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Reconsideration of Failure Initiating Mechanisms for Teton Dam

G. A. Leonards
Purdue University, West Lafayette, Indiana

L. W. Davidson
Bureau of Reclamation, Denver, Colorado

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G. A. Leonards

Professor of Civil Engineering, School of Civil Engineering, Purdue University, West Lafayette, Indiana

L. W. Davidson

Supervisory Civil Engineer, Embankment Dams Branch, Bureau of Reclamation, Denver, Colorado

SYNOPSIS A review of failure mechanisms previously suggested for Teton Dam indicated that they were not fully supported by all the available data. The original mechanisms are reviewed and a failure initiation mechanism is then proposed which does satisfy all the available data, and laboratory tests supporting this mechanism are presented. Additional lessons to be learned from the failure are listed.

INTRODUCTION

Teton Dam failed on June 5, 1976. It was the first failure in the Bureau of Reclamation's 74-year history of dam construction. It was the highest embankment dam that had ever failed catastrophically in the entire history of earth dam construction. These facts suggest that a unique combination of factors was responsible for the demise of the dam, and that full understanding of the mechanism of failure may make an important contribution to the state-of-the-art.

The failure was investigated officially by two groups, the Independent Panel (IP) and the Interior Review Group (IRG). The IP was composed of nine engineers of international repute who completed their investigation and published a report of remarkable quality in the short time of 6 months (IP, 1976). The IRG was composed of representatives from five Federal agencies concerned with dam construction; they published two reports, one in April 1977 (IRG, 1977) and another in January 1980 (IRG, 1980), the latter report following extensive excavations at the left abutment of the dam. A review of the findings given in these three reports, and a reanalysis of the factors involved, was presented at the X Intern. Conf. SMFE in 1981 and published in Proceedings (Seed and Duncan, 1982). Why, then, yet another review of this failure?

Teton Dam was breached very rapidly before the reservoir level reached the spillway sill on first filling. It is agreed that plugging of open joints in the bedrock, and filtering of the zone 1 core material should have been provided for, and that great care should have been taken in the construction of these design features. However, there is little agreement on the mechanism(s) that initiated the failure; hence, it is not certain that these measures would necessarily have prevented the failure from occurring. Moreover, if the actual mechanism of failure were understood, protection against future failures might be accomplished more reliably and at less cost. For these reasons, it is important to identify correctly the mechanisms that initiated failure.

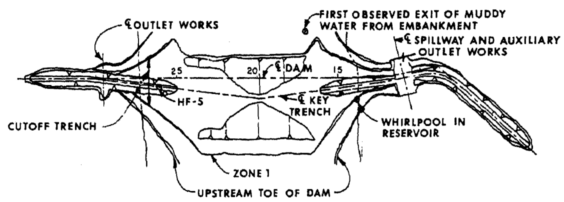
The paper reviews the likely failure mechanisms postulated in previous publications with particular attention to their evolution, and to the progressive enlargement of the lessons learned therefrom. Two new failure initiating mechanisms were reported by the IRG in 1980 and four additional mechanisms were advanced by Seed and Duncan in 1982. In our opinion, none of the failure mechanisms previously postulated are fully supported by all of the existing data. Accordingly, a failure mechanism is proposed which stands up to such scrutiny and the results of experiments, specially designed to test its validity, are reported. In conclusion, new lessons are added to the existing list, which have important implications for improving earth dam design and construction practices.

REVIEW OF PLAUSIBLE FAILURE MECHANISMS

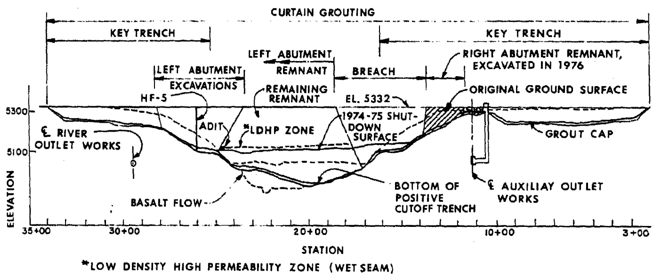
General Considerations

It is not feasible, nor is it necessary to present herein details of the site conditions, of the design features and the dam configuration adopted, and of the sequence of events preceding failure, as an excellent summary of all these facets is readily available (Seed and Duncan, 1982). However, for general reference, a plan and a profile along the dam axis is shown in Fig. 1. In particular, the geometric similarity of the key trenches in the abutments is noted. From the location where muddy water was first observed to emerge on the downstream face of the dam, from the pattern in which the failure progressed, and from the location of the whirlpool that developed in the reservoir prior to breaching, it can be deduced that failure was initiated in the key trench near Sta. 14+00 and approximately at El. 5200.

*Opinions expressed in this paper are solely those of the authors and do not reflect those of the Bureau of Reclamation.



(A) PLAN OF KEY TRENCH
AND CUTOFF TRENCH



(B) PROFILE ON CENTERLINE CREST
OF DAM (LOOKING DOWNSTREAM)

FIG. 1 TETON DAM

The acceptability of a proposed failure mechanism can be judged by the extent to which it explains the following observed facts:

1. Failure occurred at the right abutment only. Excavation at the left abutment disclosed no evidence that failure by internal erosion was imminent, or that piping had occurred to any extent, yet the size and frequency of open joints in the rock were as severe at the left abutment key trench as they were at the right. Thus, some other weakness must have existed at the right abutment that was not present at the left. None of the failure mechanisms postulated thus far have attempted to identify the nature of this weakness.

