# FONTENELLE DAM, RIRIE DAM, AND TETON DAM – AN EXAMINATION OF THE INFLUENCE OF ORGANIZATIONAL CULTURE ON DECISION-MAKING

*Fontenelle Dam, near Fontenelle, Wyoming, built by the Bureau of Reclamation between 1960 and 1964, had a serious incident in 1965 that nearly caused a breach of the structure that could have released more than 300,000 acre-feet of water. Teton Dam, near Rexburg, Idaho, built by the Bureau of Reclamation between 1972 and 1976, failed in 1976 releasing more than 250,000 acre-feet of water and causing 11 fatalities. Ririe Dam, also near Rexburg Idaho, was built in geologic conditions similar to Teton by the Walla Walla District of the Corps of Engineers between 1970 and 1976 – was subsequently transferred to Reclamation – and has performed well over its life. The dams are all located within 150 miles of each other. The story of why Teton Dam failed is more complex than the technical details surrounding its construction and destruction. The Teton disaster is a story of engineering judgment, communication, decision-making, economics, organization, culture, and individual and organizational hubris. To tell this story, one needs to examine the organizational cultures influencing the design and construction of these structures. The focus of this story will be primarily to highlight the differences and similarities between the design, construction, and organizational decision-making at Ririe Dam and Teton Dam and briefly examine the influence that the incident at Fontenelle had on both structures.*

## Introduction

In the 1970's, dams were considered robust, resilient, and for most situations, people felt safe living near them. The Bureau of Reclamation (Reclamation) and the U.S. Army Corps of Engineers (Corps) – as well as countless other organizations – had built hundreds of large dams across the country and despite several incidents; dams had proven themselves to be able to hold up well in almost every circumstance. The pioneering engineers who developed dam design philosophies in the 1940's through the 1970's learned primarily through experience. When possible, dams were modeled in laboratories, but for those features that didn't scale well in laboratories, they were designed and their performance monitored to see how successfully they behaved. In Reclamation in the 1940's and 1950's, there were even instances where foundation grouting was not part of original construction plans as designers preferred to observe the performance under first filling before attempting to improve the foundation. Designers, many of whom lived through the great depression and served in World War II, made every effort to be more efficient and minimize the costs erecting the substantial water delivery infrastructure that exists today. As far as the two federal agencies were concerned, their efforts were successful. That success led to a level of confidence at Reclamation that the methods they employed to design embankments were entirely appropriate and that their efforts to constantly make designs more cost-effective were rational and also appropriate.

Two events changed the face of embankment dam design in the 1970's; the failure of the upstream slope of Lower San Fernando Dam in 1971 and the failure of Teton Dam in 1976. Particularly with the Teton Dam failure, the face of dam design and dam safety was dramatically altered across the entire country. At Reclamation in particular, the culture underwent a shift that eventually led to the creation of the Dam Safety Office, the use of potential failure modes to augment traditional analysis, and the development of risk management.

The information in this paper was obtained during many interviews with former Reclamation and Corps staff who worked on the design or construction of Teton and Ririe Dams and examining the vast records for those facilities. It is only because of their straightforward and honest contributions and their desire to share the lessons they learned that some of the information in this paper was made possible. It was not possible to interview anyone associated with the design and construction of Fontenelle Dam, however the records of the project are extensive and detailed. Much was learned from the construction photographs, design documents, construction reports, and travel reports.

The failure of Teton Dam was investigated by more than 5 separate groups. Each of these focused primarily on the engineering causes of the failure. When one examines significant engineering disasters such as the Flixborough chemical plant disaster, the Challenger disaster, the Hurricane Katrina disaster, and the Hyatt Regency walkway collapse, it becomes apparent that there can be substantial organizational and human components that exacerbate the engineering problems. This paper will summarize the findings from the Independent Review Panel (IRP) and the Interior Review Group (IRG) and supplement their technical conclusions with some additional lessons to be learned from the construction of Fontenelle, Teton, and Ririe Dams.

## Reclamation and Corps Design Atmosphere in the 1960's and 1970's

At Reclamation during the time when Teton Dam was being designed, many other dams were either envisioned or being constructed. Across the country, it was an era where dams were literally being built as fast as possible. In the Western U.S., Reclamation was rapidly advancing its mission to 'reclaim the arid West' by constructing a vast network of dams and canals. The projects constructed or envisioned were not the result of large or wealthy industrial forces; rather they were a system of locally-supported facilities that helped individual farmers fuel the population and agricultural growth in the West. Although municipal water supplies and flood protection were important to the overall picture, agriculture was the driving political force behind the authorization of most of the dams and canals in Reclamation's inventory. The population was a fraction of what it is today, but the growth was anticipated – at least regionally. The benefits used to authorize projects were projected into the future to justify the appropriations. Regardless of the location of the project, Reclamation was very cost-conscious during both the design and construction of its facilities.

The Corps was similarly constructing a large flood control infrastructure of dams and levees across the entire country attempting to accomplish its mission to reduce the impacts of flooding along watercourses nation-wide. Both agencies had significant political support for and environmental resistance against their activities.

### *Organization*

The failure of Teton Dam and the successful operations to-date at Ririe Dam were influenced by the way in which the organizations were structured and the philosophies they were founded upon. Figure 1 illustrates how Reclamation and the Corps were organized in 1974 when both dams were under construction. For each organization, the responsibilities of design, technical review, and technical approval are indicated. These three functions are critical parts for an organization to ensure technical excellence and consistency, and that decisions are made at appropriate levels. An important distinction between the two organizations is that the authority for design and construction for the Corps was in a single line of the organization, whereas for Reclamation, design was the responsibility of the Assistant Commissioner but construction was the responsibility of the Regional Director.

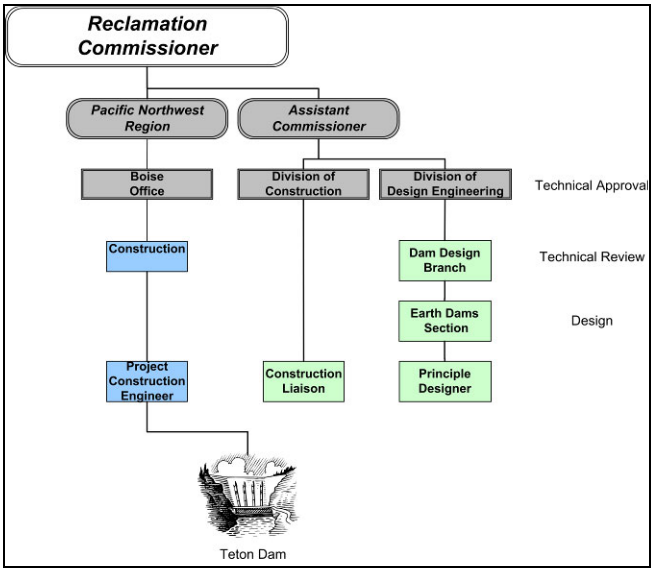


**Figure 1A.** Reclamation's organization, 1974

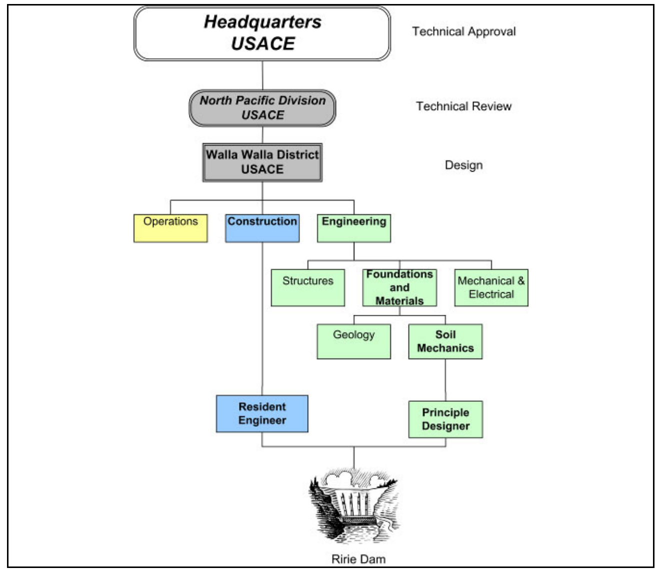


**Figure 1B**. Corps of Engineers organization, 1974

Some things we take for granted today did not exist in the 1960's or 1970's. Communications were slow, consisting primarily of telephones and, in later years, faxes. It's very difficult to describe field conditions over the phone. Having the Denver office so far from construction sites was more often problematic then than it would be today. The travel infrastructure did not exist; some structures could only be reached by train and car. Letters were often used to communicate and solve problems. At Reclamation, travel reports were required to be written by the designers after visiting the site. These reports were considered essential before any decision was finalized and unofficial agreements were not enacted until the decisions were approved by the management in the Division of Design in Denver. However, the construction forces had significant influence over the final decisions. Compounding this was the fact that very few designers ever visited the ongoing construction and thus their ability to deal with real-time problems was often very limited.

***Reclamation Design and Construction Philosophy***

At Reclamation, the 1960's and 1970's were the era of 'standard designs'. Many of the structures in Reclamation have remarkably similar cross-sections. They generally had a wide low-plasticity clay core of alluvial origins and a coarse miscellaneous shell, but many cores were silts or silty-sands as well. Although they were thought of as zoned embankments, they could just as easily have been classified as homogeneous. The designers used the closest suitable borrow and intended that the embankment be graded progressively coarser downstream. There was usually a cutoff trench in the upstream portion of the core that was tied the foundation if it was shallow enough to excavate to. A three-line grout curtain was standard.

All dam design was done at the Engineering and Research Center in Denver, Colorado. Designers worked in small groups designing dam after dam, each of which generally took several years. Geologists explored dam sites, design engineers designed dams, and construction engineers built dams. Geologists had minimal input to designs or construction. Design engineers designed structures; then handed the specifications to the construction offices to build structures. Designers rarely, if ever, visited their sites during exploration or construction. There was some friction between field engineering staffs and dam designers. Construction forces were expected to solve issues that arose during construction with minimal input from Denver. When designers did visit sites, they generally did not attempt to make significant changes. The designers felt that the field engineers did not want them on site and the field engineers felt that the designers rarely added anything of value when they did visit. Perhaps paradoxically considering the failure of Teton, most of Reclamation's dams were successfully built in this way. In retrospect, perhaps it was fortune or perhaps it was robustness in the original designs and the skill of the field engineers that allowed so many structures to be successfully built in that era without considering seepage control measures.

