The Teton Dam Failure — A Retrospective Review

La Rupture du Barrage de Téton — Un Examen Retrospectif

H.B. SEED and
J.M. DUNCAN Professors of Civil Engineering, University of California, Berkeley, CA, USA

SYNOPSIS This paper reviews the events leading to the failure of Teton Dam in Idaho on June 5, 1976 and the investigations, conducted over a period of several years, to determine the cause of the failure. Conclusions are presented regarding the probable trigger mechanisms which initiated the failure, the mechanics of failure and the significant lessons concerning earth dam design and construction resulting from the investigations.

INTRODUCTION

The failure of Teton Dam in Idaho in June, 1976, during the first filling of the reservoir, was a significant event for geotechnical engineers concerned with the design and construction of earth dams--simply because no dam of such a height (approximately 300 ft above stream bed) had previously failed. Thus the failure was both dramatic and of considerable import.

At the International Conference in Tokyo in 1977 a report on this event was presented and a detailed account of the failure has been published in the Case History Volume of the Proceedings of that Conference (Chadwick, 1981). During the intervening period a number of additional studies have been made to clarify the probable cause of this catastrophic event. Some of these have received passing mention in magazines and journals but it seems appropriate, now that all studies have essentially drawn to a close, to review the entire event and the results of all the investigations in an attempt to put them in their proper perspective.

Following the failure two investigation groups were formed.

1. The Independent Panel to Review the Cause of Failure, consisting of a number of experts in dam engineering including Arthur Casagrande, Ralph Peck, Wallace Chadwick, and others. The report of this Panel was published in December, 1976. One of the writers had the privilege of serving on this Panel and the views expressed in the following pages are undoubtedly influenced by observations from this vantage point.

and

2. A Panel of leading engineers and geologists drawn from Government agencies since Teton Dam was designed by a Government agency. This Panel, designated the Interior Review Group (IRG), continued its studies long after the Independent Panel ceased to exist.

The following review is based on the investigations and findings of both of these Panels, but the conclusions presented are those of the writers. It is appropriate first to give a brief review of significant features of the design and construction of the dam which undoubtedly had some bearing on the possible cause of the failure.

SITE CONDITIONS AND DAM DESIGN

The Teton Dam was located in a steep-walled canyon cut by the Teton River into a volcanic plateau known as the Rexburg Bench. A cross-section of the canyon approximately along the axis of the dam is shown in Fig. 1. The walls of the canyon consist of later Tertiary rhyolite welded-tuff which is strongly jointed, with joint widths varying at different elevations typically between 1/4 to 3 inches but with occasional joints up to 12 inches wide. Alluvium has been deposited in the river channel to a depth of about 100 ft and the high lands near the ends of the dam are covered with an aeolian silt deposit up to about 30 ft thick. The primary features of the site are the extensive joint system in the rhyolite-tuff which makes it extremely permeable and the abundance of the wind-blown silt deposit which led the designers to use substantial quantities of this material in the dam cross-section.

Extensive site exploration work was performed prior to construction. Percolation tests in the rock showed it to be capable of transmitting volumes of water over 100 gallons/minute. Pump-in tests in five drill holes showed similar results. In one hole, 280 ft deep, pumping water at the rate of 440 gpm for a period of two weeks only raised the water level in the hole by 185 ft but the effects were observed in holes 1000 ft away. These investigations confirmed the presence of an extensive interconnecting system of joints which made the rock extremely permeable and indicated the need to seal the joints in order to reduce the leakage to acceptable quantities.

In order to investigate the possibility of sealing the upper foundation rock by grouting, an extensive pilot grouting program was conducted

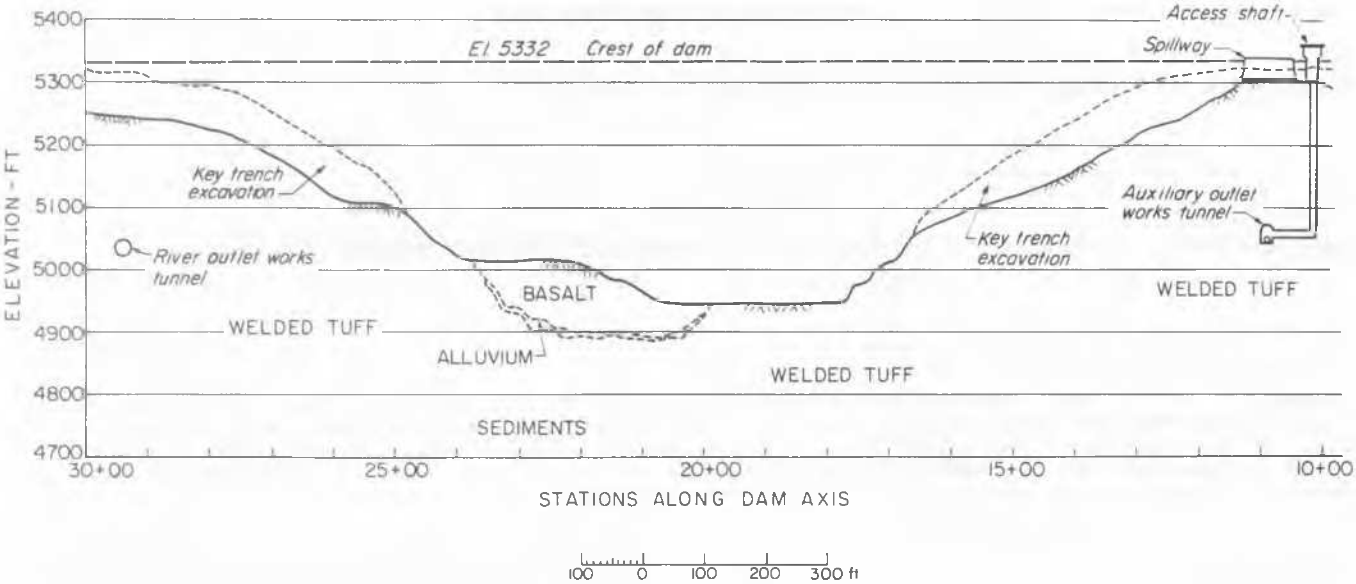

Fig. 1 Profile Along Axis of Dam

on the left abutment. Twenty-three holes were drilled, grouted and pressure-tested. There were significant takes in several holes. In fact the amount of grout injected in two holes exceeded the amount estimated for the whole program. These two holes took 15,700 sacks of cement and 17,800 cu ft of sand. The grout curtain test holes also showed exceptionally high takes at depths less than 70 ft with some grout travelling as much as 300 ft downstream. By thickening the grout using cement-sand mixes and calcium chloride the leaks tended to seal. However one persistent leak between 30 and 70 ft depths could not be filled to refusal.

Subsequently the area where the pilot-grouting program had been performed was core-drilled and water-pressure tested. Most of the test intervals showed little water loss.

On the basis of this test program, it was concluded that it would be more economical to remove the top 70 ft of rock in the abutments above El. 5100 rather than attempt to grout in this zone, leading to the subsequent adoption of a design incorporating a 70 ft deep key trench to prevent seepage.

Tests on the windblown silt deposit showed it to have good strength characteristics and low permeability (of the order of 5×10^{-6} cm/sec) but to be erodible and brittle. In order to minimize damage to the environment however, a decision was made to use as much as possible of the silt in the embankment section with a corresponding reduction in sand and gravel requirements.

EMBANKMENT DESIGN

On the basis of the site exploration program, the final design of the embankment had the configuration shown in Figs. 2 and 3. A wide core zone of the aeolian silt (upstream slope 1 on 1-1/2 and downstream slope 1 on 1) was supported by upstream and downstream shells consisting mainly of sand, gravel and cobbles. In the main section of the dam, the impervious core was extended through the foundation alluvium by means of a 100 ft deep cut-off trench backfilled with the silt. On the abutments above El. 5100, a similar section was adopted but key trenches with a base width of 30 ft and side slopes of 1/2 on 1 were excavated through the upper 70 ft of permeable rock and backfilled with the silty material used in the core of the dam.