2. Failure occurred in an extremely short period of time. The dam was breached in less than 50 days after the reservoir first reached El. 5200, where failure is believed to have been initiated. It is likely that the reservoir was at an elevation sufficient to induce piping for only a few days (perhaps only hours) prior to failure. Thus, an explanation for the cause of failure must include features that would permit the demise of the dam to occur so rapidly.

Previously postulated failure mechanisms will now be examined in the light of the two criteria listed above.

Findings of the Independent Panel (1976)

The IP concluded that the triggering mechanisms most likely to have led to the failure were:

1. ". . .the flow of water aginst the highly erodible and unprotected key trench filling, through joints in the unsealed rock immediately beneath the grout cap near Station 14+00 and the consequent development of an erosion tunnel across the base of the key trench fill."

and

2. ". . .cracking caused by differential strains or hydraulic fracturing of the core material filling the key trench. This cracking could also result in channels through the key trench fill which would permit rapid internal erosion."

The Panel noted that their description of the failure mechanism did not provide a final answer to the specific cause of failure, but they did conclude that the choice was between ". . .imperfect grouting of the rock below the grout cap, or cracking in the key trench fill [near Sta. 14+00], or possibly both."

Although the grout curtain was installed in a workmanlike manner, there is no doubt that flow could occur below the grout cap through open joints in the rock and "windows" in the grout curtain. However, for piping to be initiated from the bottom of the key trench fill to create an erosion tunnel across the base of the trench (IP, 1977, Fig. 12-4, p. 12-8), there must have been open joints at the base of the key trench.

Davidson personally observed the floor of the left key trench after it was exposed: he affirms that no open joints were visible in the floor of the trench. Other Bureau personnel that were onsite confirm this view. There were cracks through which seepage could occur, but the openings were hardly larger than those of an acceptable filter. As the evidence is overwhelming that there were no open joints in the bottom of the left key trench, and as conditions in the right key trench were stated to be better than in the left (see depositions by Bureau field personnel; IRG 1977, pp. C-14 to C-22), the IP's postulate that failure was initiated by piping through open joints at the bottom of the key trench, and the consequent development of an erosion tunnel across the base of the key trench fill, is not supported by existing data.

Open joints certainly existed in the upstream and downstream walls of the key trench. Cracking caused by differential strains within the core material filling the key trench could readily develop into erosion channels across the key trench fill leading to rapid failure of the dam. However, careful examination of the slopes of the two abutments and key trenches, of the relative overall quality of the embankment constructed at the two locations, and of the potential for unequal loading due to different construction sequences (or to seismic activity) did not disclose any reason to support a greater tendency for differential settlement at the right vs. the left abutment. As "no cracks attributable to differential settlement of the embankment were found during the left embankment excavation" (IRG, 1980, p. 7-1), it is very unlikely that this mechanism was a contributing cause of failure.

Subsequent to the failure, an open joint (No. 172) was mapped striking diagonally across the key trench floor between Sta. 14+64 and 14+83 (IP, 1976, p. E-7). In this zone, much of the grout cap had been removed by the enormous flow of water that ensued. Because inspectors had reported that open joints in the floor of the key trench had been sealed, we believe that the seal in joint No. 172 was similarly removed.

Hydraulic fracturing (HF) is an alternate way in which cracks could develop in the key trench fill. The Independent Panel argued that this mechanism of failure was plausible on the following grounds:

1. HF tests were performed in drill holes made into the unfailed portion of the left embankment remnant to determine the water pressures required to cause fracturing.

2. Finite element (FE) analyses were made to determine the corresponding stress distribution in the embankment. Soil stress-strain parameters were selected which would correctly predict the result of the borehole HF tests.

3. The parameters derived in this manner were then used in FE analyses to evaluate the stress distribution in the dam section where it was deduced failure had occurred and thereby assess the possibility that HF could cause cracks to develop in the core of the dam due to water pressure on the upstream face.

On this basis, the Panel concluded that HF was among the three most plausible mechanisms of failure. We disagree with the basis for this conclusion, for the following reasons:

1. In October-November 1976 the IP caused three holes to be drilled in the left embankment remnant for HF testing. One hole (HF-6) was abandoned due to difficulties in drilling; in the second hole (HF-7) hydrofracturing did not occur even when the casing was filled to its top elevation of 5318.5 (maximum reservoir elevation was 5302). The difficulties encountered with the third hole (HF-5) have been detailed by Davidson (1978). Drilling started with a rock bit and clear water at the location shown in Fig. 1. At a depth of 101 feet (El. 5212) all drill water was lost. Drilling continued for another 49 ft; during this drilling, 3,000 gal of water were pumped in within an hour with no return. An inner casing was placed and the annular space was sealed at the bottom for approximately 2 ft with cement grout. A smaller rock bit and air pressure were then used to extend the hole another 40 ft, at which time the seal between the casings was broken. Later, a new inner casing was set and plugged, but attempts to extend the hole below El. 5129 resulted in caving and sloughing into the hole with mud and muddy water noted to El. 5159. The IP concluded that fracturing had occurred when borehole water was first lost at El. 5212 and used a fracture head of 101 ft to back-calculate the stress-strain parameters in the finite element analysis.
During excavation of the left abutment remnant in 1977, no trace was found of the 3,000 gal of water pumped into hole HF-5 between El. 5212 and 5163. No fracture plane was found at the elevation of first water loss into the hole. In the light of this hindsight, there is no evidence to support the fracture head adopted to back-calculate stress-strain parameters for the soil. Thus the reliability of all further analyses using these parameters is open to serious question.