At Reclamation, concrete technology and structural dam design advanced tremendously between 1940 and 1970; whereas embankment dam design details changed very little. It was thought that since embankment dams had performed so well, little improvement could be made unless cost savings could be realized. The construction of a dam was not seen as a chance to educate, learn, share construction problems, or adapt new design details but was instead seen as new machine to be assembled. Designs were reviewed and approved through the chief of the engineering division, but the true responsibility was shouldered by the designers and the branch chiefs. Consultants were rarely used.

***Corps Design and Construction Philosophy***

At the Corps, each District developed its own design philosophy and construction details. Each district had a good understanding of their regional geologic setting. Designs were developed by each district, reviewed and approved at the division level, and reviewed and approved at the headquarters level. The headquarters and division offices had experienced technical expertise. Consultants were regularly used. Similar tensions existed between field staff and construction staff, but less so than at Reclamation. Embankment designs changed from the 1940's to the 1970's significantly as the importance of foundation treatment and filtering became apparent. Advancements weren't uniform, as each District functioned somewhat independently but at least for the North Pacific Division – responsible for the design and construction of Ririe Dam – some information sharing did occur because of the background and influence of the headquarters reviewers.

An examination of cross-sections of Corps dams reveals incredible diversity. It appears that designs were tailored to fit each site. In some structures, significant amounts of processed materials were used, and beginning in the early 1960's, filters were appearing in many designs. The Corps used more grout holes with thicker mixes on average than did Reclamation. Much thought was put into each design, perhaps as a result of the reviews that were necessary for the progressive approvals.

# Fontenelle Dam Design and Construction

***General***

Fontenelle Dam is located on the Green River 24 miles southeast of La Barge, Wyoming. The dam was built in 1964 to provide water for irrigation, the Seedskadee Wildlife Refuge, future municipal and industrial needs, and for generating power. The total capacity of the reservoir is 345,360 acre-feet at normal water surface elevation 6,505. The dam is a zoned earthfill structure with a structural height of 139 feet, a crest width of 30 feet, and a crest length of 5,450 feet at elevation 6519.0. The central core of impervious material (Zone 1) consists of a mixture of clay, silt, sand, and gravel compacted in 6-inch-thick layers. The core is flanked both upstream and downstream with free-draining sand, gravel, and cobbles (Zone 2) compacted in 8-inch-thick layers. Incorporated within the downstream Zone 2 portion of the embankment is a zone of miscellaneous material (Zone 3) obtained from the excavation. The upstream face of the dam is sloped at 3H:1V, and is protected by a 5-foot-thick blanket of riprap from the dam crest to a 15-foot-wide berm at elevation 6480. The downstream face of the dam is sloped at 2H:1V and has no slope protection. A concrete diaphragm wall was constructed in the late 1980's from the crest of the dam into the foundation to address potential for the dam to fail from internal erosion in the foundation or the lower portion of the embankment.

***Materials***

The materials for the Fontenelle embankment were borrowed from large alluvial deposits in the river valley. The shells were borrowed from some of the coarser deposits from the same borrow areas. The core was borrowed from the dam excavation and three upstream borrow areas. The core has an average Plasticity Index of 13. Based on the average gradation, the average plasticity index and the average liquid limit, the zone 1 material can be classified as a 'sandy silty clay' (ML-CL).

***Geology***

Fontenelle's foundation is primarily composed of sandstone and shale covered by alluvial deposits. Three main discontinuity sets have been measured and described at the site: 1) bedding plane joints, 2) vertical to near vertical tectonic joints, and 3) near-vertical relief jointing predominantly within the massive sandstone units. The bedding plane joints are evident within the platy sandstone and fissile shale units. The near-vertical relief jointing occurs predominantly within the massive sandstone units and within an area bordering the steep abutments. Because of its uniformly massive characteristics, the sandstones, particularly in the right abutment respond to stress by breaking along fractures which generally extend the full thickness of the unit and continue laterally for a considerable distance. These stress relief joints form in the most massive rock due to removal of lateral support. They are deep open joints that roughly parallel the abutment and extend at least to the bottom of the sandstone units. Five of these open joints were encountered in the spillway inlet excavation, and one was exposed in the spillway chute. They can be open to a width of up to one foot and are generally vertical and roughly parallel to the abutment contours. The layers are primarily horizontally bedded with significant and widespread vertical fractures. Figure 2A illustrates the extensive fracturing in the right abutment. Figure 2B illustrates the high permeability shown by the seepage exiting the same formation. Removal of the overburden over time also caused significant relief joints to open up in the sandstone and shale.

**Figure 2A.** A photo from right abutment site exploration in the 1950's showing the fractured nature of the foundation.

**Figure 2B.** A photo of the abutment downstream of the spillway. The seepage pattern illustrates the permeability and layering in the foundation.

***Design Considerations***

Designers knew about the fractures in the right abutment noting:

> *"The rock lies horizontally and is cut by numerous contraction joints and settlement cracks near the surface and in the steep right abutment…Special attention must be given to the contact between the embankment and the rock abutments. Rock spires, detached blocks, and loose, weathered rock should be removed to provide a solid and firm foundation. Overhangs and steep vertical or near vertical cliffs should be reduced to slopes of ½:1 or flatter under the Zone 1 portion of the embankment…Special compaction of the earthfill will be necessary against the exposed sound rock abutment."*[2]

Despite the intensity and width of the fractures in the right abutment, the planned designs relied entirely on 'special compaction' and the grout curtain to reduce the erosive forces at the contact between the cutoff trench and the grout cap. It appears that the designers did not fully appreciate the hydraulic gradient that would occur across the bottom of the cutoff and the relatively narrow grout curtain.

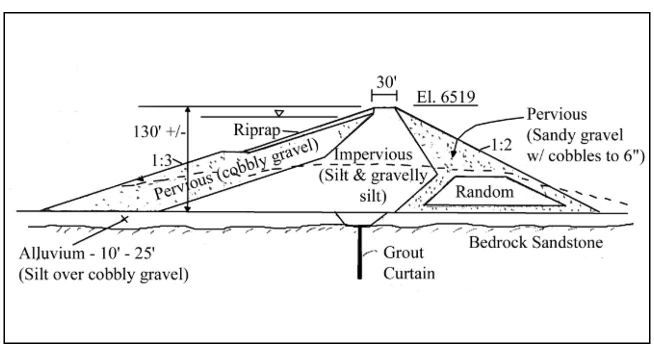


**Figure 3A.** Maximum cross-section, Fontenelle Dam.

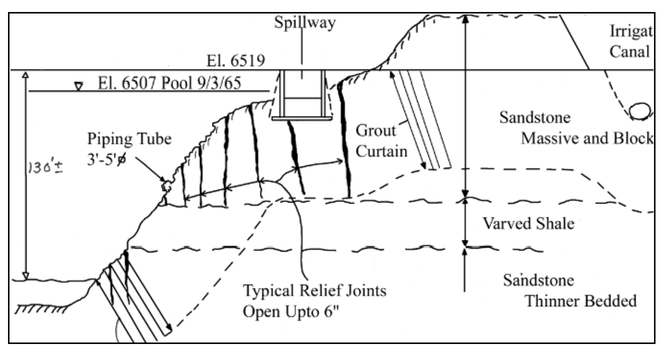


**Figure 3B.** Profile looking downstream of Fontenelle Dam, the right abutment at the spillway, and the location of the internal erosion incident.

The design documents list the Zone 2 drain material just downstream of the core as suitable for drainage; however there were no specified gradations or size limitations on the finer material. The Zone 2 was attributable to three borrow areas that were thought to contain more pervious material based on test pits in those locations. The ability for the Zone 2 to function as an actual drain is questionable, but even if it was able to do so, it did not extend below the original ground surface and would not protect the narrow cutoff trench. This same condition led to many of Teton Dam's design problems.

### *Foundation Preparation*

Foundation preparation was typical for a Reclamation structure at the time. The cutoff trench and the spillway on the right abutment were excavated to competent rock. Foundation grouting was completed through a 4-foot-wide concrete grout cap that ran the length of the centerline of the cutoff trench. Overhangs were removed where possible. No dental concrete was used and no slush grouting was used on the foundation surface. Special compaction was done at the contact between the embankment and the foundation rock. Special compaction consisted of hand-compacting a 2-foot-wide 6-inch-thick layer of soil adjacent to the foundation. Foundation grouting was minimally successful with regards to the embankment-foundation contact. Figure 4A, taken after grouting was complete, shows large fractures leading towards the grout cap, none of which have been sealed with grout. Examination of the fracture pattern and potential permeability evident in Figures 4A and 4B shows that anything less than an extensive foundation grouting and consolidation (blanket) grouting program would have had difficulty filling the extensive fractures in the foundation surrounding the cutoff trench. Overall, great care was taken to clean the foundation and carefully compact the embankment soil at the foundation contact, although some vertical features and overhangs are obvious in the photographs. The grout curtain by itself was an inadequate design feature and the foundation was not treated at its interface with the cutoff trench. Figure 4A is particularly revealing as it clearly shows wide and continuous open joints that are intersected only by the grout cap. Their openness means that hydraulic head losses were minimal upstream of the grout cap and the only defensive measure against core erosion is the 4-foot-wide cutoff trench-grout cap interface. This fatal flaw was to be repeated in the Teton Dam design six years later.

## Fontenelle Dam Incidents

Fontenelle Dam was so close to failure, it could easily have been the event that led to changes in dam safety across the country if not for some fortuitous circumstances. In many ways the story of Teton Dam is as much about lessons not learned from Fontenelle Dam as it is about Teton itself. Fontenelle Dam was completed in 1964 and began to fill with water that same year. Soon after filling started, the dam began to have performance problems. Significant seepage was observed in the left abutment, right abutment, and downstream of the dam (Figure 5).

Five significant incidents were observed at the structure, beginning in 1964. Although they were described as 'stability' problems, the photographs indicate internal erosion at the foundation contact with the cutoff trench may have been a more likely candidate for the initiating mechanism. The five incidents were:

- A partial failure of the backfill behind the river outlet stilling basin on May 27, 1964. Divers found 1,800 $yd^3$ of material in the stilling basin. Concentrated seepage was discovered exiting the shale.

- A 25 foot wide, 50 foot long, and 10 foot deep partial failure of the embankment on the left side of the spillway on May 7, 1965.
- A partial failure of the embankment on the left side of the spillway on July 8, 1965.
- A partial failure of the backfill behind the river outlet stilling basin on July 24, 1965.
- A very serious failure of a large portion of the embankment on the left side of the spillway on September 3, 1965.

