2 to10 years [13].

This event occurred during the normal storm seasonality timeframe, from June through August, when approximately 65 percent of all extreme storms occur in the Rocky Mountain region [17]. This storm was unusual in that it lasted over 1 week, while nearly 95 percent of all extreme storms in the Rocky Mountain region last less than 1 week. See figures 14 (a) and (b).

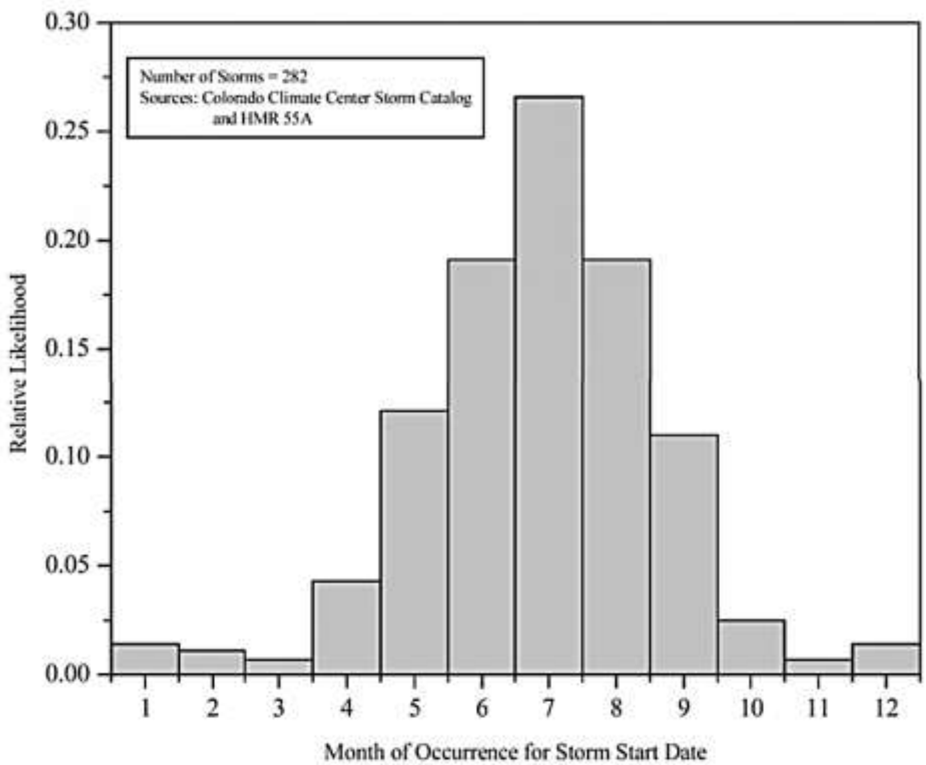

Figure 14. (a) Approximate month of occurrence of 282 extreme storms in the Rocky Mountain region. Storm seasonality is clearly high from May through September.