2. A two-dimensional FE analysis was performed with soil properties assumed to be isotropic. Considering the steepness of the abutment walls, the anisotropic nature of compacted clay, and the sensitivity of calculated lateral stresses to values of Poisson's ratio, there was little reason to place much credence in the results of the FE analysis. This is especially true since the calculated fracture pressure for hole HF-7 was 6.4 ksf while, in fact, in the only hole to be drilled and tested without incident, hydraulic fracturing did not occur at a water pressure of 7.8 ksf.

While the IP pointed out the limitations of the analysis and stated that "the criterion of initiation of hydraulic fracturing utilized herein may require modification," it seems that, even at the time, there was at least as much reason to question the viability of this mechanism of failure as there was to cite it as a prime candidate.

In retrospect, careful and detailed examination of the excavations at the left embankment remnant disclosed no evidence of hydraulic fracturing - in the sense envisioned in the IP report - due to pressure from the reservoir*. Unless it can be shown why hydraulic fracturing should occur at the right abutment and not at the left the sudden formation of vertical cracks by hydraulic fracturing, that would progress rapidly to form continuous channels from the upstream to the downstream walls of the key trench, should not be considered a plausible mechanism of failure.

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* Cracks due to drilling using air pressure of up to 150 lb/in^2 were indeed observed, but no fractures due to water pressure from the reservoir were found.

Findings of the Interior Review Group, 1977

In their first report, the IRG (1977) stated:

> "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."

This finding is identical to the second conclusion reached by the Independent Panel; the factors mitigating against its validity have already been discussed in detail. The IRG went on to say:

> "Somewhat less probable is the concept that damaging seepage started at the contact of zone 1 (impervious core) material and the rock surface. The open fractures in the abutment foundation rock allowed direct access by reservoir water to the impervious core on the upstream side of the key trench. Any water flowing through the impervious core could exit into open fractures on the downstream side of the key trench."

The exact meaning of the above quote is not clear. Water "flowing through the impervious core" could include seepage at the interface between the key trench floor and the fill. Considering the relatively even surface of the floor and grout cap in the area where failure was initiated, the special compaction specified at this juncture, and statements by construction supervision staff that this aspect of the specifications was faithfully executed (which is borne out by construction photographs), piping along the fill/rock interface should not occur unless the hydraulic gradient was abnormally high. At El. 5200, approximately where failure is believed to have been initiated, the head at maximum reservoir level was 103 ft, corresponding to a maximum hydraulic gradient of approximately 3 to 4 across the base of the key trench. Many dams have withstood gradients of this order at the core/rock interface without incident.

Water flowing through the key trench fill from open joints on the upstream wall at full reservoir head to open joints in the downstream wall could, of course, start erosion into the downstream joints. However, excavation of the left abutment key trench disclosed no instance in which flow had progressed much more than half way across the key trench fill. Again, unless it can be shown that conditions in the right abutment key trench were different than at the left, it is unlikely that this mechanism initiated the failure. It may be argued that, eventually, seepage would occur through the key trench fill. Piping could then start at joints in the downstream wall and progressively erode a channel extending to the upstream face, which would allow further erosion to take place rapidly. This could be true, although it has been demonstrated (Wittke, 1984) that piping into open rock joints can be a slow process, and may have provided sufficient warning for remedial measures to prevent breaching of the dam. Nevertheless, we fully concur that failure to seal open joints in the rock and to provide filters to prevent piping from occurring was a serious design error at Teton Dam.

In the 1977 report, the IRG recommended additional investigations of the left abutment and embankment remnant. It was stated:

> "Physical conditions on the left abutment are very similar to those of the right abutment. An investigation of the embankment and embankment foundation contact surface will be made by a means that will permit visual inspection."
>
> "The investigation will be primarily to search for cracks in the remaining left embankment and to find evidences of erosion of the zone 1 [core] and the rock surface."
>
> "Further in situ stress investigations are planned."

To the credit of the Bureau, these recommendations were fully implemented. Over 880,000 yd^3 of embankment were carefully excavated and visually examined, in open excavation, test pits, trenches, adits, and borings. Additional hydrofracturing, in situ permeability, and field density and moisture tests were carried out; an instrument to monitor foundation rock rebound was installed; record samples (including cubic ft blocks) were obtained for further testing in the laboratory; and additional core drilling was done in the right and left abutment key trenches to explore further the condition of the grout curtain. It was probably the most comprehensive post-construction investigation ever made on a civil engineering structure, whether failed or unfailed. The results were documented in a second report by the IRG (IRG, 1980). The main findings were:

1. The floor of the left key trench was relatively smooth and free of open joints.

2. The bottom 12 to 18 inches of key trench fill upstream of the grout cap were nearly saturated. Fill in contact with the foundation downstream of the grout cap generally was not saturated, but several locations were found where upstream to downstream penetration of water had occurred across the grout cap. No locations were found where the nearly saturated fill extended across the full width of the key trench floor. [The assessments were based on subjective determination of moisture content and on measured water contents and calculated degrees of saturation. In general, it was difficult to distinguish a "wetting surface" visually.]