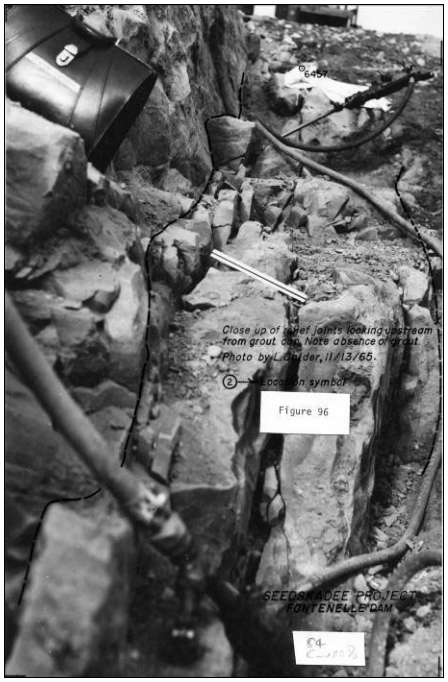


**Figure 4A.** A view from the grout cap in the right abutment looking upstream. Foundation treatment is complete, note absence of grout and openness of jointing leading towards the grout cap.

**Figure 4B.** A photo showing embankment construction along the right abutment. Note the steepness of the foundation and the narrowness of the grout cap under the ladder.

**Figure 5.** Fontenelle seepage downstream in 1965

The first four incidents have not been previously identified or published; as Reclamation felt at the time they were minor slope stability problems and were not necessarily associated with a failure mode that could affect the dam. Travel reports indicate that the design engineers in Denver visited the site and advised the field staff to replace the material that had been removed with free draining material to accommodate the seepage coming through the foundation. These smaller incidents were, in fact, a precursor to a very serious incident that occurred in September of 1965 (Figures 6 and 7) and an indication of the high permeability of the foundation that would initially result in an extensive foundation grouting program in the late 1960's and would eventually result in a significant modification to the structure in the 1980's.

A major disaster was averted in September 1965 (Figure 8) when Reclamation opened the river outlet works and the two canal outlet works at Fontenelle in an effort to lower the reservoir as rapidly as possible. So serious was the situation that equipment and formwork were washed out of the river outlet works, which was under construction to repair scour damage to the concrete stilling basin. Fortunately, the reservoir has a significant release capacity, and the pool dropped nearly 4 feet per day until the erosion waned and then stopped. The reservoir volume at the time of the incident was approximately 300,000 acre-feet, and dam failure would have released a tremendous amount of water. While the reservoir was down, the eroded portion of the embankment was replaced and the foundation was grouted extensively through 1967.

**Figure 6A.** Failure of the backfill on the left side of the river outlet stilling basin, May 30, 1964

**Figure 6B.** Seepage emerging from the shale formation along the left side of the spillway chute, May 21, 1965.

**Figure 7A.** Seepage emerging from the shale formation along the left side of the spillway chute, July 28, 1965.

**Figure 7B.** Seepage emerging from the shale formation along the left side of the spillway chute, July 28, 1965.

**Figure 8A.** Muddy seepage emerging from the embankment along the left side of the spillway chute, September 15, 1965.

**Figure 8B.** A wider photo of the large unstable area on the left side of spillway chute, September 17, 1965.

***Corps of Engineers Site Visit***

Kenneth S. Lane, the Chief of the Engineering Division for the Missouri River Division (MRD), U.S. Army Corps of Engineers (Corps), visited the site on November 19, 1965 with a group that included engineers from MRD, Fred Walker, the Head of the Earth Dams Branch for Reclamation, and Reclamation's Construction Engineer, Mr. Hatch. Mr. Lane's observations and conclusions were transmitted to the Chief Engineer for Reclamation, Barney Bellport, in a November 29, 1965 memorandum that contained valuable information relating to the failure mode that had progressed at Fontenelle. He made four important conclusions listed below:

1. *This [the failure] emphasizes the Corps policy that designers inspecting earthwork and foundation conditions at key times during construction is a must.*
2. *Fractured abutment rock deserves very conservative treatment, including consolidation grouting for near full width of the core plus a conservative grout curtain (deeper than 60-foot curtain installed initially at Fontenelle).*
3. *An irregular rock surface should be smoothed out and all overhangs removed in order to facilitate compaction of the embankment at the rock contact.*
4. *Locating a concrete spillway close to an abutment edge is not desirable. At Fontenelle it prevented smoothing the rock surface by pre-splitting as done at Milford, illustrated by enclosed sketch. Actually it seemed there may have been another spillway location in the opposite abutment where possibly even an unpaved channel would have been practical. This points up the desirability of studying possible layouts in order to choose the best from an overall standpoint.*

In addition to the points listed above, Mr. Lane identified the failure mode at Fontenelle – which could just as easily describe the situation at the Teton Dam site – noting "*essentially this is a case of piping through the earth embankment at its contact with the rock abutment. It is felt seepage entered through the open relief joints in the rock and doubtless eroded soil into these open joints. Most probably the seepage jumped the grout curtain (and its 3 to 4 ft. wide concrete grout slab) by passing through looser pockets in the embankment. As the piping tube enlarged, this would attract additional seepage, accelerating erosion to cause the very severe situation shown by the enclosed ENR photo."*

Mr. Lane also noted a particular characteristic of the right abutment of Fontenelle. Understanding this statement would have had a tremendously beneficial impact had it been considered during the design of Teton Dam.

> "*both Mr. [Fred] Walker and the writer had some suspicions on the possibility of cracking in the embankment as a result of differential settlement at this steep rock abutment – somewhat from Corps experience at Wister and East Branch dams where cracking occurred from differential settlement and was enlarged by piping. However, it was understood that men crawled into the piping tube at Fontenelle and found no evidence of cracking in the embankment. Furthermore, the gravel to silt mixture would be less prone to cracking due to its well graded nature. In the last analysis, whether or not embankment cracking also occurred is a rather academic question, since at best it was probably not more than an additional contributing factor to the main cause of piping at the earth-rock contact as discussed above."*

After Mr. Lane returned to MRD, he toured all Kansas City District (KCD) dams under design and construction with Jake Redlinger and Neil Parrett, who would eventually be the headquarters reviewer for Ririe Dam. That field review trip resulted in KCD adding a filter zone on the downstream side of cut-off trenches in all their structures under design or construction. These lessons could have also been valuable considerations for dam construction in Reclamation after 1965 had they been shared throughout the organization. Interviews with design engineers that worked for Reclamation in that era indicated that many of them had heard that Fontenelle had an incident, but none of them had specific information concerning either the nature of the incident or the potential lessons that could have been learned from it. Instead, the lessons Reclamation learned were documented in an ICOLD journal article by Mr. Bellport[1], and he made only a passing mention of the actual mode of failure at Fontenelle, instead writing in his conclusion "*it is apparent that the weak spot was in the abutment and not in the embankment.*" Bellport's assessment makes no mention of the interaction between the embankment and the foundation or the gradient that existed across the narrow foundation cutoff, highlighting a philosophy that would have a profound affect on the organization in the following years.

The ICOLD article would have been written by a lower level embankment designer and reviewed by the head of the Embankment Dams Branch, Mr. Walker. The Denver Construction Division and the regional construction office would also have reviewed the article. It was not accepted practice at the time to reveal flaws in either designs or construction. Few people ever got a realistic account of the incidents at Fontenelle. Most heard only a brief mention of it and it is clear that few, if any, really understood what this near failure meant. No design philosophies, design details, specifications, or construction practices were changed as a result of the incidents.

## Ririe Design and Construction

Ririe Dam is a 253-foot-high, 1,070-foot-long zoned embankment on Willow Creek, 25 miles northeast of Idaho Falls, ID. The crest elevation is 5128.0. It has a reservoir capacity of 90,500 acre-feet at the top of active conservation, elevation 5112.8. It has a gated spillway on the right abutment with a discharge capacity of 48,762 $ft^3$/sec at elevation 5119.0 and an outlet works with a discharge capacity of 4,250 $ft^3$/sec at elevation 5119.0. The spillway has never operated, as flows have not exceeded the outlet works capacity.

### *General*

Planning for Ririe Dam was begun in the 1950's by the Walla Walla District of the Corps of Engineers. Ririe Dam is primarily a flood control facility, although there are irrigation, municipal, fish and wildlife, and recreation uses. Figure 9A shows the outline of the dam from the planning document; Figure 9B shows the dam as it is today. The Walla Walla District was experienced building dams on volcanic foundations, as many dams in the Pacific Northwest are built on those types of foundations.


**Figure 9A.** Planning photo from 1965 looking downstream at the Ririe Dam site.

**Figure 9B.** Photo of Ririe Dam from 2006.

***Materials***

Seventeen types of materials were used to construct Ririe Dam. The core of the Ririe embankment is composed of low plasticity silts borrowed from alluvial deposits upstream. The sand filter was processed from alluvial deposits upstream and meets modern filter criteria for the core. The gravel drain was also processed from alluvial deposits upstream and meets modern filter criteria for the sand filter. The gravel zone downstream meets modern filter criteria for the sand filter. All of the materials are volcanic in origin.

***Geology***

The canyon walls at the site are composed primarily of volcanic flows, chiefly basalts, with interbedded thin layers of fluvial sediments and soil horizons. The uppermost geologic unit is a rhyolite bed that averages 30 feet in thickness. The rhyolite is moderately hard to soft, pink to gray in color, fine grained with needle-shaped crystals with a thin (5-foot) layer of volcanic ash at the bottom of the flow. Beneath this, five individual basalt flows are recognized. Some of the flows are separated by plastic clay interbeds that also contain stringers of silt and occasionally a little gravel. The interbeds reach as much as 35 feet in thickness, but are normally much thinner. The basalt flows consist of fine to medium grained rock that is hard, drak gray to gray in color, and nonvesicular to highly vesicular in texture.

The volcanic flows are highly variable in texture, structure, thickness, and often even continuity. This variability is caused by differing cooling conditions, preexisting erosional surfaces, lava composition, and many other factors. Many types of basalt are present at the site, including columnar, brecciated, vesicular, massive, and combinations of these. Many of the flows also have a several-foot-thick zone at their bases that are composed of contact breccias, a highly fractured basalt and soil mixture resulting from sudden cooling against the relatively cool, preexisting surface on which the lava came to rest.

The basalt layers contain both open and silt- and clay-filled joints and fractures resulting from gentle folding and arching. The open and filled joints range from minute fractures to one-foot openings. Most of the joints and fractures have been predominantly filled with silty clay materials. Figures 10A and 10B illustrate the nature of the foundation at Ririe.