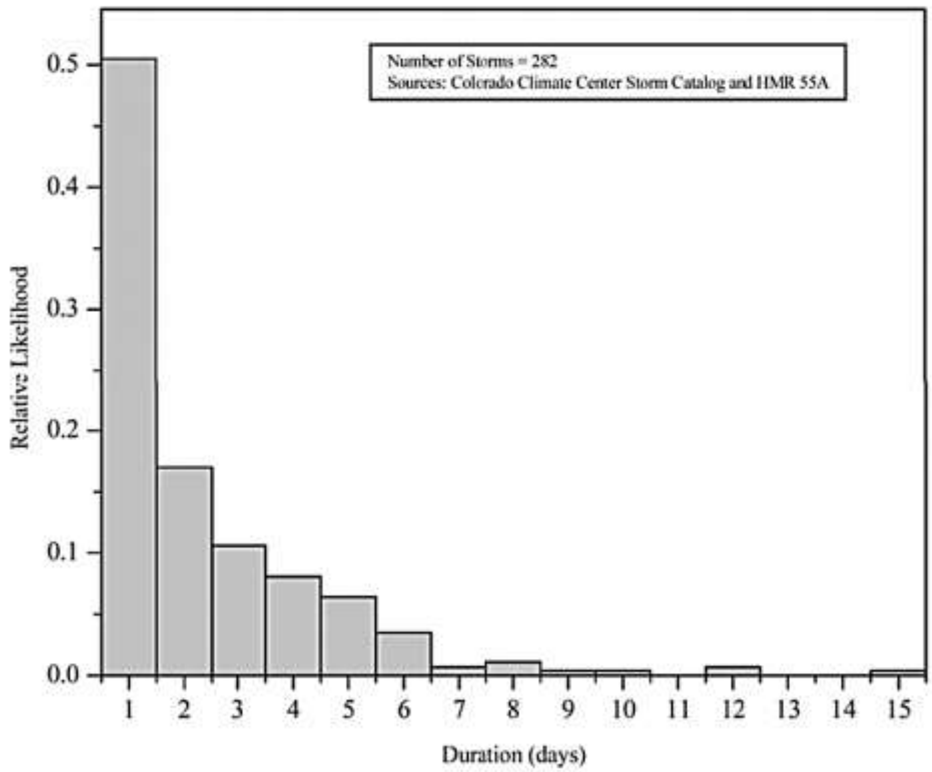

Figure 14. (b) Approximate durations of 282 extreme storms in the Rocky Mountain region. The majority of storms last less than 3 days in duration.

2. Comparison of 2010 and 2011 Snowpack and Flooding

While the 2010 winter snowpack was slightly above average, the 2011 winter produced an abundance of snowfall. In many cases, it was one of the snowiest winter seasons on record across central Wyoming and northern Colorado. The SWE (the amount of water present if all of the snow melted) for 2011 was estimated to be a 25- to 50-year event. By comparison, the 2010 SWE was estimated to have a return period of 5 to 6 years [14].

The 2011 snowpack was much larger than the 2010 snowpack; however, flooding occurred in 2010, not 2011. In 2010, a series of upper level lows caused rain to fall on snow for much of the first half of June. However, in 2011, most of the major storm systems produced heavy rain, either east or south of the North Platte River Basin upstream of Glendo Dam, despite the Western United States being in an active storm pattern. For example, Wheatland, Wyoming, located approximately 25 miles south of Glendo Dam, received 3.04 inches of rain in June 2010 and 3.03 inches of rain in 2011 [15]. The main difference in the flooding between 2010 and 2011 is that there was no rain-on-snow component, even though there was nearly the same amount of rain recorded at Glendo Dam. Because the rain in 2010 had a larger aerial extent in the downstream basin than in 2011, downstream tributary flows and reduced irrigation demand required reduced releases, and this also contributed to higher reservoir levels at Glendo

Dam. Without any major rain-on-snow events in 2011, the snow melted at a normal rate without the threat of flooding, as was seen in 2010.

C. Hydrologic Loading, Operations, and Response

In 2010, the snowpack in the basin above Glendo Dam was below average from December through early April. The forecasts were for a below-average inflow in April 2010. In June, however, the precipitation was widespread in the basin and extended downstream into the irrigation delivery area. Because of the June rains, there was very little irrigation demand. The timing of high flows coming down the North Platte also coincided with reservoirs on the Laramie River filling and spilling. The high flows from the Laramie River entered the North Platte River downstream of Glendo Dam. In an attempt to mitigate downstream flooding resulting from Glendo Dam releases, combined with Laramie River releases and the reduced irrigation demand, the U.S. Army Corps of Engineers directed that releases from Glendo Dam be reduced to allow the Laramie River flood peak to pass.

The combination of June rain storms in the basin above Glendo Dam with the above average runoff from the snowmelt and the reduced releases contributed to the high reservoir levels at Glendo Dam during construction in 2010. The reservoir reached elevation 4648.83 feet, which was the highest elevation since the dike seepage berms were constructed in 1989 (see figure 15). These high reservoir levels led to concerns on June 20, 2010, at the dikes where cloudy seepage was observed and instrumentation readings were abnormal and out of expected limits. Several wet spots were observed during this incident. As the reservoir reached elevation 4648.83 on June 28, the most notable location of seepage was at the left groin of Dike 3, where seepage was observed bubbling and exiting at a point 4 feet lower than the reservoir elevation. The exit gradient produced sufficient velocity flow to cause incipient sand size particle movement. Water could be heard flowing under the cobbles on top of the seepage berm. The water level at inspection well IW-H was about 1 foot below the top of the cobbles, and water was observed entering the manhole through the joints in the riser pipes. Many other seepage locations were observed at the dikes, and their locations were surveyed by Reclamation. As a result, a risk team was formed and the hydrologic internal erosion failure modes at the dikes were evaluated. The understanding of the failure mode changed sufficiently to result in the removal of dike raise modifications from the contract. A corrective action study was initiated to evaluate alternatives to mitigate risks associated with internal erosion failure modes at the dikes.