3. While grout from rock surface treatment was found in some of the joints in the left key trench walls, mostly in the downstream wall, many open joints were also encountered. One such joint in the downstream wall was nearly vertical, 3 ft high, and about 6 in wide.

4. In compliance with the specifications, fill material had been compacted into open joints in the trench walls. Material within joints at the upstream wall face became

wetted with reservoir water and was loose. [However, we are aware of no case where sloughed material was found penetrating more than a few inches into the main body of the fill.]

5. Joints in the left abutment rock outside of the key trench were generally open but no fill erosion into these joints was observed.

6. Nineteen core holes were pressure tested with water to determine the tightness of the grout curtain. Water losses indicated that the grout curtain was of adequate quality to control seepage losses, but it was not impervious enough to prohibit flow from occurring.

7. Three additional holes were drilled for hydraulic fracturing testing using air-foam drilling. Great care was exercised during drilling, placement of casing, and sealing of the test sections. In all three cases, the holes were filled with water to the top of the casing but no hydraulic fracturing was observed; at this time the water level in HF-11 (in the key trench) was 18 ft above the reservoir elevation at failure, while in holes HF-14 and HF-15 (in the center of the embankment) it was 30 ft above this reservoir level.

8. No erosion of fill was observed either above or below the grout cap.

9. No visible cracks, except those clearly associated with hydraulic fracture tests were observed in the fill. No piping of any kind was observed.

It is worthwhile to pause and remind ourselves that the primary purpose of this massive additional investigation was "to search for cracks in the remaining left embankment and to find evidences of erosion channels through the core or at the contact of the zone 1 and the rock surface" (IRG, 1977, p. 105). In fact, absolutely nothing of this nature was uncovered. However, on October 5, 1977, a surprising and heretofore unobserved phenomenon was discovered: a soil layer in zone 1 was noticed to be seeping water at Sta. 24+50, 150 ft upstream from the dam centerline and near El. 5113. There had been no awareness of extremely wet fill as the removal of material had proceeded through the elevation of the seepage zone between the embankment remnant and the abutment. During subsequent field investigations, numerous such seepage zones were encountered, which were termed "wet seams." The discovery of the wet seams on the left side of the embankment led to speculation that a similar seam on the right side may have been responsible for the failure of the dam. In view of this possibility, a careful study to determine the cause, character, location, and significance of the wet seams was immediately undertaken by the IRG. It is not feasible to present herein even a brief review of all the major findings; hence, we must content ourselves with a discussion only of those features which led us to propose our version of the most likely mechanism of failure.

The "Wet Seams"

The characteristics of the wet seams most relevant to this discussion are:

a. Geometry. The thickness of a wet seam never exceeded one loose lift (8 in $\pm$); usually, they were 4 in or less in thickness. What was initially thought to be a nearly continuous single wet seam was actually multiple wet seams, localized pockets, and discontinuous lenses. Association of wet seams with variations in the compaction process is inescapable.

b. Moisture/Density. The in situ water content ranged from 22.3 to 33.0 percent, average 28.0 percent, 9.4 percent above the average for all zone 1 fill. The in situ dry unit weight ranged from 82.3 to 96.5 lb/ft^3, average 90.3 lb/ft^3, 8.8 lb/ft^3 less than the average for all zone 1 fill. The Proctor optimum moisture content and dry unit weight were 3.1 percent above the average OMC (19.7 percent) and 5.3 lb/ft^3 below the average maximum density (101.1 lb/ft^3) for all zone 1 fill. These results show that:

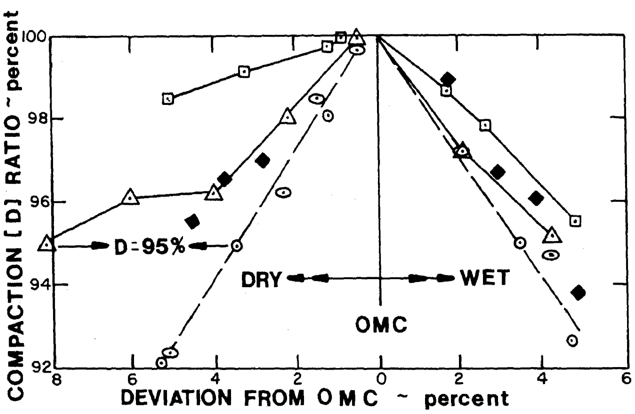
- In situ moisture contents are far above optimum, and the average is more than 5 percent wet of optimum, although the frequency distribution for zone 1 fill (IRG, 1980, p. 4-22) shows that virtually 100 percent of the fill was placed drier than 1 percent wet of optimum. Therefore, the high water contents must be due to post-placement infiltration.

- In situ dry unit weights and Proctor optimum densities are very low in comparison with the average of zone 1 fill. This suggests that the properties of the material in the wet seams are different from the average, but as the high water contents are due to post-placement infiltration, other layers with properties similar to those in the wet seams could exist that have not yet been infiltrated and, therefore, would not appear as wet seams. Thus, it is not necessary for a wet seam to have been in contact with the rock of the right abutment. What is relevant is the possibility that material similar to that in the wet seams could have been compacted in the right abutment key trench.