**Figure 10A.** Ririe construction photograph showing foundation in fault zone.

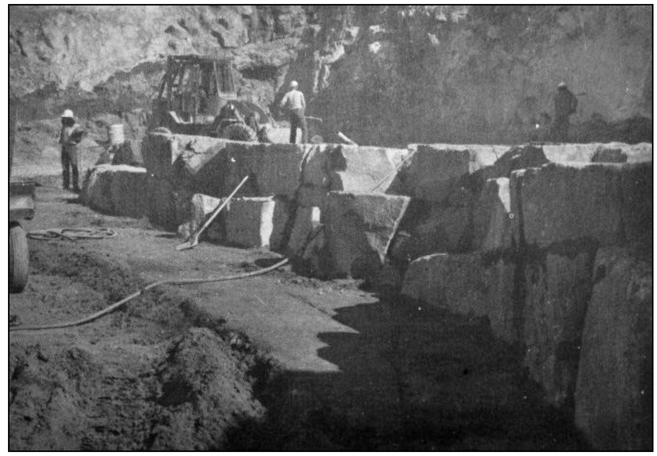

**Figure 10B.** Construction photo of Ririe right abutment.

***Design Considerations***

Before designs were envisioned, the Corps was concerned about the competence of the foundation of Ririe Dam. The first published design which was planned in the early 1960's and included in the design report in 1966 is shown in Figure 11. Interestingly, the design included a 10' wide chimney filter and 10' wide chimney and blanket drain. It had a wide core, but did not include a cutoff trench. Following substantial exploration at the dam site and review comments from headquarters, the design was changed in 1971 as shown in Figure 12. This design change added a cutoff to competent rock and reduced the size of the core.

The Corps contracted excavation of Ririe and construction separately. That turned out to be a fortunate plan, as the foundation conditions encountered were worse than expected. Instead of immediately starting construction of the dam, construction was delayed for a year as the embankment was re-designed and a more extensive foundation grouting program was completed. The design anticipated that upstream and downstream rockfill materials would be highly pervious. The core was widened, the foundation contact was widened, and gravel zones that were compatible with the sand filter were added upstream and downstream of the core (Figure 13). The delays and re-designs cost the Corps more than $4 Million on a project that was originally expected to cost $11 Million (1974 dollars). Although the District design staff was concerned about the foundation as well, some of the revisions were initiated during the headquarters on-site design review. Most levels of the Corps felt that the revisions were necessary and agreed to the changes despite the significant cost increase.

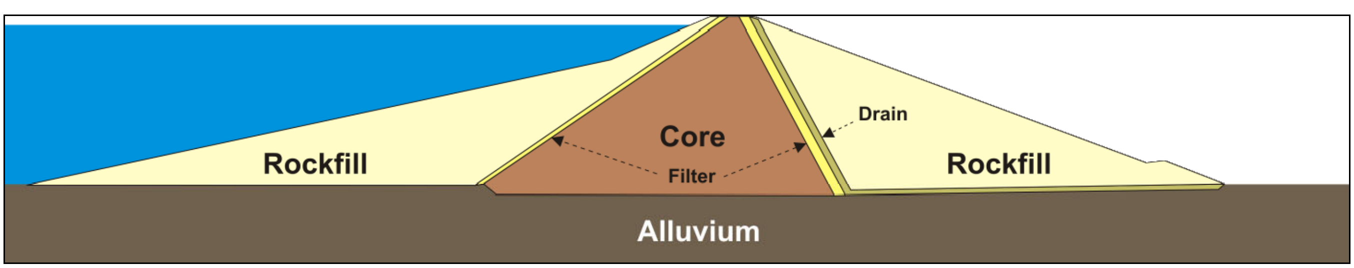


**Figure 11.** Ririe Dam Cross Section 1966

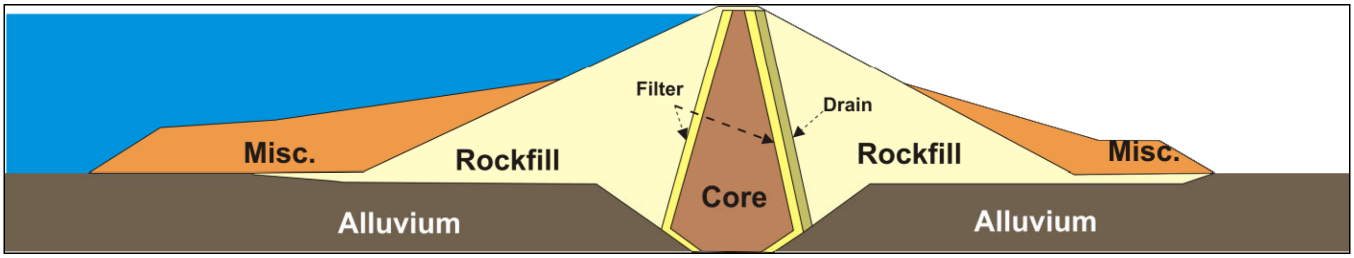


**Figure 12.** Ririe Dam Cross Section 1971

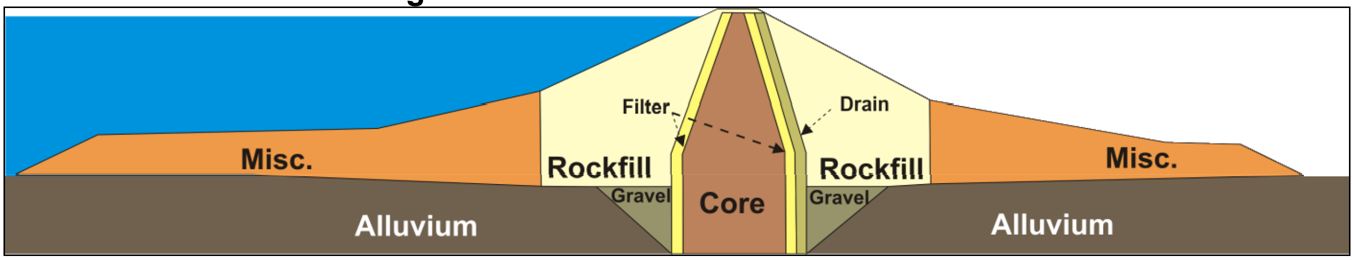


**Figure 13.** Ririe Dam Cross Section As Constructed

### *Foundation Preparation*

A number of foundation preparation and surface treatment provisions were employed to deal with the many foundation defects encountered during construction. The Corps extensively documented methods and locations, and numerous construction photos exist (Figures 14A and 14B). Zones of concern were protected by a shotcrete cover layer or mass dental concrete. The Corps began applying the lessons learned from Ken Lane's trip report about the Fontenelle Dam incident by specifying conservative treatment of contact surfaces between dams and their foundations and the contact surfaces of adjacent zones of different materials. At about the same time that the Corps was using shotcrete to protect the foundation, a few miles away, Reclamation was rejecting the use of shotcrete for a key trench in very similar material.

Particular attention was paid to the foundation treatment in the impervious core and filter zones of the foundation. Immediately prior to core and filter placement, the rock foundation areas were cleaned by hand labor using shovels, brooms, and air-water jets. Loose rock and debris were removed and exposed fractures were sealed with a broomed-on sand-cement slurry. Cavities, depressions, wide joints, and shear zones were cleaned out to depths equal to the width and backfilled with dental concrete. In some areas, hundreds of yards of concrete were placed to ensure a satisfactory foundation. Thin coatings of shotcrete or concrete that could crack or break under embankment placement were not permitted. Foundation treatment details included:

- Shotcreting breccias and clay interbeds exposed on the abutment keyways.
- Placement of sand and gravel filters over clay interbeds below the upstream and downstream shells.
- Faults were treated by excavating loose material and backfilling with concrete.
- Placement of dental concrete and fillets.
- Abutment drainage provisions.

**Figure 14A.** Construction photo from Ririe showing dental concrete and foundation cleanup.

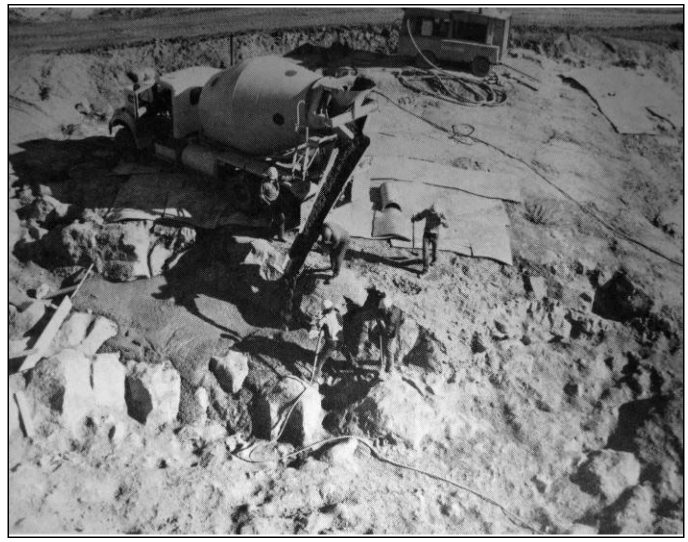

**Figure 14B.** Construction photo from Ririe showing the placement of dental concrete.

## Ririe Dam Performance

Seepage is measured at Ririe in two locations. The flows measured in the toe drain system have decreased over the last 35 years. Typical flows from the spillway drainage gallery are quite low as well. No other seepage has been observed. No material has been observed moving into or from the foundation. The piezometers respond to reservoir levels typical to those of other mature reservoirs. The piezometric levels indicate an effective reduction in head by the Zone 1 core. Regular visual inspections have not revealed unusual conditions.

## Teton Design and Construction

The project was initially envisioned as a run-of-the-river power generation facility with limited storage capacity. Field reconnaissance of dam sites was done from 1904 through 1945 in various locations on the Teton River and its tributaries. Eventually, Reclamation decided to select a dam site to maximize storage, minimize the distance to the lands that were to be irrigated, and minimize construction costs. Further reconnaissance was completed from 1945 to 1965. The project was eventually designed to provide irrigation, flood protection, and power generation in the lower Teton region of southern Idaho. The dam was finally located just upstream of Newdale, ID on the Teton River (Figure 15A). Figure 15B shows a plan view of the embankment as it was constructed.