In 2011, the largest snowpack on record was in the basin upstream of Glendo Dam. The 1.97 million acre-feet of runoff was about 2.8 times the average of 700,000 acre-feet and significantly larger than the 1.5 million acre-feet that was the second highest year on record. Due to this large record snowpack, the reservoir was predicted to reach between elevations 4649 feet and 4660 feet in early May 2011, depending on the temperature conditions and the potential spring rains. The Wyoming Area Office began releasing water in March, about 2 months earlier than in a normal water year, to create reservoir storage capacity in the system reservoirs with the key concern that a repeat of the 2010 water year with June rains and a fast snowmelt could produce high flows in the North Platte River.

Figure 15. Aerial view of the Glendo dikes during the 2010 flood.

Based on the projections for high reservoir levels due to the large snowpack, Reclamation implemented a temporary emergency modification to the 1989 seepage berms downstream of two of the dikes. The original seepage berm at Dike 2 had 10 feet of berm material above the toe, and the design team judged this would be adequate. Therefore, design and construction efforts were focused on Dikes 1 and 3. The objective of the design was to filter and collect seepage. The emergency modification to the dikes was completed using the contractor who was already working onsite on the auxiliary spillway and dam raise.

The scope of the modification was limited due to the rate at which the reservoir was rising. Excavation was kept to a minimum, and the construction time period was limited to a few weeks to allow the construction to be completed in advance of the rising reservoir. Given these restrictions, the design intent of the emergency modification was to reduce risk of failure up to the predicted reservoir water surface elevation of 4653 feet. The emergency modification consisted of the construction of an emergency berm, which required removing approximately 2 to 3 feet of cobbles from the top and downstream slope of Dike 1 and Dike 3 in order to expose the sand and gravel zone of the 1989 seepage berms. The topsoil on a portion of the downstream slope of Dike 1 and Dike 3 was stripped. A small prism or trench was excavated an additional 4 to 5 feet into the sand and gravel of the 1989 seepage berms (see figure 16). The excavated surface was lined with a sand filter zone that was 1.5 feet thick on horizontal surfaces and 3.5 feet thick on sloped surfaces. The sand filter zone extended up the downstream slope of Dike 1 and Dike 3 to a height of about 6 to 9 feet. A gravel drain zone surrounded the perforated toe drain pipe downstream of the sand filter zone. The gravel drain extended to the same elevation up the downstream slope of Dike 1 and Dike 3 as the filter sand. The miscellaneous bermfill extended up the downstream slope about 2 feet above the sand filter and gravel drain zones. The emergency berm had a width of about 45 feet. Toe drain outfalls extended downstream to the existing weir boxes. In addition to the seepage berm modification, additional sand filter and gravel drain material was stockpiled at the dikes and at the dam in case an emergency arose at the dikes or dam.

Figure 16. View of the seepage berm modifications at the Glendo dikes in spring 2011.

With the contractor onsite working 7 days a week, the emergency repair work was completed in 5 weeks and was in place by the end of June 9 (see figure 16). Fortunately, a cool spring with no rain-on-snow events allowed the snowpack to melt slowly through the summer months. In the summer of 2011, the reservoir eventually rose to an elevation much lower than had been projected in May. The peak daily inflow was 9,100 ft^3/s into Glendo Reservoir in late May. The seepage berm modifications remained until they were removed as part of permanent dike modifications in the summer of 2014.

V. Summary and Conclusions

Three case histories for dam modification have been presented which highlight the need for determining the appropriate level of flood protection during construction, especially where the dam under modification is a significant- or high-hazard facility and consequences may extend beyond the value of the construction work and into the downstream population at risk and property. These case histories also highlight the usefulness of understanding potential meteorological conditions that could occur and the need for a plan that can be put in place to reduce risks associated with construction floods.