- If the compaction characteristics of the material in the wet seam were misjudged and thought to be more like the average of material in zone 1, the placement water content could actually be much drier than the 3.5 percent below OMC permitted by the specifications.

The result is the extreme likelihood that layers of silt compacted very dry of optimum were incorporated in the dam, at the left as well as the right abutment. This is clearly indicated in Fig. 2, which shows that samples of the Teton silt can be compacted to D-ratios >95 percent at water contents more than 8 percent dry of optimum, while the Nebraskan silts fall below D = 95 percent when the water content is 3.4 percent dry of optimum.

c. Permeability. Horizontal and vertical permeability tests were conducted on samples from hand-cut blocks taken from zone 1 fill. Fig. 3 shows the relation between average horizontal permeability and dry density obtained from these tests. Typically, permeabilities in the wet seams are an order of magnitude higher than outside the wet seams. The IRG attributed these large differences in permeability primarily to differences in dry density and not to other physical, chemical, or mineralogical differences. We interpret it to mean that the wet seam samples not only had lower densities but also were compacted initially well on the dry side of Proctor optimum moisture content.



SYMBOL	SAMPLE NO.	LOCATION	γ_d max [pcf]	OMC percent
—△—	51B-328	TETON DAM WET SEAM	97.0	22.0
—□—	51B-31	TETON DAM	92.7	23.7
—◆—	COMPOSITE	ZONE 1	96.3	21.6
—⊙—	60N-642	PEORIAN SILT NEBRASKA	105.2	18.5
—⊖—	TURNBULL 1942	NORTH PLATTE NEBRASKA	107.3	17.0

FIG. 2 COMPACTION RATIO VS. DEVIATION FROM OMC FOR TETON AND NEBRASKAN SILTS

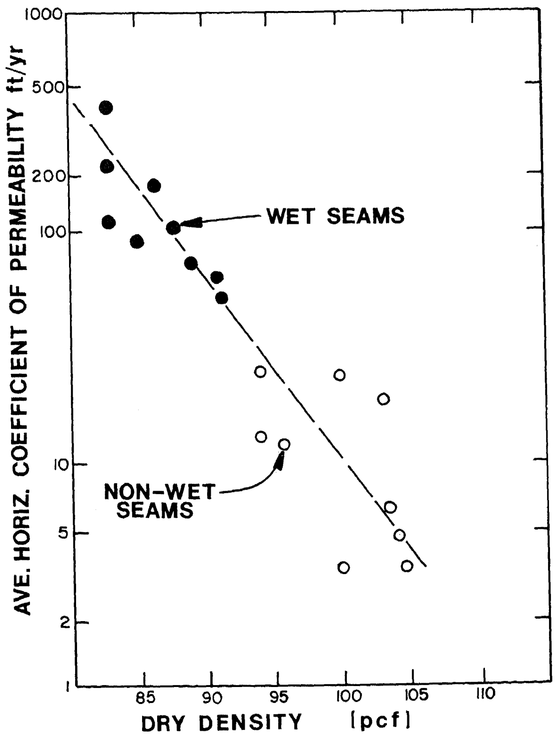


FIG. 3 AVERAGE HORIZONTAL COEFFICIENT OF PERMEABILITY VS. DRY DENSITY, BLOCK SAMPLES FROM LEFT ABUTMENT EXCAVATION

d. Collapse Potential. Three specimens from a wet seam sack sample were compacted to approximately 90 lb/ft^3 (93 percent of Proctor maximum) at moisture contents 0.4, 1.1, and 3.5 percent dry of Proctor OMC. After curing they were consolidated rapidly without wetting to 80 lb/in^2, the approximate preconsolidation pressure of block samples from the wet seams. The 80-lb/in^2 pressure was maintained for 24 hours before the samples were wetted. Fig. 4 shows the additional compression, or "collapse," on wetting. The 2.6 percent collapse was considered excessive and undesirable, as it was thought it would probably lead to cracking in the relatively brittle zone 1 fill.

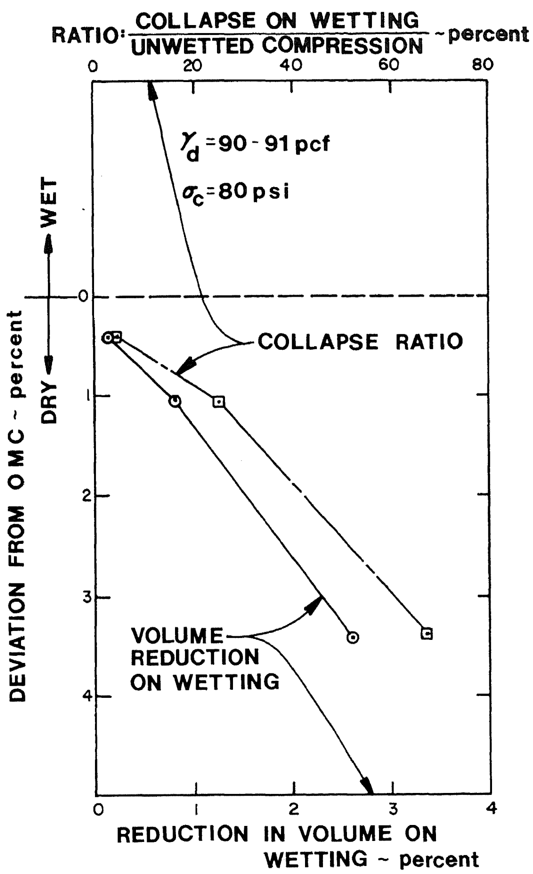


FIG. 4 PERCENT VOLUME REDUCTION AND COLLAPSE RATIO VS DEVIATION FROM OPTIMUM MOISTURE CONTENT, SAMPLE FROM "WET SEAM"