Downstream of the core was a drainage zone of selected sand and gravels. However no transition zone was provided between the core and the sand and gravel, nor between the impervious core and the river bed alluvium or between the key trench fill and the rock walls on the downstream side of the key trench. However the core material in the key trench was placed directly against the rock using special compaction of a 2 ft wide zone of core material placed at a water content above optimum. Compaction of this zone was by hand-operated compactors or rubber-tired equipment.

In addition, the design required that joints encountered in the bottom of the key trench be treated by cleaning and low-pressure grouting. A grout curtain was also installed along the full length of the dam, some holes extending to depths of 300 ft. Grout holes were along a single line with primary holes 10 ft apart, and split spacing where the primary holes did not indicate a tight curtain. However lines of barrier holes, intended to prevent excessive flow of grout from the main grout curtain, were installed on 20 ft centers 10 ft upstream and downstream of the main grout curtain. It was not required that either the upstream or downstream rows of holes should form tight curtains.

To help prevent seepage, the key trenches and grout curtain were continued well beyond the ends

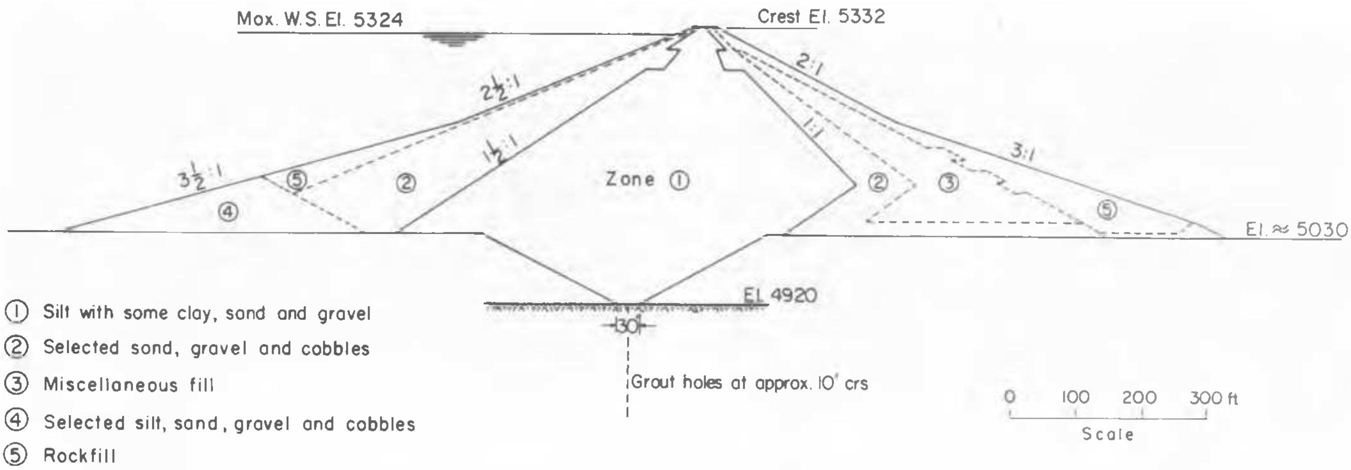

Fig. 2 Cross-Section Through Center Portion of Embankment Founded on Alluvium

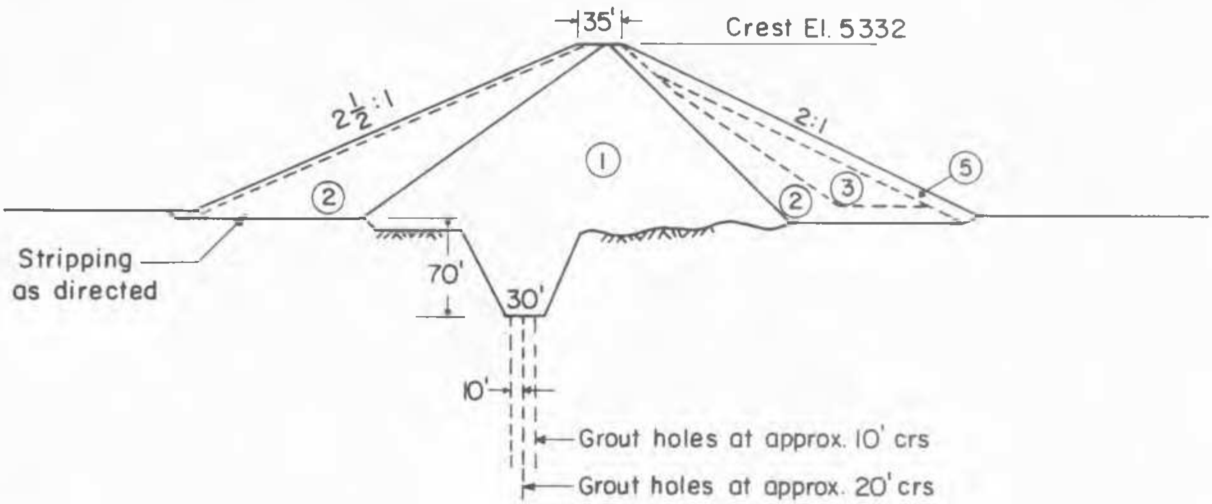

Fig. 3 Typical Cross-Section Over Abutment Sections Founded on Jointed Rhyolite

of the embankment, the curtain extending 1000 ft into the right abutment and 500 ft into the left abutment.

Thus, as noted in the report of the Independent Panel, "The final design depended for seepage control almost exclusively on the impervious core, the key trench backfill and on the grout curtain...the only downstream defense against cracking in the impervious fill or against concentrated leakage through it was the drainage zone and this did not extend into the key trenches." In the key trenches the silt backfill was in direct contact with the jointed rock.

No instrumentation other than settlement monuments was provided to monitor the performance of the embankment.

RESERVOIR FILLING

Reservoir filling began in November 1965, and the water level began to rise rapidly during the Spring of 1966. It was intended that the filling rate would be restricted to one foot per day, but a heavier than expected spring run-off from the watershed together with a delay in completing the river outlet works led to a much higher rate of filling which during May 1966 reached about four feet per day. By June 5, the day of the failure, the water level stood at El. 5302, just 3 ft below spillway crest elevation and 30 ft below the embankment crest.

THE FAILURE

On June 3, two days before failure occurred, some small springs, flowing in total at about 100 gpm, were observed at riverbed level about 1500 ft downstream from the embankment. On June 4, some additional springs with a flow of about 20 gpm had developed about 400 ft from the downstream toe. An inspection of the upstream and downstream slopes of the embankment at about 9:00 pm that night showed no unusual condition however.

On June 5 at about 7:00 am, when the first workers reached the site, water was observed to be flowing from the downstream face of the embankment about 130 ft below the crest (at about El. 5200), the flow of about 2 cu ft per second coming from a point near the junction of the embankment and the abutment at about Station 14+00 on the right abutment. At about the same time a flow of about 25 cu ft per second was observed emerging from the talus near the toe of the embankment. The water in this latter flow was clearly dirty. In the next three hours the rate of flow from the higher elevation gradually increased to about 15 cu ft per second and at about 10:30 am, one eyewitness reports a loud burst and a coincident movement of the seepage to a point about 15 ft in from the abutment. From this time onwards the seepage increased rapidly accompanied by progressive upward erosion; at 11:20 am the eroded hole in the dam was so large that bulldozers sent to fill the hole sank into the flow, and at about 11:55 am the dam crest was breached and a complete failure occurred. The resulting flooding downstream caused the loss of 14 lives and about 400 million dollars in damage. A view of the embankment after failure is shown in Fig. 4.

Fig. 4 View of Teton Dam After Failure

INVESTIGATIONS BY THE INDEPENDENT PANEL (1976)

The flow of water through the gap in the embankment led to erosion of both the embankment and a portion of the underlying rock. A cross-section through the embankment showing the extent of erosion is shown in Fig. 5.