In the case of the Glendo Dam modification, risks during construction were evaluated, and an interim construction EAP was prepared in advance of construction to supplement the existing EAP. In 2011, when it was realized that the large snowpack could melt rapidly in combination with early summer rains, as had occurred the previous year, the National Weather Service (NWS) was used to provide real-time information on snowpack, SWE data, runoff data, and real-time future projections on atmospheric conditions. The reservoir operation plans were adapted to begin releasing water more than 6 weeks earlier than normal to create space in the upstream reservoirs for the eventual runoff from the large snowpack in the upper basin and the potential for rain-on-snow events. The NWS data supported the decision to perform a modification on the dikes during construction to reduce the risk associated with rising reservoir levels. With the longer lead time provided by the early March snowpack data, the decisions made in 2011 by the Wyoming Area Office to adapt the reservoir system operation was key to preventing downstream flooding conditions and high reservoir levels at Glendo Dam. The Glendo Dam case history also highlights a situation where a contractor already onsite can assist in implementing emergency modifications.

The flood event during the Theodore Roosevelt Dam modification highlights the potential for rare hydrologic conditions to occur and the importance of an appropriately designed cofferdam. In this case, multiple back-to-back atmospheric river events brought rain to an area of saturated soils. The capability for the cofferdam to withstand overtopping without failing prevented a much larger and sudden discharge, as well as the benefit of attenuating the flood flow releases because of the large surface area of the reservoir. Theodore Roosevelt Dam may have been the first large modification where risk based approach was used to study the potential risks during construction.

For the Jackson Dam modification, the key was the adequate design of a cofferdam/diversion system. Understanding the consequences of failure of a cofferdam highlights the need to use risk-based analysis approaches when defining the risks during construction and in selecting the appropriate level of flood protection for the cofferdam/diversion system. Current Reclamation guidance is available on this issue [1, 2].

In all of these construction case histories, antecedent conditions and forecasts projections were indicating the potential for a major flood event. This raised the level of awareness and concern resulting in increased monitoring of inflow and NWS data, and operations were modified where possible.

- Antecedent conditions included saturated soils in the basin and the buildup of snowpack.

- Long-term forecasts were predicting large snowpack conditions with the potential for rain-on-snow conditions or atmospheric conditions that were likely to produce flooding.

- The floods were generally caused by a series of back-to-back precipitation events resulting from an atmospheric river condition that continued to funnel moisture into a critical location within the upstream drainage basin.

A key decision in each of the case histories was who would have the responsibility for the design of the diversion plan and cofferdam. In cases where no downstream life loss or significant economic consequences exist, the contractor can be responsible for the diversion during construction. However, in cases where it has been determined that life loss and significant economic damages could occur, Reclamation's *Design of Gravity Dams* [18] states:

> "Designer's Responsibilities. – For difficult and/or hazardous diversion situations, it may prove economical for the owner to assume the responsibility for the diversion plan. One reason for this is that contractors tend to increase bid prices for diversion of the stream if the specifications contain many restrictions and there is a large amount of risk involved. A definite scheme of cofferdams and tunnels might be specified where the loss of life and property damage might be heavy if a cofferdam built at the contractor's risk were to fail."

The case histories presented in this report were all appropriately deemed to have significant risks during construction and were Reclamation designed. Based on these case histories, it is essential that the following points be considered when determining the design level of flood protection and other requirements during construction, as well as the process for monitoring and responding to emergencies:

1. If necessary, perform a risk-based analysis to determine the level of flood protection needed to protect the population at risk downstream of the dam, as well as to protect the work associated with construction. The risk-based analysis may be useful in evaluating and selecting the construction design flood, the type of cofferdam/diversion plan, reservoir flood storage, construction staging, work restriction requirements, and actions that may be needed in the event of an emergency. The evaluation of alternatives and operations during construction (including storage space and diversion release requirements) and the flood routing studies can be a significant design effort, and their importance should not be reduced or diminished due to design schedules and budgets. Consultant review board and management concurrence on the proposed level of flood protection is an important part of the design and decision process.

2. The specifications should provide the contractor with information necessary to design the features necessary to provide adequate diversion and care of the stream flows during construction. The specifications should include hydrographs of historical stream flows, work restrictions, the design flood level of flood protection, and any construction staging requirements related to times of the year when there is a reduced risk of flooding.

3. An interim or supplemental construction EAP should be developed that identifies potential failure modes during construction, as well as a plan to monitor and take appropriate actions if a large flood event develops. The supplemental construction EAP should define the roles, responsibilities, and line of communication. If an operations office that can monitor snowpack and runoff is not part of the EAP team, the tasks of monitoring rainfall and runoff, interpreting information produced by the NWS, and advising decisionmakers on the potential of changing conditions can be given to a meteorologist or hydrologist assigned to the construction support team. The NWS can be used to provide real-time information on precipitation, snowpack, and runoff data. In addition, the NWS provides forecasts and outlooks on atmospheric conditions. This information can be used by decisionmakers to reduce the risk associated with rising reservoir levels and to adapt the reservoir operation plans to address changing conditions.