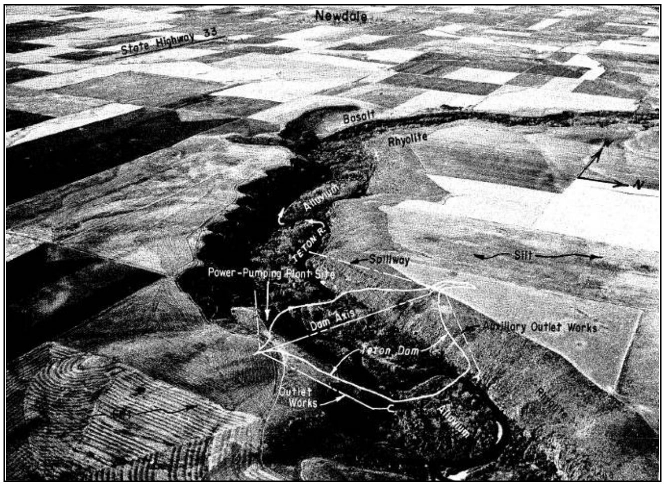


**Figure 15A.** Photograph of the plan for the construction of Teton Dam.

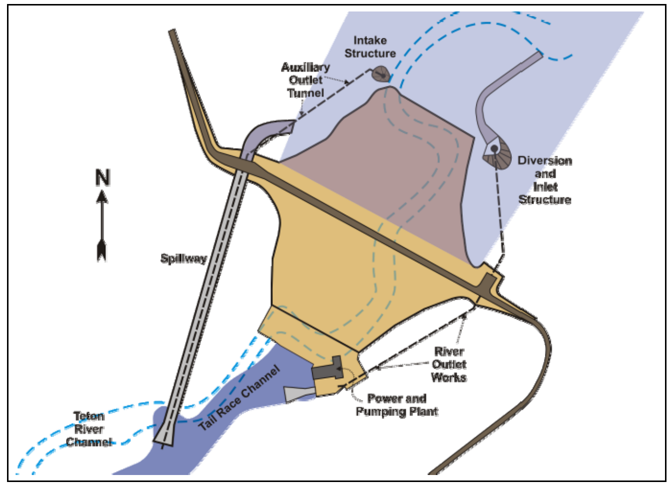


**Figure 15B.** Plan view of Teton Dam.

***General***

The dam was designed as a zoned earthfill with a crest elevation of 5332, a maximum height of 305 feet above the valley floor and 405 feet above the lowest point excavated in the foundation. The crest length was 3,100 feet. There were 5 embankment zones:

Zone 1 - Central Core
Zone 2 - Upstream and Downstream material adjacent to Zone 1 and in a blanket under zone 3 in the river valley and abutments
Zone 3 - Random fill downstream of zone 2
Zone 4 - Upstream cofferdam, later incorporated into upstream toe of dam
Zone 5 - Protective exterior upstream and downstream rockfill

There were other appurtenant structures associated with the dam:

- A three-gated chute spillway on the right abutment
- An auxiliary outlet works and access shaft in the right abutment
- A power generating station and pumping station at the toe of the left groin
- A river outlet works tunnel and gate shaft in the left abutment

***Left Abutment Test Grouting Program***

Pre-construction exploration consisted of:

- 26 holes between 19.6 and 556.0 feet deep in the area of the left abutment
- 9 holes between 58.5 and 505.8 feet deep in the valley floor
- 9 holes between 296.8 and 698.0 feet deep in the area of the right abutment
- 56 holes between 26.0 and 303.7 feet deep near the appurtenant structures (most of these were shallow)

During the feasibility phase of the project, a pilot-grouting program was completed in the area of the key trench on the upper portion of the left abutment. The results showed that above El. 5100, the upper 70 feet of rock was so permeable that blanket grouting was not practical from a cost standpoint. To compensate for the high grout losses in the pilot grouting program, a key trench was designed above El. 5100 to connect the embankment core to the rock foundation. To minimize costs, the trench was a narrow 30' wide at its base and had sidewalls or side slopes of 0.5H:1V for most of its length. On the left and right abutments, the sidewalls are near vertical in some locations. The deep key trench was a first for Reclamation and won a design award for its cost-effective approach, which was later rescinded.

### *Environment and Funding*

Before construction of the dam could begin, a group of environmental organizations filed a complaint in Idaho District Court on September 27, 1971 (Trout Unlimited v. Morton), to prevent construction of the dam. The lawsuit was dismissed from Federal District Court. The legal actions continued through 1974, with the plaintiffs alleging violations of numerous laws, including the National Environmental Policy Act of 1969 (NEPA). On December 23, 1974, the Ninth Circuit Court filed an opinion affirming the District Court's dismissal of the case, effectively ending the lawsuit. One of the main claims made by the plaintiffs was that the costs and benefits were misrepresented by the Government. The cost-benefit ratio for the Teton project was 1.0:1.75, which is not a high ratio. Design engineers, construction forces, and other Reclamation employees who were associated with the Teton project recall being constantly reminded about the extremely tight budget for the project. The likely source of the cost pressure was the cost-benefit ratio and the somewhat difficult process obtaining authorization in the face of litigation.

As *Trout Unlimited v. Morton* went through the courts, the Teton Basin Project moved forward under Project Construction Engineer Robert Robison. Reclamation awarded the contract for Teton Dam and Power and Pumping Plant to Morrison-Knudsen-Kiewit, and the contractor received the notice to proceed on December 14, 1971.

### *Materials*

On the uplands bordering the canyon the welded tuff is overlain by windblown silt, or loess, which ranges in thickness from less than 1 foot near the canyon edge to more than 50 feet. These deposits served as the source of the zone 1 material of the embankment. Alluvial deposits having a maximum thickness of about 100 feet at the dam site underlie the flood plain of the Teton River. This alluvium, used for Zone 2 and portions of the miscellaneous fill, consists of an upper unit about 80 feet thick composed of sand and gravel with some cobbles and boulders, and a lower unit about 20 feet thick composed of silt and clay. A blanket of slopewash generally less than 10 feet thick obscures large areas of the canyon walls. The slopewash consists of mixture of silty soil and fragments of welded tuff. The slopewash obscured the intensely fractured, near surface abutment rock.

The core material was low plasticity windblown silt (loess) mined from deposits above the right abutment. The average plasticity index ranged from 0 to 11 with an average of 3. However, 40% of the samples tested had no plasticity. The silt would classify today as a dispersive soil. Zone 2 was broadly graded; it was intended to serve as ‘filter’ and a drain downstream of the zone 1 core. Although the zone 2 was intended to function as a drain, the material in the remnant stands on a vertical face, calling into question the ability of the zone to perform that function. It did not meet filter compatibility with the zone 1; it met retention criteria, but contained too many fines to allow drainage and instead the entire dam performed more like a homogeneous embankment. Post-failure testing verified the lack of permeability.

### *Geology*

The Teton dam site is adjacent to the eastern Snake River Plain, a volcanic filled depression that was formed by downwarping and downfaulting in late Cenozoic time. Older volcanic rocks are not exposed along the edges of the plain. The Teton River incised into a portion of the volcanic upland near the eastern end of the plain creating a steep-walled canyon at the dam site. The site is in an area of generally low seismicity. The foundation cutoff trench was excavated into bedrock along the entire length of the dam. The regional groundwater table is far below the river, though perched groundwater can be found above the channel.

The canyon walls are composed of a rhyolite welded ash flow tuff. The tuff was exposed in some areas, but talus slopewash and alluvial deposits predominantly cover it. The welded tuff is between 50 and 600 feet thick near the dam site and has prominent and abundant jointing intersecting and high and low angles. Most of the joints are near-vertical. The major joint set, strikes N25W to N30W, and is well developed on both abutments and in both outlet tunnels. A second joint set, striking N60W to N70W, is well developed in the lower upstream part of the right abutment, the river outlet works tunnel, and the downstream portion of the auxiliary outlet works tunnel. A minor set of northeast-trending, high-angle joints is also present in the welded tuff.

Continuous high-angle joints in the right abutment have been traced for lengths of as much as 200 feet, but most are between 20 and 100 feet long. The aperture of most high-angle joints is less than one-half inch, but many joints are as much as several inches wide and some are several feet wide. Examples of the jointing can be seen in Figures 16 and 17.