In the months after the failure the Independent Panel initiated a number of studies to help determine the probable cause of failure. A number of hypotheses were developed and investigated. To aid in these studies, the soil remaining in the unfailed part of the key trench on the right abutment, see Fig. 9, was carefully removed and the walls and base of the key trench exposed. Joints in the rock at the walls and base could be clearly seen and two possible mechanisms of failure emerged:

1. Seepage Under the Grout Cap

The first of these was that seepage under the grout cap in unsealed joints in the rock could have led to erosion along the base of the trench and thereby to a piping failure through the key trench fill. The conceptual mechanism for such erosion is illustrated in Fig. 6.

To explore this possibility tests were made to investigate the transmissibility of joints on the bottom of the trench near the point where failure had occurred. For this purpose water was ponded over selected joints on the upstream side of the grout cap and observations were made for resulting flows on the downstream side and the rate of fall of the pond level.

At Station 13+90, for example, a two inch wide vertical joint was observed striking South 68° West across the alignment and beneath the grout cap. A small pond on the upstream side of this joint showed a flow rate of 28 gpm but the flow was limited by the capacity of the hose used for filling the pond. Such tests revealed a number of joints between Stations 13+00 and 13+90 through which water could pass freely beneath the grout cap. Accordingly this hypothesis of failure was considered plausible.

2. Piping through Cracks Caused by Hydraulic Fracturing or Differential Settlement

Detailed investigations were also made of the possibility that a piping failure could have been caused by seepage through cracks in the key trench fill caused by hydraulic fracturing or differential settlement.

For this purpose hydro-fracturing tests were performed in drill holes made into the unfailed portion of the embankment to determine the water pressures required to cause fracturing. At the same time finite element analyses were made to determine the stress distribution in the embankment. By comparing the results, appropriate stress-strain parameters were determined which would correctly predict the results of the hydro-fracturing tests. These parameters were then used to evaluate the probable stress distribution in the section where failure occurred and thereby assess the possibility of hydraulic fracturing occurring in the core of the dam due to water pressure on the upstream face.

The possibility of arching over the key trench with a corresponding reduction in stress on the key trench fill is clearly evident from the geometry of the section shown in Fig. 3. A calculated stress distribution in the embankment and in the key trench at Station 15+00 before wetting of the fill is shown in Fig. 7. Vertical

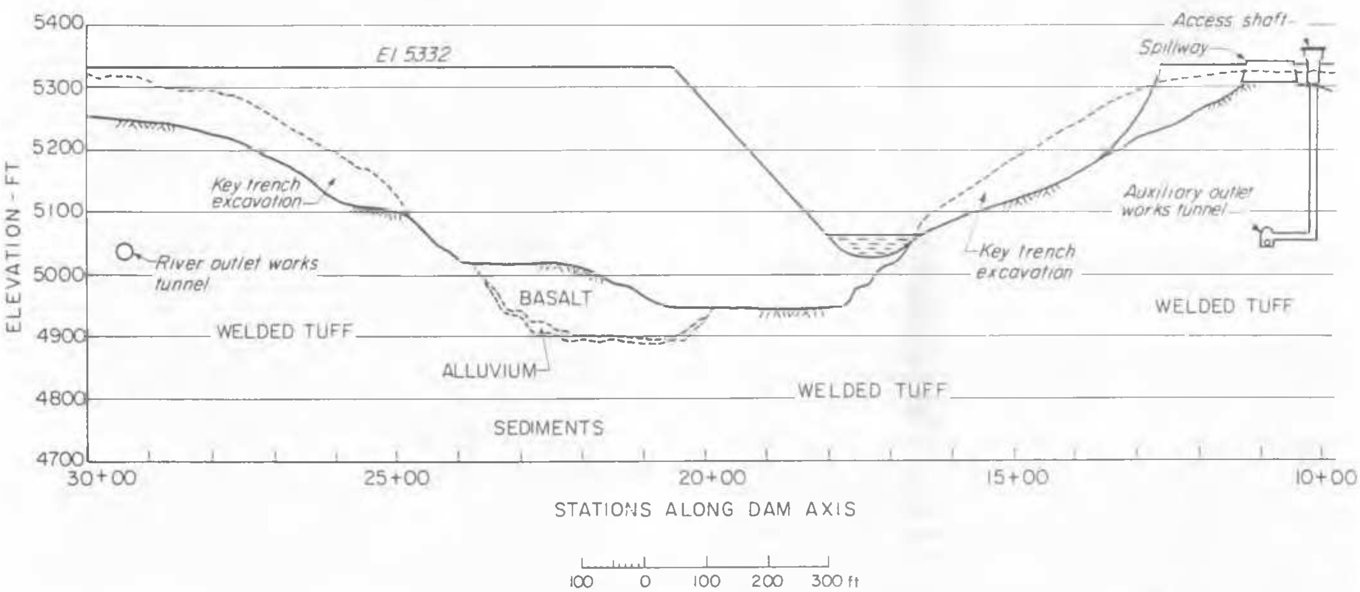

Fig. 5 Profile Along Axis of Dam After Failure

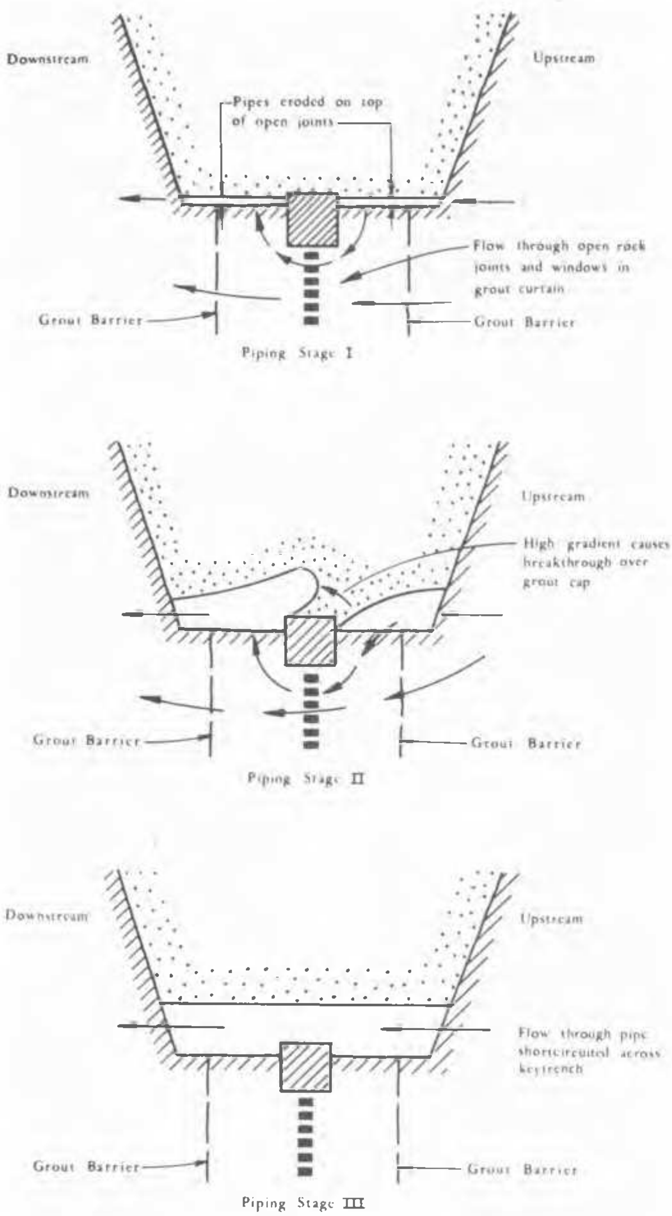

Fig. 6 Conceptual Mechanism of Failure Due to Seepage Under Grout Cap

pressures are shown as a proportion of the overburden pressure and the marked reduction in stress in the key trench is clearly apparent.

Finally comparisons were made of the computed values of transverse stress in the key trench fill with the water pressures developed on the upstream side of the fill. It was considered that fracturing might occur if the water pressure exceeded the sum of the transverse normal stress and the tensile strength of the soil. A comparison of these stresses at Station 13+70 is shown in Fig. 8. The soil zones where fracturing could develop on the basis of the foregoing concept are shown shaded in the figure.