VI. References

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Appendix A

Auburn Cofferdam Case History

Appendix A

Auburn Cofferdam Case History

Background and Description

Auburn Dam was planned as a double curvature, concrete arch dam. The dam was to be constructed upstream of Folsom Dam on the American River. The construction contract for the foundation excavation was awarded, and the diversion tunnel and cofferdam were in place. The foundation excavation for the dam and left spillway were nearing completion when the construction was stopped in 1979, due to safety and environmental concerns. The Auburn cofferdam case history is different from other construction flood case histories presented in this report in that the construction was suspended and the cofferdam was in place for about 7 years until it was overtopped and breached in 1986.

The diversion plan consisted of a diversion tunnel and cofferdam designed for a 25-year flood event. The cofferdam was a zoned earthfill and rockfill embankment and was constructed to elevation 715. The cofferdam had a crest width of 30 feet and was about 1,200 feet long. A 33-foot-diameter, concrete-lined, horseshoe-shaped diversion tunnel was constructed in the left abutment. The diversion tunnel had a discharge capacity of 74,000 ft^3/s.

Figure A1. Aerial photo of downstream face of cofferdam.

The cofferdam was later raised to elevation 719 feet to provide protection for a 30-year flood event when it was determined that the cofferdam would be in service longer than was originally anticipated. The cofferdam was about 265 feet high and had about 120,000 acre-feet in storage capacity. In addition, an emergency spillway was constructed on the right abutment. A fuse dike was constructed as part of the emergency spillway with a crest elevation of 715 feet. The fuse dike was 220 feet long and 10 feet high, and its elevation was about 2 to 4 feet below the crest of the cofferdam. The channel downstream of the fuse dike was about 400 feet long on a buttress fill.

Meteorological Setup for the Flooding Event: February 12-20, 1986

A high precipitation event occurred over northern and central California from February 12-20, 1986. This event caused flooding on the American River, which then caused the failure of the cofferdam near Auburn, California.

The February 1986 flooding event is different than most major West Coast storms in that it occurred during the La Nina phase of the El Nino Southern Oscillation. Typically, in the La Nina phase, the jet stream remains farther north over the Pacific Northwest. However, during this event, the jet stream dipped south, allowing an Atmospheric River event to develop and inundate much of California. Previous to the high precipitation event, the central Sierra Nevada Mountains were slightly below normal for snowfall.

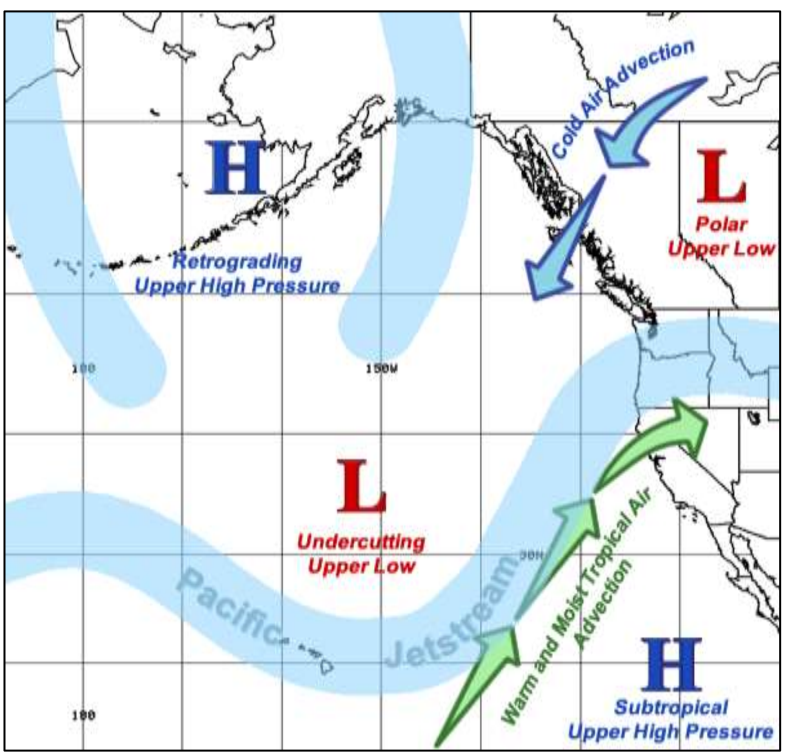

Figure A2. General meteorological setup for February 12-20, 1986 storm. (Source: National Weather Service, 2012)