Conclusions of the IRG, 1980

In 1980, the IRG concluded that the discovered conditions in zone 1 fill supported the following physical modes by which piping could have been initiated:

> 1. "Seepage of reservoir water along either the fill/rock contact surface or near the top of the grout curtain in the right key trench, or
>
> 2. "Seepage of reservoir water through a low density, high permeability lens located within or adjacent to the right key trench."

These conclusions are drastically different from the findings stated in the 1977 report and is illustrative of the evolution in thinking regarding the primary mechanisms of failure as more data were developed and more time to study all aspects of the failure became available. Nevertheless, they went on to say:

> "No cracks associated with reservoir-induced hydraulic fracturing or differential settlement within the embankment were discovered. However, evaluation of hydraulic fracture test data indicates the possible occurrence of embankment stresses low enough for hydraulic fracturing to occur."

Arguments supporting our belief that neither conclusion 1, nor cracking due to hydraulic fracturing are likely mechanisms of failure have already been presented. Although we believed that the dam failed too rapidly for the mechanism described in conclusion 2 to have occurred, it "rang a bell" which ultimately led to our explanation of the failure mechanism.

Findings of Seed and Duncan (1982)

The retrospective review by Seed and Duncan (1982) included several important contributions, foremost among which was the review of additional hydro-fracturing studies conducted at the University of California (Jaworski, 1979; Jaworski, Duncan, and Seed, 1981). Among the "useful tentative hypotheses" proposed, relating to the occurrence of hydraulic fracturing were the following:

- "Hydraulic fracturing is promoted by, and may, in fact, require the presence of a discontinuity, such as a borehole, an existing crack, or loose soil adjacent to a rock joint, within which the water pressure can act to create tensile stresses through a wedging action in the soil."
- "Hydraulic separation can occur at an interface between soil and an adjacent dissimilar material such as concrete or rock as soon as the water pressure reaches the same magnitude as the normal stress across the interface" [assuming no adhesion between the soil and rock].
- [As tests showed that the rate at which the water pressure is increased, as well as the size of the zone within which stress redistribution and changes in water content occur, have a profound effect on the fracturing pressure.]
- ". . . thus, the pressures required to cause fracturing during reservoir filling may be different for a borehole test than the water pressure application on the core of a dam."

Seed and Duncan concluded:

> "As a result of these studies it was concluded that hydraulic fracturing could occur in the key trenches under the seepage and water pressure conditions existing at the time of failure. However, it could only occur in limited portions of the fill having the right combination of
>
> (1) open rock joints
> (2) soil type
> (3) location of joints in the key trench
> (4) outlet rock joints on the downstream face
> (5) in-situ stress conditions.
>
> This may explain why failure occurred near Station 15+00 and nowhere else."

It has already been pointed out that the rock joints in the left abutment key trench were, if anything, more open than on the right, and that the geometries of the two key trenches were essentially identical. No reason is given why the soil type in the right abutment key trench should be more susceptible to fracturing than in the left key trench. As no evidence of hydraulic fracturing due to reservoir seepage was found in the left abutment excavations, the argument that it caused failure at the right abutment is not supported by the existing data.

Seed and Duncan (1982) proposed three additional possibilities for the failure, all three of which required

> "water to flow, with accompanying erosion, from an upstream open joint [in the floor of the trench] along the base of the key trench, over the grout cap and into a downstream joint [in the floor of the trench] as illustrated in Fig. . . ."

As stated previously, we believe the probability that such open joints existed in the floor of the right key trench to be very remote. Seed and Duncan also offered a fourth potential mechanism of failure:

> "The remote possibility that a wet seam existed in the right abutment key trench permitting seepage directly through the seam and associated internal erosion."

This mechanism is similar to conclusion (2) in the 1980 IRG report, except that the IRG did not require that the low density-high permeability layer initially be a wet seam. This possibility will be discussed later.