From such analyses it was concluded that hydraulic fracturing could possibly have occurred in the range of Stations 13+70 to 15+00 but it was not likely to develop elsewhere on the right abutment. Since this range of stations coincided with position of the seepage and erosion failure of the dam, the possibility that the failure could have been caused by this mechanism was considered plausible.

It was noted that the key trench fill tended to decrease in volume when wetted under sustained pressure. Since differential wetting would occur in the key trench, differential settlements leading to cracking and reduced stresses in the wetted portions of the fill could also have developed.

Conclusions

From these and other studies which eliminated a number of possible mechanisms of failure, the Independent Panel concluded in its Report of December, 1976, that the two triggering mechanisms most likely to have led to the failure were:

1. "...the flow of water against 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 also concluded:

> "In either case, leakage occurring through the key trench ultimately initiated further erosion along the downstream contact of the core and the abutment rock. Since the core material was both easily erodible and strong, any erosion channels in the core, along the contact with the rock, readily developed into large tunnels or pipes before becoming visible along the downstream parts of the dam.
>
> "It should be noted that this description of the failure mechanism does not provide a final answer to the specific cause of failure of Teton Dam. Clearly many aspects of the site and the embankment design contributed to the failure, but because the failed section was carried away by the flood waters, it will probably never be possible to resolve whether the primary cause of leakage in the vicinity of Sta. 14+00 was due to imperfect grouting of the rock below the grout cap, or cracking in the key trench fill, or possibly both. There is evidence to support both points of view. Nevertheless, while the specific cause may be impossible to establish, the narrowing of the possibilities to these two aspects of design and construction is likely to serve as an important but tragic lesson in the design and construction of future projects of this type."

Soil Properties

γ (kcf)	ϕ	$\Delta\phi$	C (tsf)	K	K_{ur}	n	R_f	G	F	D	σ_{3T}	β
0.117	31°	0	1.65	470	750	0.12	0.79	0.35	0.10	4.0	0.0	0.003

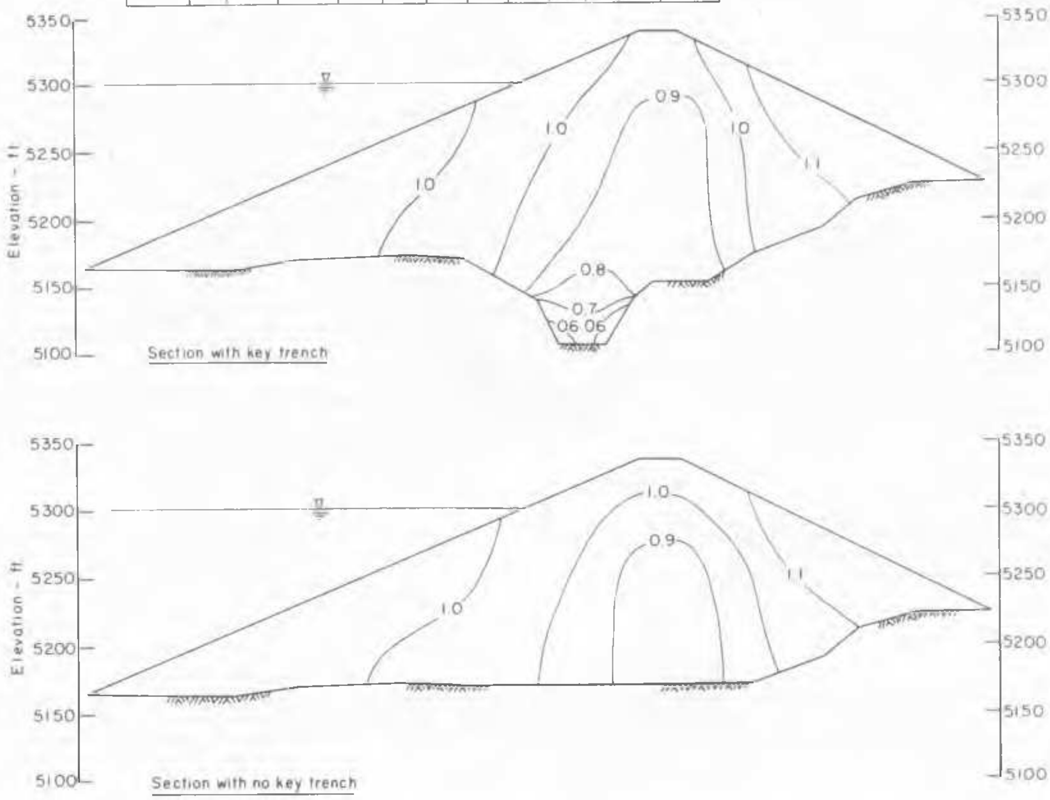

Fig. 7 Computed Values of Vertical Stress Expressed as a Proportion of Overburden Pressure - Station 15+00 - Before Wetting

INTERIM REPORT OF THE INTERIOR REVIEW GROUP (1977)

The Interior Review Group (IRG) which conducted its own studies but shared information with the Independent Panel issued an interim report in 1977. The primary conclusions of this report concerned the mechanism of failure and stated:

> "Teton Dam was constructed as specified and failed as a result of inadequate protection of the Zone 1 impervious core material from internal erosion. The most probable physical mode of failure was cracking of Zone 1 material that allowed the initiation of erosion; however the erosion could have been initiated by piping at the contact of the Zone 1 and the rock surface."

However the IRG went on to recommend additional investigations as follows:

1. To further test the grout curtain.
2. To excavate the left remnant of the dam to allow inspection of the embankment-foundation contact surface and to search for cracks in the remaining embankment and for evidence of erosion channels.

and 3. To perform finite element studies of the left abutment and supporting study of the relevant parameters.

These additional studies were conducted during 1977-1979 and the results are described in the following section.

ADDITIONAL STUDIES DURING 1977-79

1. Excavation of Embankment on Right Abutment

(a) Discovery of Wet Seams

During the summer of 1977 the embankment fill overlying the left abutment key trench was excavated and careful observations were made of the conditions of the fill. The extent of this excavation is shown in Fig. 9. No findings of major significance were noted until the excavation had almost reached the base of the key trench, when a thin zone of soil of very high water content was encountered. A few hours after it was discovered, water was found to be seeping from the exposed face of this zone. The zone was termed a "wet seam." The investigations of the wet seam and other studies are described in the Final Report of the Interior Review Group, issued in January, 1980.