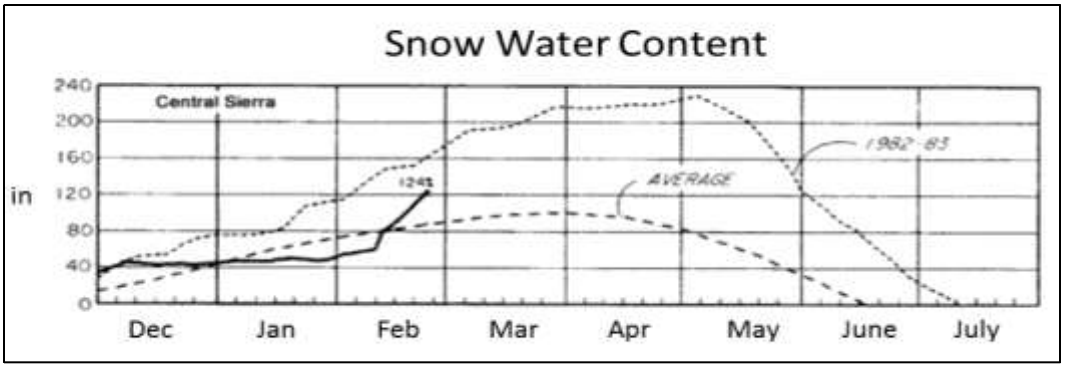

Figure A3. Snow water content for central Sierra Nevada Mountains showing slightly below normal snowfall prior to the February 12-20, 1986 storm. (Source: National Weather Service, 2012)

A series of three storms began to impact California from February 12-20, 1986. The first storm arrived on February 12 and lasted until February 13. It generally brought 1 to 4 inches of rain to the Middle and North Fork of the American River basins, even though it was the weakest of the three systems. The second system arrived on February 14 and lasted until February 15. This system generally brought 2 to 6 inches of rain to the Middle and North Fork of the American River basins. During the first two storm systems, the snow level remained between 8,000 and 9,000 feet, creating the potential for a rain-on-snow event to occur. The last storm system transpired from February 16-20 and was, by far, the strongest of the three storms. It brought large amounts of mild Pacific air, which raised the snow level to 10,000 feet. Because the highest elevation in this basin is only around 9,000 feet, a rain-on-snow event occurred for the entire basin. In addition to rain-on-snow, the storm produced anywhere from 8 to 25 inches of rain over the basins. The higher amounts of rain were located at higher elevations, where the winds were nearly perpendicular to the Sierra Nevada. This orographic forcing, coupled with efficient rainfall processes caused by the warm, moist Pacific air, led to substantial amounts of runoff. The return period for the 15 to 35 inches of rain that fell over the 9-day timeframe was estimated to be approximately a 50-year rainfall event. The lower rainfall amounts (at lower elevations) corresponded to a 25-year event, while the higher rainfall amounts (at higher elevations where snow would usually occur) corresponded to a 100-year event.

This event took place during the normal storm seasonality timeframe from December through March, during which approximately 68 percent of the mean precipitation falls at Auburn, California.