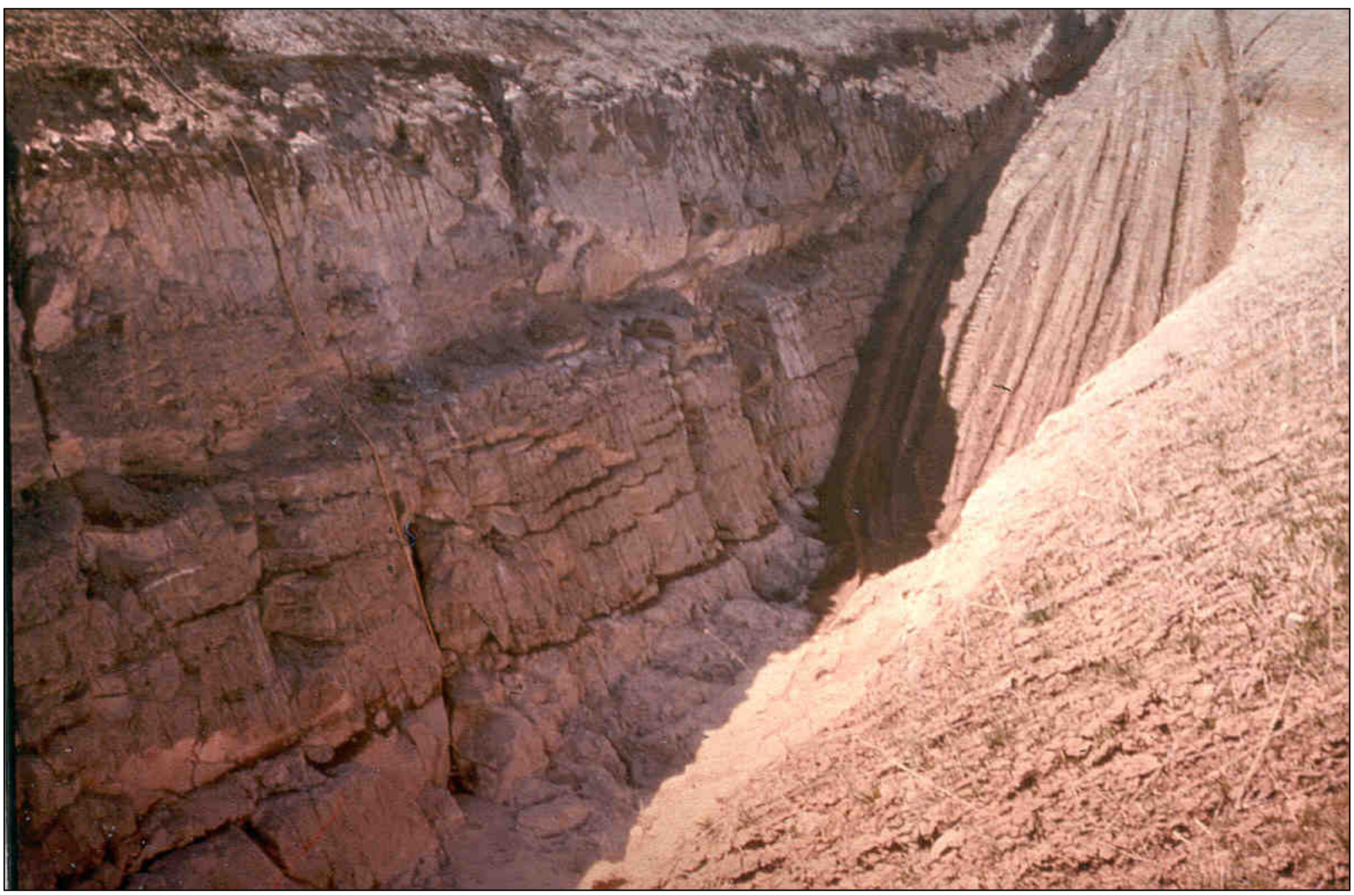

**Figure 16.** A view of the right abutment key trench showing the jointed nature of the foundation

Low-angle joints parallel the flat-lying or gently dipping foliation. Several low-angle joints in the upper part of the welded tuff have been traced for about 200 feet, and a joint-like discontinuity between the middle and lower units has been traced for at least 400 feet upstream from the dam centerline. Many joints are open; others are partially or wholly filled with clay, silt, silty ash, soil, or rubble, especially near the natural ground surface. The permeability of the welded tuff is due entirely to the presence of open joints. The joints are most abundant and open, and rock-mass permeability is much higher above El. 5100. Many of the joints were infilled with erodible material that would soften on contact with reservoir water.

Underlying the welded tuff are materials of lacustrine, alluvial, and pyroclastic origin. Information about these materials has come mainly from drill holes, commonly with poor or no core recovery, and to some extent from deep grout holes, and thus is rather fragmentary. Although there is little information about the various units underlying the tuff, sand and gravel and variably cemented sandstone and conglomerate are commonly present. Thick claystone and siltstone are present under at least part of the left abutment and channel section. Thin ash-fall tuff and other pyroclastic materials were found below the welded tuff in some core holes. The contact between the sedimentary materials and the welded tuff is an irregular erosion surface with a local relief of at least 440 feet and some slopes steeper than 30 degrees. The permeability of most of the sedimentary materials is less than that of the intensely fractured welded tuff, but is highly variable. The sedimentary materials are at least 390 feet thick; the depth to the materials underlying these sediments is unknown.

**Figure 17.** A view of a large fissure in the right abutment key trench

Basalt is present in the bottom of the Teton River Canyon and is a remnant of a lava flow that filled the canyon to about El. 5005 (see geologic section, Figure 18). In the dam foundation, the basalt is restricted to the left side of the river channel section, where it has a maximum thickness of about 124 feet. It is separated from the underlying welded tuff by a deposit of alluvial material consisting of silt, sand, and gravel from 4 to 22 feet thick. The basalt is dense to moderately vesicular and contains closely spaced, randomly oriented joints and other fractures. In spite of its fractured nature, it is an adequate foundation rock for the dam. Water pressure tests showed the basalt to be tight and the thin alluvial fill between the basalt and the welded tuff to be permeable.

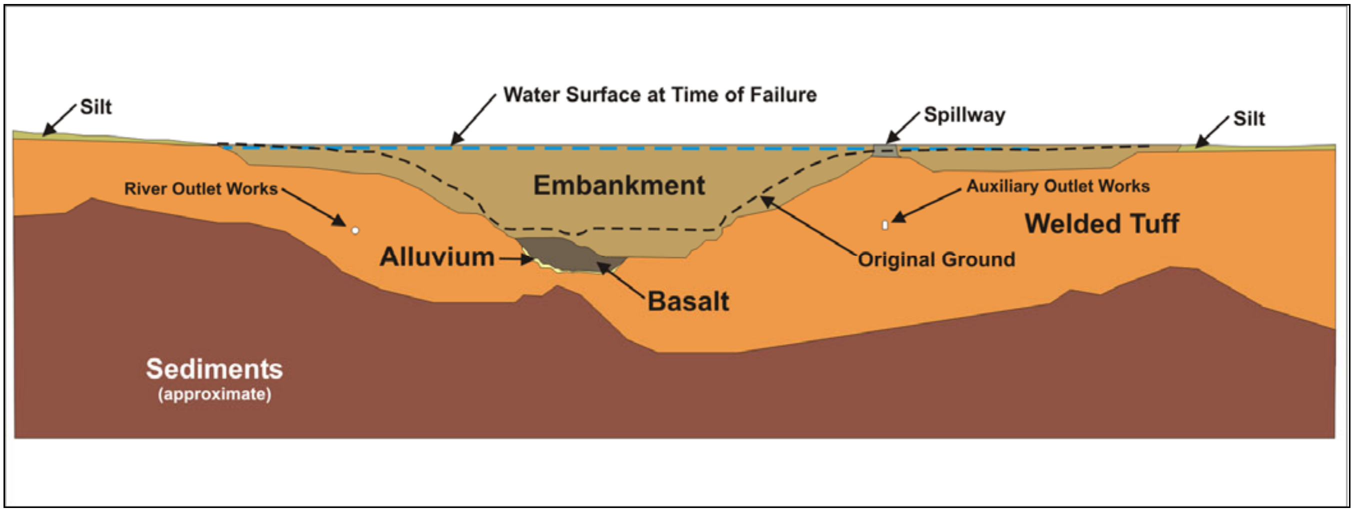


**Figure 18.** A profile of the Teton embankment and foundation looking downstream.

***Foundation Preparation***
Foundation preparation details were almost identical to the details from the Fontenelle design. The cutoff trench was excavated to competent rock. Foundation grouting was completed through a 3-foot wide concrete grout cap that ran the length of the centerline of the cutoff trench. Overhangs were removed where possible, although excavations in the left abutment remnant found areas where overhangs were not removed and abutment rock was not shaped. On the right abutment, shaping was limited because the location of the spillway prevented dramatic changes to the profile. No dental concrete was used and no slush grouting was used on the foundation surface above El. 5200. Below El. 5200, ‘dental concrete’ was limited to using structural concrete leftover from placements elsewhere on the site, as there was no bid item for either dental concrete or foundation treatment. The rock surface treatment may have hastened the failure, because the criterion for treatment was not based on condition of rock, but on when excess concrete was available. ‘Slush grout’ was similarly taken from leftover foundation grouting materials.

Special compaction was done at the contact between the embankment and the foundation rock. During excavation of the left abutment remnant, it was discovered that special compaction was impossible to perform next to some of the open jointing. Foundation grouting was difficult given the extraordinary permeability of the Welded Tuff. Occasionally, structural concrete was poured in the large fissures and voids in the foundation where grout was considered inadequate. There was no consistent method to treat the rock foundations within the key trenches and there was no strong direction from designers as to how to treat the foundation. It is apparent that none of the designers knew how to adequately treat the foundation and that the organization did not understand the importance of doing so. Treating the foundation would also have meant ignoring the cost and schedule pressure from the construction office.

***Design Considerations***
The design for Teton followed standard practices with an impervious core and progressively coarser materials used as the zoning progressed downstream. In the maximum section, a relatively shallow-sloped cutoff leads to a narrow 30-foot-wide contact with a competent foundation. The intent of the design was to rely on the grout curtain and the cutoff to create an impermeable barrier to protect the main core of the dam. As shown in Figure 19A and 19B, there are no additional defensive measures provided in either the main embankment or the abutment sections. On each abutment, the side wall of the cutoff trench steepens to 0.5H:1V. However, owing to the steepness of the abutment looking parallel to the axis, the actual sidewalls approach vertical on both abutments, which can easily be seen in Figure 20.

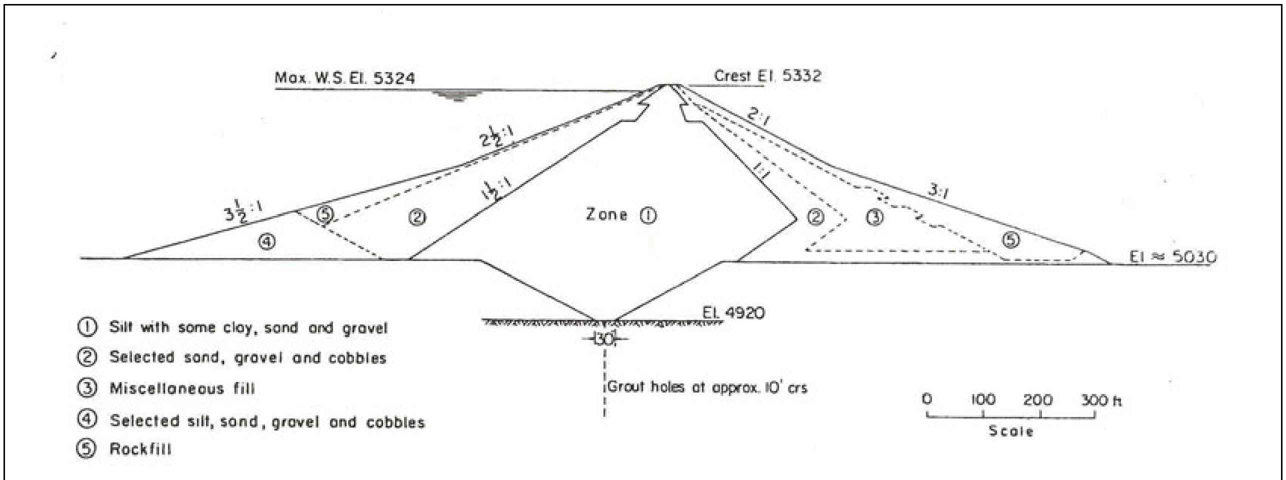

Fig.2. Cross-section through center portion of embankment founded on alluvium.