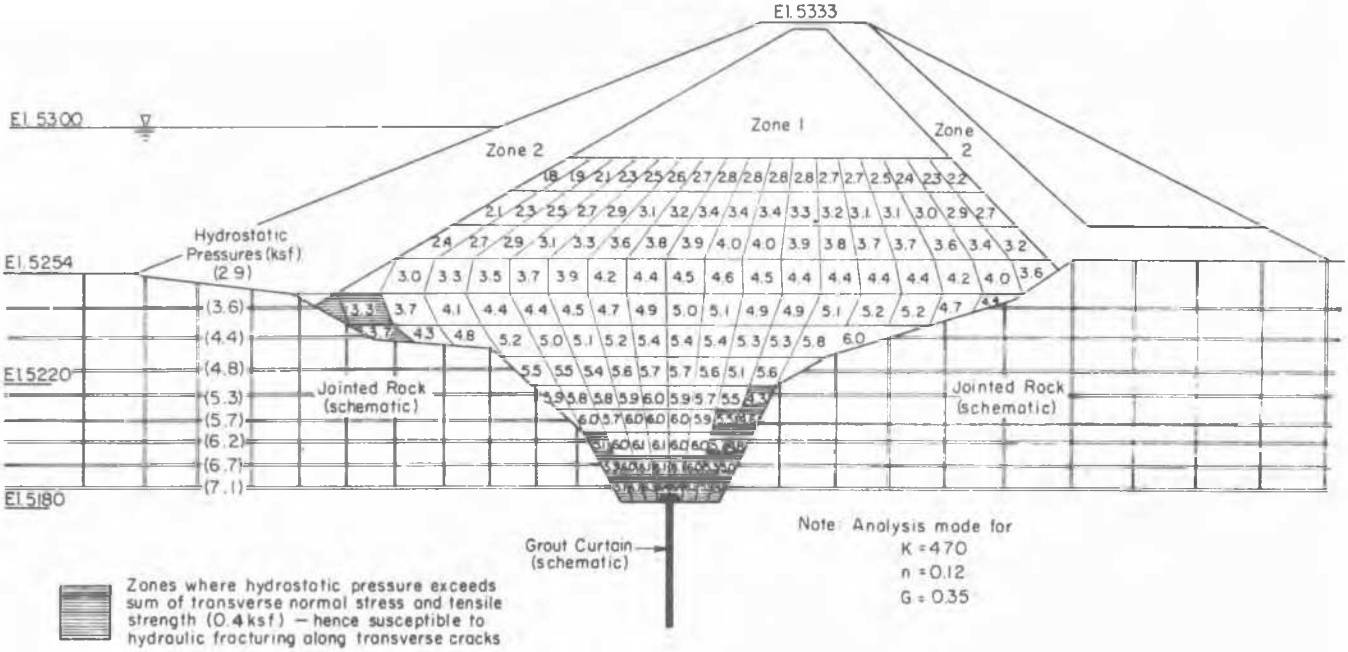

Fig. 8 Computed Values of Normal Stress on Transverse Section in ksf Sta. 13+70 and Comparison with Upstream Hydrostatic Pressures

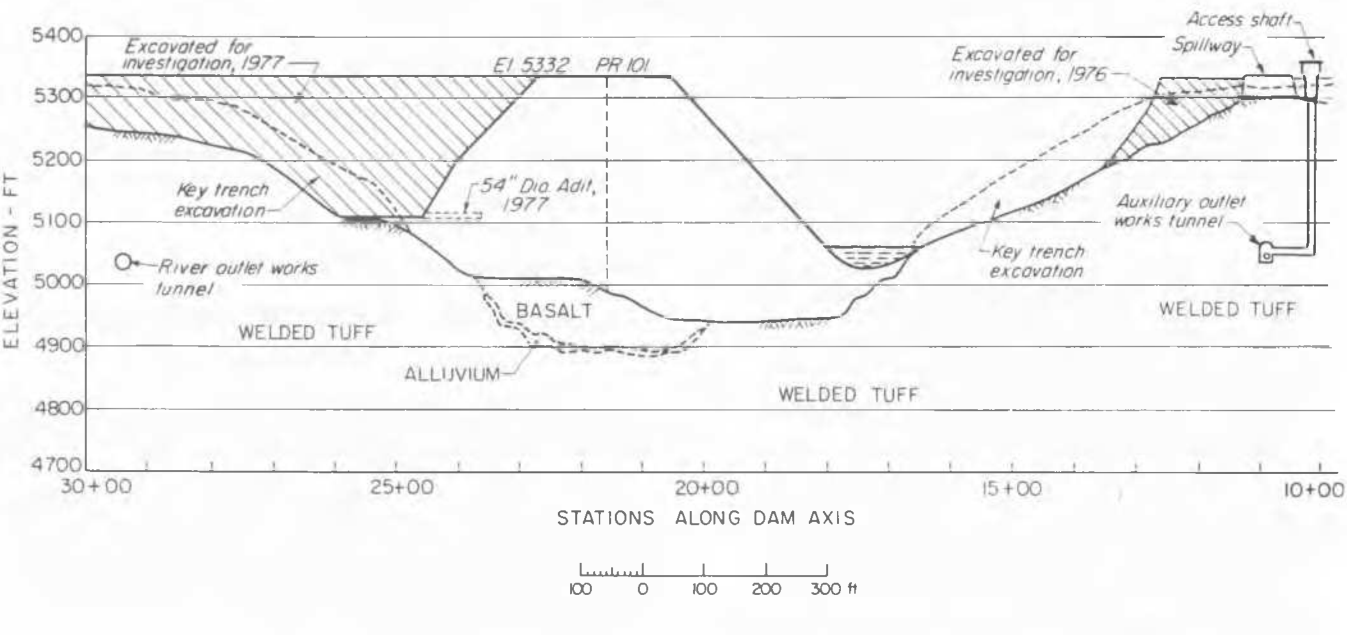

Fig. 9 Excavation of Embankment on Left Abutment in 1977

The initial wet seam discovered during the excavation of soil above the left abutment varied in thickness from 3 to 8 inches but as the investigations progressed and the extent was determined by means of borings in the remaining embankment, other wet seams were found. Each seam was usually less than 4 inches thick but in some areas, multiple seams, one above the other, were found. For example, in Boring No. PR-107 near the left abutment, such seams were found at Elevations 5112, 5115, 5117.5 and 5119, covering a depth of about 7 ft, while in Boring PR-108 near the center of the embankment, a single wet seam was found with a depth of about 5 inches. The total depth of the zone of wet seams never exceeded 12 feet at any location and it tended to become thinner as it approached the center portion of the embankment, the outermost borings (towards the center of the dam) showing no conclusive wet seams. The position of the main wet seam is shown in Fig. 10.

The total area covered by the major wet seam between Els. 5112 and 5139 was approximately 270,000 sq ft (600' x 450'). However other smaller seams were located at Els. 5050, 5070 and 5184.

Typically the soil comprising a wet seam was saturated silt with about 85% of the particles finer than 0.08 mm and an average dry density of about 90.3 lb/cu ft. For comparison the average dry density of the soil in the central impervious core was about 100 lb/cu ft. The Standard Proctor maximum density of the wet seam soil was only about 96 pcf--about 10 pcf lower than that expected to be placed in the core of the dam. Thus required placement dry densities were only about 90 pcf. The presence of such material, with low density, comparatively high permeability and capacity for moisture absorption, in the borrow area was not identified during design, nor was it revealed by routine embankment control procedures during construction. The average temperature of the wet seam soils in 1977 was found to be about 1° Celsius lower than that of the surrounding soil.

The discovery of this extensive wet seam on the left side of the embankment immediately led to speculation that a similar seam on the right side of the embankment 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 seam was clearly warranted and such an investigation was immediately undertaken by the Federal Government Interior Review Group.

Important findings of this group (IRG Report, 1980) were as follows:

1. The location of the main wet seam was essentially parallel to and just above the 1974-75 winter shut-down surface (see Fig. 10). This finding seems to eliminate frost action in the soil placed in 1974 as a possible cause of the wet seam.

2. The soil in the embankment where the main wet seam was found to exist was placed in the period April 29 to May 29, 1975. During this period of construction "there were two extended periods of shut-down due to wet weather (May 6-11 and May 22-26)....Also construction inspectors reported either snow or rain occurring <u>during construction</u> on May 5, 6, 19 and 21."