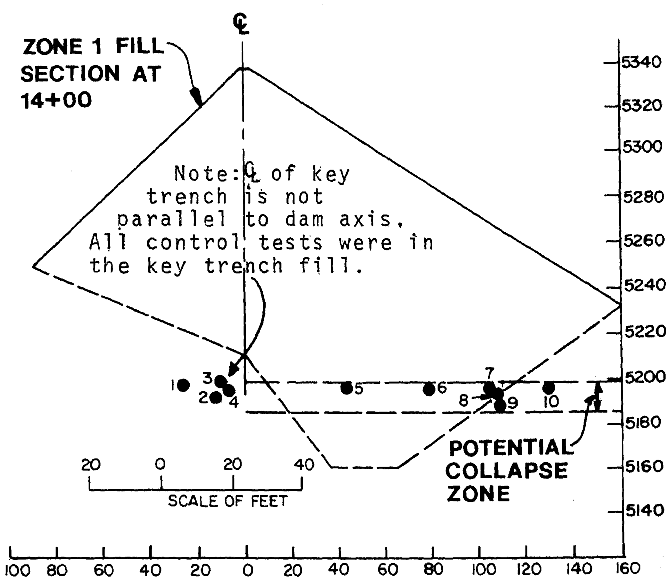
PROPOSED FAILURE MECHANISM

The basic requirements of a viable mechanism to explain the failure of Teton Dam are reiterated as follows:

1. A weakness with respect to seepage erosion must have existed in the right abutment and not at the left, and

2. The opportunity must exist to promote failure by seepage erosion in a matter of a few days after the reservoir reached an elevation sufficient to initiate the erosive action.

As soon as it was recognized that a high permeability layer could have been built completely across the right key trench fill and not across the left, it immediately became apparent that this could be checked using the records of compaction control tests. The data were computerized and printouts of the moisture/density test results, station by station, were obtained. Data at locations corresponding to the key trench fill, on both abutments, were then plotted for each 100-ft interval. At only one locale did the data suggest a continuous stratum of soil compacted on the dry side of optimum, which - notably - turned out to be 14+00, as illustrated in Fig. 5. We appreciate that the data are insufficient to reach a definitive conclusion; nevertheless, this remarkable result led us to believe there is a high probability that a continuous layer of high permeability was constructed across the right key trench and not across the left. However, as we have argued before, this is not by itself sufficient to explain the rapid demise of the dam because (1) time is required for seepage to transit the width of the key trench, and (2) due to the relatively modest initial exit gradient, soil would merely slough into an open joint on the downstream wall and piping would only work its way, slowly back to the upstream face. We concluded that the combination of these two effects would require much more time to fail the dam than the limited number of days (perhaps hours) in which the dam was breached. It seemed necessary for the flow rate into a downstream joint to be very high initially so that sloughed soil would immediately be washed out, thereby permitting a larger and larger channel to form rapidly. Only a preformed crack, or hydraulic fracturing, seemed to satisfy this requirement. The dilemma posed by this requirement remained unresolved for months until, one day, Leonards recalled earlier studies on the collapse of compacted clays (Leonards and Altschaeffl, 1971). The brief treatment of "collapse potential" in the IRG report (summarized in Fig. 4) came into focus, and the additional reduction in normal stress at the base of the key trench due to arching that would accompany the collapse was realized.



TEST NO	DEV. OMC %	γ_d pcf	TEST NO	DEV. OMC %	γ_d pcf
1	0.9dry	95.4	6	2.0dry	98.6
2	1.5dry	96.6	7	0.5dry	89.8
3	1.5dry	97.4	8	1.6dry	96.7
4	1.0wet	106.4	9	2.2dry	92.3
5	2.2dry	100.5	10	1.4wet	103.3

FIG. 5 COMPACTION CONTROL TESTS RESULTS BETWEEN STA.13+90 AND STA.14+40, AND EL.5184 AND EL. 5198

Thus,

SUBSIDENCE OR "COLLAPSE" OF A PERMEABLE DRY-SIDE COMPACTED LAYER SPANNING THE WIDTH OF THE KEY TRENCH ON THE RIGHT ABUTMENT, PERMITTED HYDRAULIC FRACTURING (OR SEPARATION) TO OCCUR IN THE KEY TRENCH FILL THEREBY ALLOWING FLOW FROM OPEN JOINTS IN THE UPSTREAM WALL (WITH ACCESS TO THE RESERVOIR) TO OPEN JOINTS IN THE DOWNSTREAM WALL,

is proposed as a failure mechanism that is fully supported by all the existing data.

ONLY THE COMBINATION OF A PERMEABLE LAYER COLLAPSING ON SATURATION, AND OPEN JOINTS IN THE WALLS OF THE KEY TRENCH WITH ACCESS TO THE RESERVOIR, COULD PROVIDE THE NECESSARY DISCONTINUITY AND SUFFICIENT REDUCTION IN STRESS ON HORIZONTAL PLANES TO INDUCE HYDRAULIC FRACTURING AND RESULT IN SO RAPID A FAILURE OF THE DAM.

We immediately decided to test the validity of our proposal with large-scale model studies in the laboratory but, alas, the necessary resources were not available to us. However, the experiments described below were conducted.

As in previous collapse potential tests, samples were compacted dry of optimum into consolidation rings. Compression of the samples was measured, without wetting, up to a pressure of 80 lb/in^2. The samples were then wetted; but, instead of allowing collapse to occur, the stress reduction necessary to maintain approximately constant volume was measured. Subsequently, collapse was allowed to occur. The results are shown in table 1. Thus, collapse of a dry-side compacted layer in the key trench fill at the right abutment would reduce vertical stresses by soil arching and promote hydraulic fracturing at the interface with the stiff adjacent fill. We believe that the interface between two soil layers with different permeabilities is a discontinuity along which hydraulic fracturing can occur, even if it is not initially a minor principal plane. This is evidenced by the fact that hydraulic fracturing due to drilling at high air pressures caused only horizontal cracks to develop at least 25 ft - and possibly more than 50 feet - from the drill hole.*

LESSONS LEARNED

Following their retrospective review, Seed and Duncan (1982) listed 11 lessons to be learned from the Teton Dam failure. We basically agree with all of them, especially with the principle of "multiple lines of defense" advocated by Karl Terzaghi and Arthur Casagrande. Failure to adopt this approach, especially for the conditions extant at Teton, was a serious design error. The principle also underscores the importance of understanding the actual mechanisms of failure, which provide the only rational means of assessing benefit/cost ratios of different proposed "lines of defense." Moreover, they may avoid building in features that, for example, may be conducive to hydraulic fracturing, or to other undesirable conditions, even if measures to prevent seepage erosion are provided for.