**Figure 19A.** Teton Dam maximum section[4].

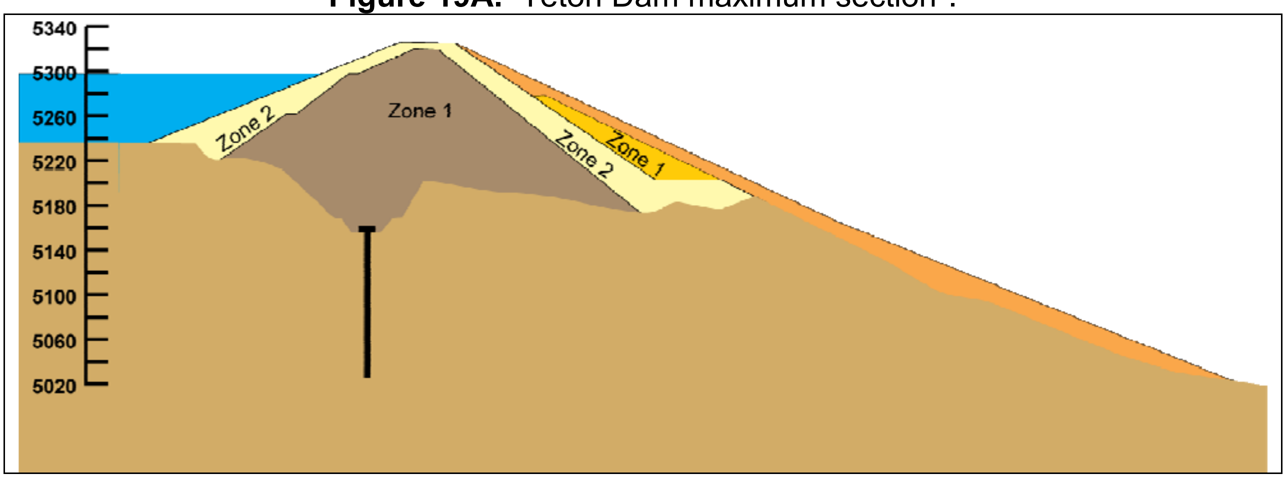


**Figure 19B.** Teton Dam right abutment section.

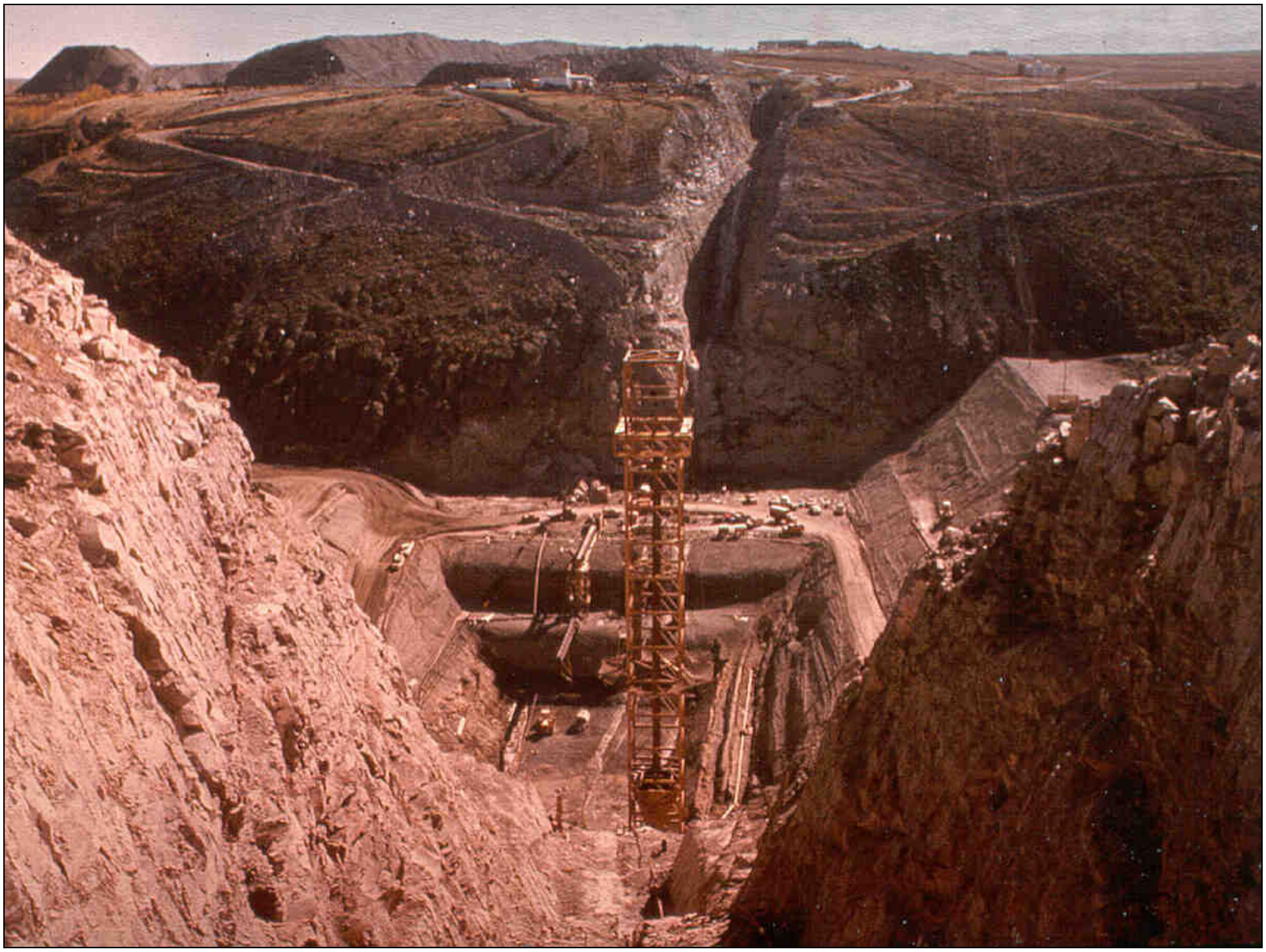

**Figure 20.** A view looking towards the left abutment from the right abutment.

## Teton Dam Performance

At the time the failure occurred, the power generating station was not yet complete and the auxiliary outlet works was under construction, limiting the release capacity of the facility during first filling. As the reservoir began to fill for the first time, a large amount of water pressure built up quickly in the lower portion of the cutoff trench on the right abutment probably between El. 5050 and El. 5200. Untreated joints in the welded tuff in the upstream wall of the cutoff trench allowed full reservoir head to build up on the upstream face of the cutoff and an extraordinarily high gradient to be induced across the trench. Because of the narrowness of the cutoff trench, there were potentially low-stress areas due to soil arching. Soil arching in the right abutment key trench was demonstrated using finite element analyses[4,5]. Following the investigations of the Independent Panel and the Interior Review Group, Jaworski, Seed, and Duncan[8] demonstrated by laboratory testing that soil on the upstream face of the key trenches that was adjacent to open joints would soften and collapse after wetting, allowing reservoir water closer to the grout cap and further increased the already enormous hydraulic gradient across the floor of the key trench.

Either in the lower portion of the cutoff or across the top of the grout cap, seepage from the high gradients began to erode material from the downstream face of the cutoff trench and carry it through the open jointed welded tuff to unprotected exits in the valley wall and through the pervious downstream embankment zones. Eventually, enough material washed out allowing erosion to progress upstream and creating an open pipe that connected to the upstream open joints and eventually connected to the reservoir. The pipe enlarged, progressed vertically and laterally, and breached the crest of the embankment causing a catastrophic failure and release of the entire pool[4,5]. The uniqueness of the failure was the very rapid progression from discovery of a rather minor seep at the abutment to the total collapse of the dam. The speed of the progression to failure highlights the gross inadequacy of design and treatment to control seepage through the foundation

This embankment failure is the largest in the United States, based on the dam's structural height. 300 square miles extending 80 miles downstream were fully or partially inundated. 25,000 people were displaced, 11 people were killed, and the flood caused approximately $400 million in direct and indirect damage.

## Conclusions

Many interviewees from both the Corps and Reclamation expressed surprise that a dam could exhibit signs of distress at 7:00am and fully breach by 11:30am. Before Teton, this seemed implausible to even experienced dam designers. The rapid nature of the failure and the vast devastation downstream changed both the dam engineering community and the public's perception of their own safety.

### *Engineering*

The engineering factors that contributed to the failure of Teton are numerous and not all are presented here, but the key findings (extracted and combined from the various panels and interviews) are:

- There was little evidence of slush grouting in joints exposed in the key trench walls. The embankment's safety depended entirely on an intact and comprehensive grout curtain to reduce the potential for erosion. There were numerous locations where the silt-filled joints and cracks may have limited grout travel, softened upon wetting and eroded into larger downstream cracks.
- Slush grouting stopped at El. 5200. Neither the designers nor the liaison engineer were aware of the decision to stop slush grouting until after the failure of Teton Dam. According to field personnel, the field geologists also played no part in the decision to stop surface grouting. No reason has yet been found as to why this practice was discontinued.
- The rock surface was not adequately sealed under the impervious core surface upstream and downstream from the key trench. The implication of this information is that reservoir head would have been felt upstream and atmospheric pressure would have been felt downstream. The resulting gradient across the core trench would have been equal to the entire reservoir head dissipated across either the key trench or the grout cap.
- The dam failed as a result of inadequate protection of the impervious core material from internal erosion. The most probable physical mode of failure was cracking of the impervious core material either due to hydraulic fracturing or differential settlement within the embankment that allowed the initiation of erosion. It is also possible that damaging seepage started at the contact of the zone 1 material and the rock surface.

- However, while the first Independent Review Panel (IRP)[4] study concluded that the dam was built as specified, the Interior Review Group (IRG)[5] cited instances where that was not the case. Infractions of specifications were found in the construction of the sidewalls of the key trench and the grout curtain. In addition, inspection procedures for control of placement of zone 1 material were not always adequate. Among the construction faults found were:
    - Overhangs and ledges of rock on slopes within the key trench were not prepared adequately to receive fill.
    - Dry, low-density fill was placed in zone 1 and accepted as suitable because of the incomplete procedures used for evaluation of density data from construction control tests.
    - Low density material was placed against and in open cracks in the key trenches.
- Despite these findings, a technical analysis accompanying the report states: "If a defensive design had been provided for protection of the core of the dam, the consequences of these deviations would probably not have been serious enough to contribute to a failure. Because of inadequate design, these construction deviations may in some way have contributed to one or more of the possible modes of failure."