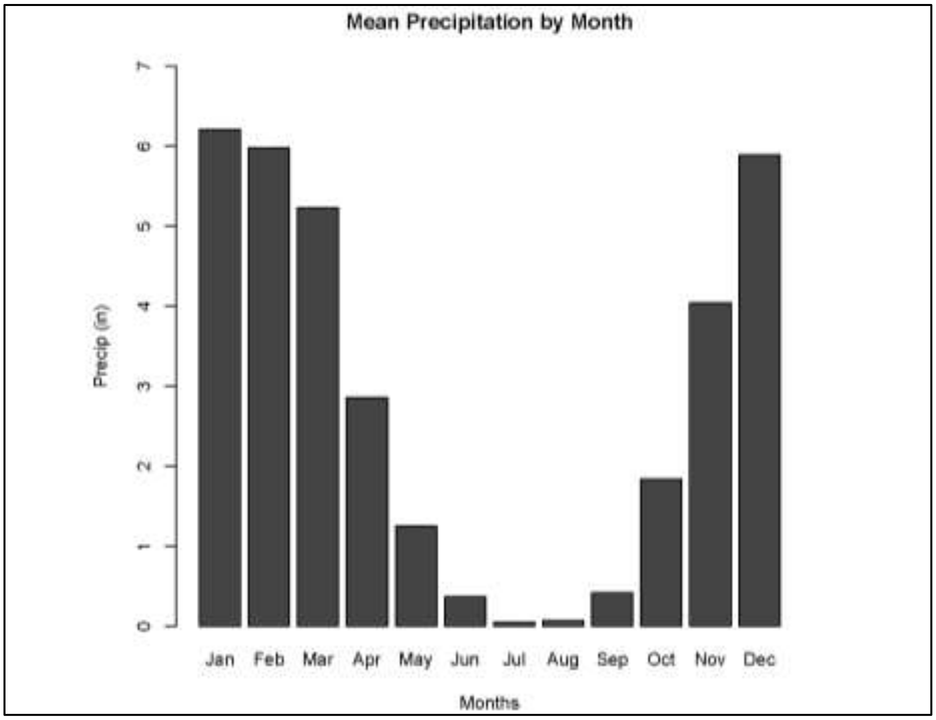

Figure A4. Mean precipitation by month for Auburn, California. Note that December through March is the general storm seasonality. (Source: National Climatic Data Center, 2014)

Figure A5. Aerial photo of the downstream face of cofferdam and foundation excavation during large flood event. The fuse dike and the long buttress fill channel downstream of the fuse dike can be seen to the left of the cofferdam.

Hydrologic Loading, Operations, and Response

An emergency preparedness plan was prepared in advance [A2]. The emergency preparedness plan was put into effect when inflows exceeded 30,000 ft^3/s, and a team was on staff to monitor site conditions and weather reports every 3 hours. This plan included a communications directory and instructions to notify local authorities and take appropriate measure if failure of the cofferdam was considered imminent. The emergency preparedness plan indicated that if the cofferdam failed, the released water could be safely retained without overtopping Folsom Dam or without requiring Folsom releases to exceed the safe channel capacity through Sacramento, California. Plans were also in place to excavate a small pilot channel in the fuse dike if the water surface reached elevation 713. To monitor the rate of reservoir rise, markers were placed every 5 feet in elevation rise on the upstream slope of the cofferdam. The emergency dike was intentionally placed on the right side of the cofferdam, where there was a substantial buttress fill about 400 feet in length.

Coordinating the operations of 14 reservoirs required multiple agencies, including the United States Army Corps of Engineers, Bureau of Reclamation, California Department of Water Resources Flood Control Center, the Sacramento County Office of Emergency Services, and others. The operations were guided by constant monitoring of the reservoir storage, inflows, and releases, as well as by the information provided by the National Weather Service River Forecast Center.

Figure A6. Photo of upstream slope of cofferdam showing markers placed every 5 feet in elevation.

A large 3-day storm began on February 16, 1986. Project personnel started the flood watch at 3:00 a.m. on February 16. By 11:00 a.m. on February 17, the reservoir created by Auburn cofferdam had filled to about one-third of its capacity. At the rate the reservoir was filling, it would overtop the emergency spillway in 14 hours. The peak inflow of 135,400 cubic feet per second was recorded at midnight on February 17. On February 18 at 5:30 a.m., the water surface had risen to elevation 715, which was above the crest elevation of the fuse dike in the emergency spillway. The cofferdam overtopped from 5:30 a.m. to 12:00 noon without erosion of the core of the cofferdam. The water surface eventually rose to elevation 717.6. By 3:00 p.m. on February 18, the cofferdam had breached. The emergency spillway in the right abutment area with the fuse dike and buttress fill was effective in delaying the breach. Because the failure of the cofferdam was anticipated, storage space including flood surcharge storage space was still available in Folsom Reservoir. As a result of this flood and the breach of the cofferdam, the reservoir water surface at Folsom Dam rose about 1.5 feet into the surcharge storage space with about 8 feet remaining. The peak inflow into Folsom resulting from storm runoff, plus the cofferdam breach, was estimated to be about 553,000 ft^3/s [A3].

Figure A7. Aerial photograph of the cofferdam breach during the 1986 flood.

The levees downstream of Folsom Dam were not overtopped, and flooding was avoided. The flood runoff was estimated to have a 105-year return period and was the flood of record at the time [A4]. Forecasts from the National Weather Service River Forecast Center were important in guiding the operations of the various reservoirs during the flood.

Appendix A References

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