To the 11 lessons cited by Seed and Duncan, we add the following:

I. Samples of Teton silt can be compacted to compaction ratios greater than 95 percent at water contents more than 8 percent dry of their Proctor optimum water contents; hence, it would be particularly difficult to judge the acceptability of field compaction by visual inspection. Therefore, the frequency of compaction control tests should have been higher than usual. In the future, the "norms" for frequency of compaction control tests should not be applied indiscriminately to all soils.

II. Considering the seriousness of the potential damage that could result, the difficulty of recognizing by visual inspection that something is amiss, and the inevitable occasional departures from the specifications associated with large construction operations, specifying dry side compaction (as low as 3.5 percent dry of OMC at Teton) should be viewed with great caution.

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* After our tests had been initiated, we became aware of a draft report (Johnson and Palmerton, 1977) reviewing Appendix D of the Independent Panel's 1976 Report. Therein it was stated:

> "If an abnormally dry layer or layers of core material extended across the core trench, the material would consolidate, if saturated, much more than surrounding material. This may have resulted in arching at higher elevations and possibly in horizontal channels. The most likely failure process would involve also hydraulic fracturing, since hydraulic fracturing and local volume decrease would each promote the other."

Apart from the requirement that the collapsing layer initially must have a much higher permeability than the adjacent fill, to provide the necessary discontinuity for hydraulic fracturing to occur, these statements are similar to the conclusions we reached following more than 2 years of painstaking efforts.

Table 1. Results of "Collapse" tests

	Peorian silt, Nebraska* sample 60N-642	Teton Dam "wet seam" sample 51B-328
Inherent Characteristics		
Passing No. 200 mesh sieve, percent	100.0	75.0
Plasticity index, percent	10.0	N.P.*
Proctor Test		
Max. dry density, lb/ft^3	105.2	97.0
Opt. water content, percent	18.5	22.0
Compaction Conditions		
Dry density, lb/ft^3	97.0	92.5
Water content, percent	13.5	17.0
Compaction (D) ratio, percent	92.2	95.4
Axial Strain		
Unwetted compression 0-80 lb/in^2, percent	3.3	2.9
Compression on wetting at 80 $lb/^2$, percent	3.9	0.3
Wetting to unwetted compression, percent	118.0	11.0
"Restrained" Test		
Stress reduction on wetting, percent	39.0	15.4
Axial strain during "restraint," percent**	1.4	0.2
"Restraint" strain/ wetting strain, percent	36.0	67.0

* A more plastic sample from the wet seams was not available for testing.
** Due to tolerances in assembly linkages.

III. Variations in the properties of compacted embankments are generally larger than those commonly anticipated. It is essential that steps be taken in design and construction practices not so much to reduce these variations greatly - which would be very expensive - but to identify potential problems and insure that layers with adverse properties do not have sufficient continuity to be troublesome. At Teton it was most likely a continuous layer of silt compacted in the key trench well dry of optimum; at Lake Shelbyville, it was established that a thin weak seam of more highly plastic clay was largely responsible for an upstream slope failure shortly after the dam was topped out (Humphrey and Leonards, 1984).

IV. In spite of numerous past studies of the properties of compacted fine-grained soils, there is still much to learn about (a) the mechanics of the field compaction process, including the manner in which sheepsfoot rollers compact silts and silty clays dry of optimum; (b) the differences in compacted properties - especially optimum moisture content, permeability, and compressibility - between field and laboratory compacted soils; and (c) the mechanics of hydraulic fracturing. Hydraulic fracturing is, potentially, so damaging a process that it clearly merits further intensive study.

V. A methodology for investigating failures (Leonards, 1979) was applied to Teton Dam. The search for a mechanism of failure continued until one was found compatible with <u>all</u> the known facts, which, in turn, suggested specific investigations that otherwise would not have been thought of. This resulted in: (a) the discovery that Teton silt had unusual compaction characteristics dry of optimum (Fig. 2); (b) a directed search of compaction control records to identify the potential existence of continuous layers in the key trenches compacted dry of optimum (Fig 5); and (c) special tests to measure the relief in stress associated with collapse on wetting (table 1).

VI. Each successive study of the Teton Dam failure contributed to our understanding of earth dam behavior. This demonstrates that there is much to be gained from continued retrospective analyses of past failures, in the light of current knowledge. It has been shown that there is no reason to be complacent about the lessons learned from past investigations of failures (Leonards, 1982). A National Agency possessing (among other attributes) the expertise and resources for such continued studies would be of great benefit to the civil engineering profession and to the public at large.

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