Between Reclamation and the two independent panels' forensic investigations, the grout curtain was drilled and explored, the foundation permeability was examined, wet seams in the embankment explored, hydraulic fracturing tests were completed in the embankment remnant, samples were taken and tested for strength, gradation, and erodibility, and numerical modeling was completed. Many papers are available on the technical aspects of the failure. One of the unfulfilled expectations of the excavation of the abutment was to find an in-progress seepage pipe of embankment materials into foundation rock joints.

The key overarching conclusion from the failure is that the design of Teton Dam was not uniquely tailored to the site. The intense fracturing and open joints were not compatible with low-to-no plasticity silt. The left abutment grouting program should have been an indication of how susceptible the embankment might be to erosive forces. Instead of incorporating this into the design philosophy, the designers attempted to change the parameters of the problem by using the key trench in place of blanket grout.

Experience from Fontenelle should have indicated that the embankment-foundation contact is a critical consideration for both design and construction, but the lessons learned from Fontenelle were not applied at Teton.

Experience in the dam engineering industry was not heeded with respect to filters. At the Corps' Kansas City District, the North Pacific Division, and perhaps other districts and divisions, designs in the late 1960's incorporated elements that are part of modern filter design. This information was available to Reclamation as well, but this knowledge was not incorporated into the Teton Dam design partly because of the success enjoyed by years of acceptable performance at Reclamation dams. While, as would be expected by the individual nature of each dam site, no Reclamation dam has encountered the combination of dispersive embankment materials and a highly pervious foundation found at the Teton Dam site. However, much of Reclamation's Dam Safety resources have been required at numerous other structures to resolve inadequate control of seepage through embankment foundations and to improve inadequately filtered adjacent material zones.

***Organizational Responsibility and Communication***

Reclamation's organization and philosophy contributed to the failure of Teton in the following ways:

- Lack of communication between the design branches in Denver made it difficult to learn from mistakes.
- There was insufficient to non-existent review of designs and specifications by experienced designers, external experts, and field personnel.
- The designers failed to understand and incorporate actual geologic information into the design and geologists failed to present any strong resistance against the misinterpretation of this data.
- The absence of quality communication between the designers and the field offices did not allow changes to be made to the design that would have appropriately reflected field conditions.
- There was reluctance on the part of Reclamation's designers and construction engineers to adapt during construction. This was in large part caused by the inexperience of many embankment designers, even though they may have been at Reclamation for many years. The feeling was that someone else would catch any problems.
- Designers did not receive an influx of new design ideas from external contacts, in-house lectures, or onsite experience.
- Many designers did not have either the experience or the courage to make controversial design decisions.
- There was a failure of the embankment dams branch to report or learn from its own mistakes.

Design branches in Reclamation in the 60's and 70's did not enjoy the same communication they enjoy today and did not share information to learn corporate lessons. The designers on Teton Dam and the rest of the organization did not benefit from the lessons learned from a serious incident at Reclamation's Fontenelle Dam 10 years earlier under similar circumstances. Construction forces indicated that had they been informed about the specifics of the Fontenelle incident, they would have attempted to change the approach at Teton, although financial and schedule pressures may have pre-empted their good intentions. There is some indication that some in Reclamation may have taken great care to conceal the Fontenelle incident from engineers working in Denver and elsewhere.

Reclamation's past success building dams led to a sense of confidence that was unwarranted for a site as challenging as the Teton site. Branch chiefs and division chiefs reviewed designs, but rarely did designers outside each branch or design team review each other's design. Reviews external to Reclamation were not done.

Conflicts existed Reclamation-wide between construction offices, geologists, and designers. It was a dual failure on the part of the organization. The construction offices did not welcome designers on site, which stems at least partly because the designers did not offer useful guidance or advice when they were on site. This relationship is difficult to describe adequately, but in essence, once designers designed a structure, they turned the specifications over to the construction office which then built the structure according to the specification. Both groups were comfortable with the situation and many dams were built under these circumstances. This situation arose out of the early history of Reclamation when it was extremely difficult for designers to communicate with and travel to distant design sites. Because of the tensions between design and construction, the principal designer spent a total of only 16 hours on site for the duration of the construction of Teton Dam.

Once decisions were made and designs turned over to the construction office, the organization rarely supported changes to the specification. This was a particular problem at Teton Dam as the design team did not envision – though perhaps they should have – the foundation conditions that were uncovered during the excavation. The cost-benefit ratio for the project was so tight, that significant funding pressure was exerted on both the designers and construction forces. The combination of overconfidence in established design practices, organizational reluctance to modify designs to reflect differing site conditions, and restrictions on available funding were key factors leading to inadequate foundation treatment that contributed to the failure.

A subcommittee of the House of Representatives Committee on Government Operations held hearings on the failure in August 1976, chaired by Congressman Leo J. Ryan. Members of the subcommittee questioned Harold G. Arthur, Reclamation's Director of Design and Construction. During the hearings, Congressman Ryan raised what he called the "momentum theory." Ryan asked both Arthur and Reclamation Commissioner Gilbert Stamm if Reclamation would stop construction of a dam after it advanced past the foundation excavation stage. He especially seemed concerned when Stamm informed him work on the foundation of Auburn Dam, in California, continued after postponement of actual dam construction. Harold Arthur admitted to Ryan that although Reclamation often stopped a project before construction started, it had never stopped a project once actual construction of a dam started.

Gilbert Stamm testified to the Senate subcommittee on January 24, 1977, that Reclamation believed the deep narrow key trench was the main contributor to failure, saying:

> *"That element of the design was unique and appears to have been the feature that gave rise to the series of circumstances that permitted internal erosion that led to failure."*

Mr. Stamm admitted that because giving the key trench a shallower slope would have increased the cost, the design was not changed.

### *Individual Responsibility*

Designers noticed the differences between what the geologic investigations indicated and what the observations were in the key trenches. They developed alternatives to address the fissures seen in the walls of the key trench. Although some surface treatment was accomplished – in the form of limited slush grouting – it was not consistent, planned, or part of the specifications. The construction office objected to the change in the key trench design, and – sensitive to the financial constraints on the project – these objections were sustained by the design managers in Denver. The designers also approved a faster-than-normal filling of the reservoir to take advantage of a large snowpack. Engineers in Denver responsible for the design of Teton Dam exacerbated the organizational problems by not being firm with their requests to change the design of the key trench and restrict the filling of the reservoir. There was significant pressure to refrain from delaying the construction, and the designers bowed to this pressure in several key instances. Engineers involved with the initial design of Teton indicated that the organization as a whole understood the difficult geologic situation encountered at the Teton site and assumed that because many site issues had been listed in the design considerations report that someone else had taken care of the issues.

Reclamation's construction forces in the early 1970's had a wealth of dam building experience. At Teton, the ground-level construction inspectors and laboratory field staff had reservations about the composition of the core and the steepness of the abutment key trenches. Laboratory staff observed unpredictable behavior of the core material including:

- Extreme sensitivity to the moisture content of the silt. There were situations where the optimum moisture content varied 10-12% within close proximity.
- A tendency to have areas of very low density material in the embankment (less than 85 lbs/ft$^3$) when material was borrowed from certain areas.

Construction inspectors remarked that the right and left abutment key trenches were too small to facilitate good compaction and were concerned that good foundation contact could not be assured given the steepness of the abutments. They also were restrained from using 'dental' concrete to treat open fissures because of cost considerations. When excavating the left abutment remnant, overhangs in the foundation were observed despite the specifications prohibiting this condition.

The general sense of unease on the part of the experienced field staff, their observations about specific potential deficiencies, and the failure of design and construction managers to solicit and incorporate the field observations had a subtle but important contribution to the failure of Teton Dam. Field staff and design staff are sensitive to the focus of management no matter the situation. Intense management focus on keeping costs low, which was repeated and emphasized by each individual interviewed, trickled down to the lowest levels of design and construction. Had the same attention and focus been given to protecting the core from erosive forces as it was to keeping costs low, perhaps a different outcome would have been realized.

***Decision-Making***

The IRG noted that "design notes, developed early in the design process, indentify and report a variety of potential design problems and possible design alternatives. However, there are no records, documents, or reports that show: (1) the logical resolution of each of the identified design problems, (2) why a particular design alternative was considered satisfactory and selected in preference to others, and (3) why an identified design problem was subsequently judged important or not important and omitted from, or included for, further consideration."

The IRG also noted that "surface grouting stopped at El. 5200. Neither the designers nor the liaison engineer were aware of the decision to stop surface grouting until after the failure of Teton Dam. According to field personnel, the field geologists also played no part in the decision to stop surface grouting."

Documenting decisions is a critical step to ensure safety decisions are given proper considerations. During the design and construction of Teton Dam, decisions were made by both design engineers in Denver and construction engineers in the field that remain undocumented.

Reviewing the decisions made during construction, what seems to be very clear is that the designers seemed not to be aware of the magnitude and threat that the dam site posed. They submitted to the pressure of the Region and Project Construction offices to keep costs low and keep the project on schedule. In the one clear case where shotcreting of the key trench walls was proposed in the 1974 construction season, it was dismissed without documentation and referenced with only a one-sentence comment in the post-failure investigations when it was mentioned by the construction liaison. Once construction began, no recommendations originating from the designers were made that significantly altered the dam design or delayed the contract.

***Lessons for the Future***

Changes recommended by the various panels were implemented by Reclamation by 1980. Reclamation also implemented many changes in addition to the recommended changes that had a more dramatic impact on the culture. Engineers that began their careers following the failure of Teton began with an entirely new attitude that embraced state-of-the-art ideas, external review of decisions, and significantly more interaction between design and construction forces. Following the failure, Reclamation focused significant energy on education and it thrived on in-house and external educational opportunities

The final lessons to be learned from the near-failure of Fontenelle Dam, the failure of Teton Dam, and the success of Ririe Dam are:

- Use more than one line of defense against seepage.
- Flaws can occur in man-made structures, and defense measures should be designed assuming that flaws do occur.
- External review of designs and decisions is a key step to evaluate the safety of a structure
- Critiquing problems and discussing controversial conditions is an important step to understand problems and planning the resolution to them.
- A central presence to facilitate communication between geologists, designers, and construction forces is important.
- Foundation approval documented by designers and geologists for each square foot of material placed is an important consideration. Digital records of the foundation inspections should be required in the specifications.
- Incidents and failures should be openly discussed and presented as learning tools for all dam engineers.
- The decision structure of the organization must be continually observed and evaluated to see that effective decisions are being made.
- Communication with the downstream population is an important step to mitigate potential disasters.
- Hubris has no place in dam engineering.


**Figure 21.** The consequences of poor design.

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