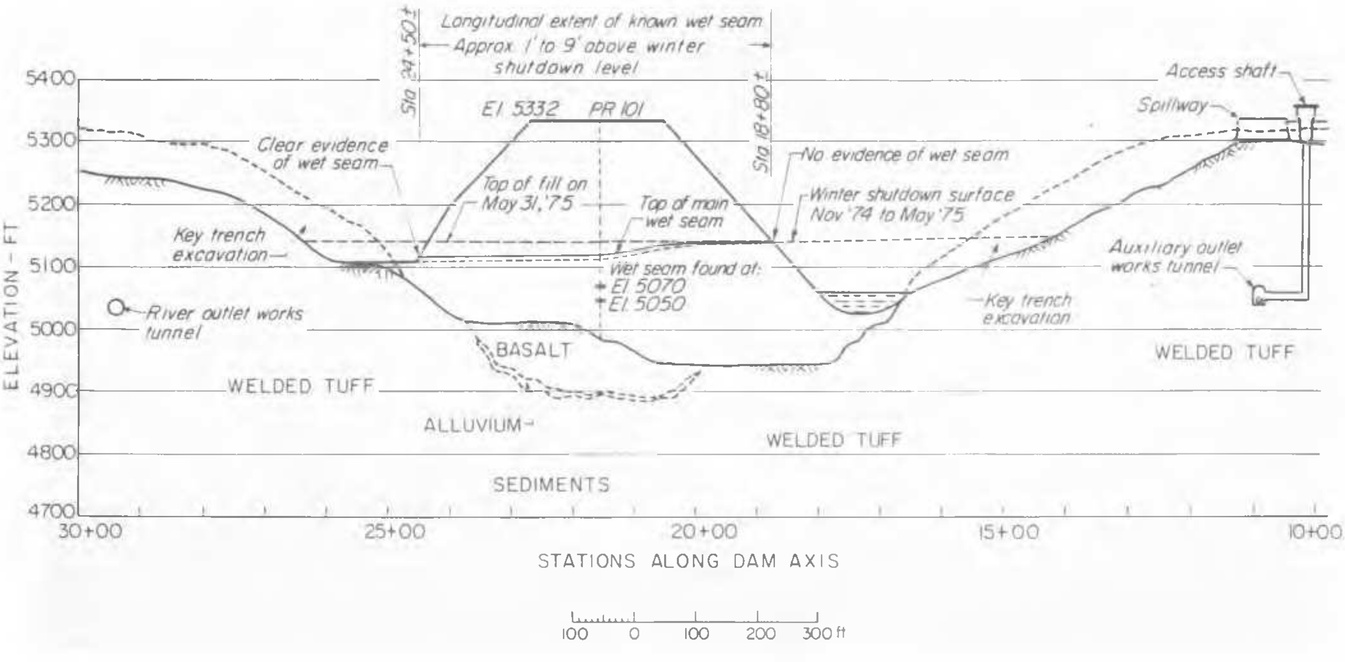

Fig. 10 Zone of Main Wet Seam Between Els. 5112 and 5139

3. "The fill surface was wet on several days, particularly May 1, 6, 7, 19 and 21.... Ponding of water on the surface was reported during each shut-down period....Since the embankment fill was higher near the right abutment than the left abutment during the 1974-75 winter shut-down, there was a water ponding problem on the left side embankment during periods of precipitation. This problem existed until late May, 1975." A photograph of the water ponded on the fill near the left abutment taken on May 7, 1975 is shown in Fig. 11.

   "There was a significant volume of rain and snow during May, 1975 and the infiltration capacity of Zone 1 (core) material is greater than ordinarily anticipated. The amount of removal of wet embankment surface material was minimal following the periods of construction shut-down due to wet weather. The time spent to prepare the wet surface for new fill placement was relatively brief."

4. "The wet seams encountered during the investigation can be associated with periods of precipitation. This is the case in May 1975 and also late July and early August 1974." It is also true for a wet seam found to be placed in July 1975 leading to the results shown in Table 1.

5. "Nearly all the wet seams encountered were well above or well below any elevation of embankment surface that could have been affected by frost action...." Analytical studies of frost penetration based on climatological data indicate that "complete thaw probably occurred before May 1, 1975, the first day Zone 1 embankment was placed."

6. There is no evidence of wet fill being brought to the site and thereby serving as a source of the wet seam material. In fact no period of fill placement experienced construction equipment trafficability problems comparable to those experienced during

Fig. 11 Photograph of Water Ponded on Fill on Left Side of Embankment on May 7, 1975

TABLE 1

Elevation of Wet Seam	Dates of Soil Placement	Dates of Significant Rainfall
≈5050	Late July 1974	Late July 1974
≈5070	Early August 1974	Early August 1974
5112 to 5140	April 29 to May 30	May 6-11; May 22-26, 1975
≈5184	July 11-21	July 14-16, 1975

excavation of the wet areas in the left part of the embankment. Thus the high water content of the soil in the wet seams must have developed after fill placement either by

(1) rain and snow falling on the fill

(2) infiltration of water by seepage from the reservoir

(3) possible presence of ice crystals in the imported fill.

Analyses and studies show that the wet seams could not be attributed to:

1. Wet fill brought to the site from the borrow area.
2. Horizontal infiltration of water from the reservoir (maximum distance of percolation is not likely to exceed about 100 ft).
3. Possible importation of frozen soil from the borrow area (analyses indicate that all frost would be thawed before May 8).

However the Interior Review Group investigation team concluded that factors associated with the formation of the wet seams besides the rain and snow during the period of placement in May, were

1. Unsuccessful attempts to mix dry fill with wet soil on the fill surface. "The inability to properly mix adjacent wet and dry layers of material would permit wet zones to remain and create the potential for low density zones within the dry layers.
2. The fact that "placement of the fill was initiated in the spring of 1975 with several notable deficiencies"--mostly in earthwork control practices.

Deficiencies identified by the investigation team included:

1. "Zone 1 fill placement began on May 1, 1975. According to daily reports, the earthwork inspection staff did not reach full strength until May 12, 1975 (Appendix A2).
2. "Estimates of the volume of Zone 1 fill placed in the first part of May 1975 infer that the frequency for performing earthwork control tests was considerably less than the required minimum. Reference 1-3.1. recommends one control test for every 2,000 cubic yards of Zone 1 fill placed."

"The following tabulation shows the volume of fill placed and the number of control tests for routine compaction:

Date	*Approximate volume of fill placed, yd^3	Number of earthwork control tests
May 1	7,000	0
May 2	8,000	3
May 5	12,000	2
May 12	25,000**	3

*Estimated volume based upon load count

**Followed wet weather shutdown which began May 6th.

The preceding information clearly indicates that several Zone 1 lifts placed in early May were not evaluated by earthwork control tests."

Embankment placement during May 1975 resulted in lower densities than during other periods examined by the Interior Review Group. Table 2 from their report shows that Zone 1 fill placed during May had a mean dry density of 92 pcf compared with values of 97 to 100 in June.

These findings led the Interior Review Group investigating the cause of failure to the conclusion:

> "The wet seams within the Zone 1 resulted from the placement of layers and lenses of material which, when compacted to acceptable D-ratios (fill dry unit weight divided by Proctor maximum dry unit weight) had a lower dry density and a higher permeability than the surrounding material.... The low density layers were wetted after placement in the embankment by rain and snow melt percolating into them and, in some locations, by reservoir water infiltration."

There seems to be no good reason to disagree in any way with this finding although there is some reason to believe that some of the water in the wet seams might have resulted from melting of ice crystals brought in with fill from the borrow area in early May. The IRG studies of thawing rates do not support this possibility however.

(b) Potential for Wet Seams Near the Right Abutment

If the foregoing hypothesis for the formation of the wet seams is valid, then it seems unlikely that a similar wet seam could have existed on the right abutment for the following reasons:

1. The main wet seam extended across the left part of the embankment at levels which were 5 to 40 ft lower than the fill level in the right abutment key trench during the 1974-75 winter shut-down (see Fig. 9).

2. Virtually no fill was placed in the right abutment key trench at the same time that the soil containing the wet seams was being placed on the left side of the embankment; the soil containing the main wet seam was placed on the left abutment in the period May 1 to May 29, 1975. Construction did not begin in the right abutment area until May 29. Thus essentially all right abutment key trench fill was placed later when acceptable fill was also being placed on the left side of the embankment.

3. There was no evidence of any wet seam on the exposed face of the embankment after the failure occurred. Thus there was apparently no wet seam in the central portion of the embankment and this material was placed at similar times to the soil in the right abutment.

4. Although the wet seam was clearly evident very near the left abutment just a few feet from the key trench it could not be traced to contact with the rock or into the left key trench, presumably due to greater care in placing soil in this zone. It is to be expected that similar care was exercised in placing the soil near the right abutment and in the right abutment key trench. Thus it seems unlikely that the wet zone could have extended several hundred feet to the right when it is known to have disappeared a few tens of feet to the left.

5. The mean dry density of soil placed in June 1975 near the right abutment was 98 pcf compared with the lower value of 92.4 pcf for fill placed in May in the part of the embankment near the left abutment where the wet seam was located.

(c) Significance of Wet Seam with Regard to Failure

Since it seems unlikely that a wet seam existed in the right side of the dam where the failure was initiated, such a non-existent seam could not have played any role in causing the failure. Furthermore although a wet seam of substantial extent is known to have existed on the left side of the embankment it clearly did not lead to failure on that side of the embankment. Thus where a wet seam is known to have existed failure did not occur and where a wet seam almost certainly did not exist, failure did occur. It seems probable, therefore, that some mechanism other than the existence of a wet seam was responsible for triggering the failure by piping of Teton Dam.

The Interior Review Group also concluded:

> "The wet seams that were encountered during this investigation could not be related to the cause of failure."

However since all direct evidence was destroyed by erosion of the fill during the failure of the dam, the remote possibility of a wet seam on the right abutment cannot be completely eliminated.

Whether a wet seam existed near the right abutment or not is perhaps, to some extent, a moot question since the provision of measures to protect the Zone 1 fill against internal erosion due to any cause would have been equally effective against excessive seepage through a wet seam. In this respect the presence or absence of a wet seam in this area is of relatively little significance but since the preponderance of evidence seems to argue against the existence of such a seam, it directs increased attention to other possible triggering mechanisms for the internal erosion which undoubtedly occurred through the core of the dam.

(d) Other Findings from Excavation

The excavation of fill from the left abutment area also revealed some important findings concerning the condition of the fill in the left abutment key trench:

1. "Fill in contact with the foundation downstream of the grout cap was generally not saturated. However 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."

2. "As the reservoir rose the Zone 1 fill became saturated and settled away from the tops of some joints leaving open voids. Some of these voids were several inches wide....At other locations on the upstream wall of the left key trench, the surface of the Zone 1 fill, observable within joints, had been eroded by flowing water. The observed settlement of fill in joints, apparently as a result of saturation, was judged to be indicative of poor compaction."

3. "During excavation, a number of localized wet spots were found adjacent to the upstream rock wall of the left abutment key trench. These wet spots were of limited extent. The maximum horizontal penetration of water into the fill was about 12 ft and the vertical extent was limited to about 3 ft. Wet spots were adjacent to joints in bedrock. Generally the joints were 3 or more inches wide."

(e) Field Hydro-fracturing Tests

Finally the field investigations included a series of borehole hydraulic fracturing tests. Essentially these confirmed the results of the tests performed earlier, but they also showed, by placing dye in the water in the boreholes and excavating the soil after the test, that the fractures which developed were mostly vertical or close to vertical.

Comparative tests at similar elevations in boreholes penetrating the key trenches and those located entirely within the main body of the embankment also demonstrated the reduced stresses in the key trench due to arching in the soil. This also confirmed the previous conclusions regarding this possibility.

2. Laboratory Hydraulic Fracturing Studies

During 1978 and 1979, a supplementary program of laboratory hydraulic fracturing tests was performed at the University of California, Berkeley (Jaworski et al., 1981). In order to gain an improved insight into the mechanics of this phenomenon tests were conducted under conditions where the stresses and procedures could be controlled accurately and the soil could be examined after failure.

For this purpose, 20.3 cm cubical soil samples were placed in a cubical stress apparatus where three independent normal stresses could be applied to the faces as shown in Fig. 12. A 0.48 cm uncased borehole, 5.1 cm long, drilled in the center of the sample was sealed at the top with an epoxy plug and water pressure was then applied to the walls of the borehole through a steel tube cemented into the epoxy. Water dyed with rhodamine W.T. was used for the fracturing fluid so that the resulting stains in the soil could be used to determine the orientation and extent of the fracture plane following a test. A complete description of this equipment has been given by Jaworski (1979). Tests were performed on both undisturbed and recompacted samples with applied stresses simulating an in-situ stress condition. The results of a typical test are shown in Fig. 13. As the water pressure in the borehole was increased, the flow rate from the hole increased. As the fracturing pressure, u_f, was approached the flow rate increased very rapidly with little or no increase in pressure.

It was found that fracturing always occurred along vertical planes and that test results were very similar to those obtained in the field test program. Of particular interest was the wide scatter in the results of tests performed using the same procedure, compaction water content and density, as shown by the results in Fig. 14. It was found that small variations in soil composition had a far more important effect on hydraulic fracturing pressure than did variations in density or water content.

Of particular interest are the results of two tests in which the borehole water pressure was increased slowly so that failure occurred after a period of days rather than hours. The results of these tests are shown in Fig. 15. It may be seen that the fracturing pressure in these tests, which lasted 9 days and 12 days, are substantially higher than those in similar tests lasting 3 to 4 hours. This effect is probably due to differences in the extent of the zone of seepage around the borehole. The comparative effect of rapid and slow filling of a borehole is shown in Fig. 16, and with a slow rate of wetting the effective water pressure tending to cause fracturing is substantially less than the hydrostatic water pressure in the borehole.

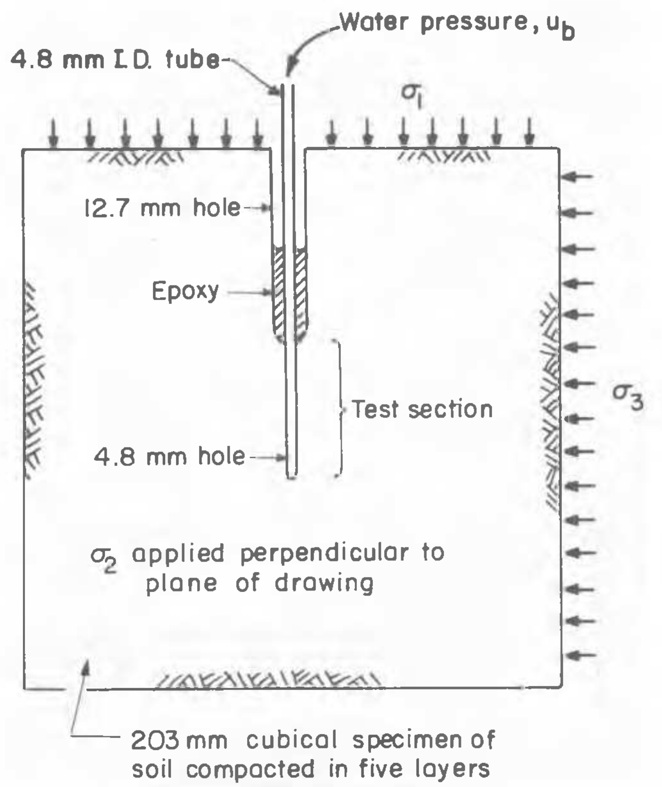

Fig. 12 Laboratory Test of Hydraulic Fracturing Around a Model Borehole

In recognition of this fact and giving consideration to the rate of wetting of the Teton Dam Zone 1 impervious fill it was concluded that a fracturing pressure expressed by

$$u_f \simeq 1.3\ \sigma_H + 1.7\ t_s$$

where σ_H is the minor principal stress and t_s the tensile strength of the soil, is a better basis for evaluating the hydraulic fracturing pressure than the equation

$$u_f \simeq \sigma_H + t_s$$

used in the earlier studies. While this reduced substantially the extent of potential zones of hydraulic fracturing it was also found that pressures determined by this new equation would still indicate that hydraulic fracturing could occur near the base of the key trench fill in the vicinity of Station 15+00, where the failure actually developed (Seed et al., 1976).

Another series of tests was performed to simulate the conditions at the wall of a key trench in jointed rock when backfill is compacted directly against the trench wall. For this purpose a number of tests were performed in which soil was compacted into the cubical test cell with a slot on one side simulating a rock joint

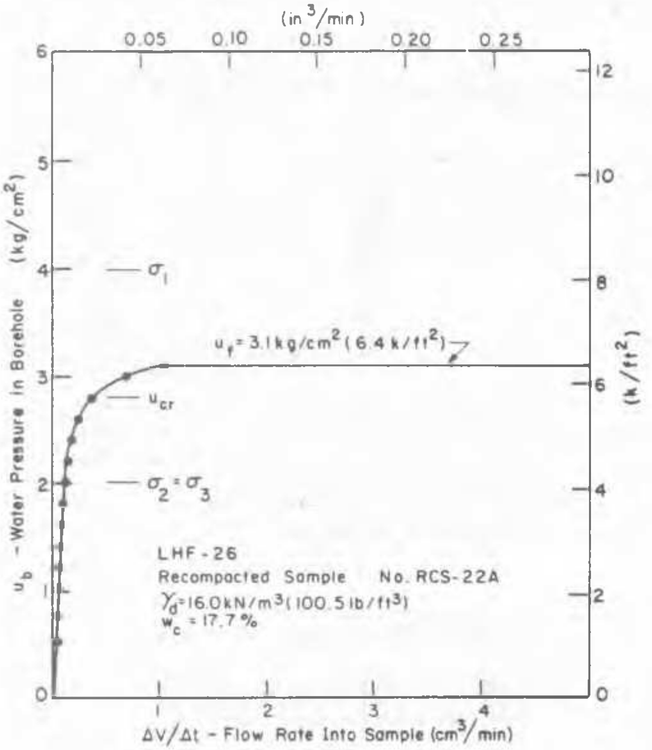

Fig. 13 Typical Result of Borehole Hydraulic Fracturing Test

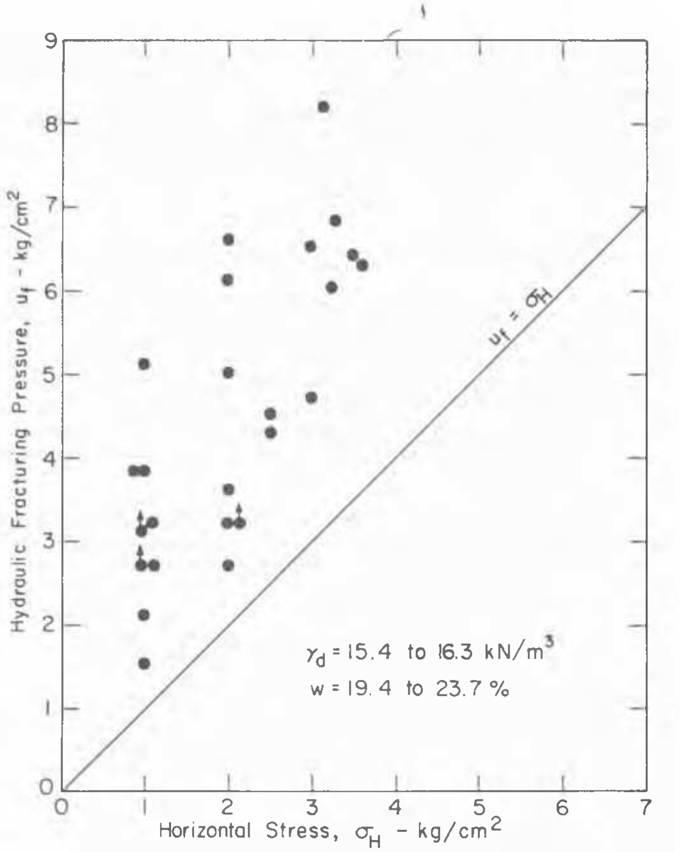

Fig. 14 Results of Hydraulic Fracturing Tests on Recompacted Samples of Teton Dam Zone I Fill

1/2 inch wide and 4 inches high. Subsequently the water pressure in the joint was increased to cause fracturing.

As shown in Fig. 17, attempts to compact the soil adjacent to the open slot or joint led to the formation of a loose zone of soil in this region. When water pressure was introduced into the slot it quickly penetrated the loose zone and acted to wedge the soil apart and cause hydraulic fracturing. As shown on the right side of Fig. 17, fracturing did occur at a water pressure (u_f) which exceeded the value of σ_3 by 1.4 kg/cm^2. This is within the range of values measured in the borehole fracturing tests. The observed fracture coincided with the loose zone adjacent to the simulated rock joint and followed the σ_3-plane across the sample. Similar failures were found in other tests where similar discontinuities were created on the side of a test sample.

In another rock joint test it was found that when water was introduced into the simulated rock joint, the loose soil sloughed into the joint, as shown in Fig. 18. The size of the sloughed zone increased progressively, eventually involving well-compacted as well as poorly-compacted soil, and working its way inward and up to the top of the sample. This observation seems quite significant, because it indicates that the size of a discontinuity resulting from a rock joint can be many times the dimensions of the joint if the soil sloughs as shown in Fig. 18. Clearly not all soils will be vulnerable to sloughing in this way, but some samples of the aeolian silt which formed the core of Teton Dam have this characteristic.

It may be noted that similar behavior was also noted in the field during the excavation of soil in the left abutment key trench. The report of the investigating team states:

> "As the reservoir rose the Zone 1 fill became saturated and settled away from the tops of some joints leaving open voids. Some of these voids were several inches wide....At other locations on the upstream wall of the left key trench, the surface of the Zone 1 fill, observable within joints, had been eroded by flowing water. The observed settlement of fill in joints, apparently as a result of saturation was judged to be indicative of poor compaction."

These observations suggest the mechanism of hydraulic fracturing of the Teton Dam key trench fill which is illustrated in Fig. 19. It was

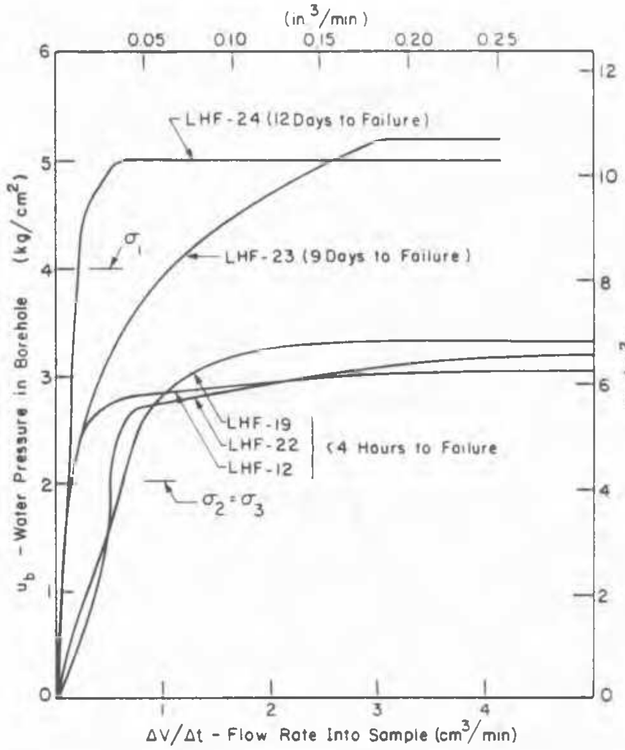

Fig. 15 Effect of Duration of Tests

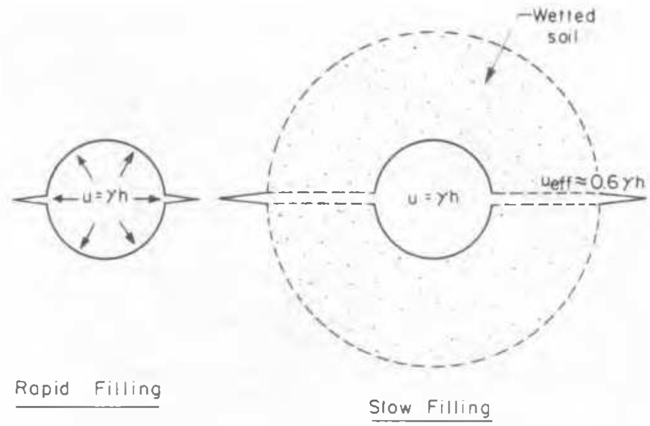

Fig. 16 Comparative Effects of Slow and Rapid Borehole Filling in Hydrofracture Tests

concluded that a zone of loose soil formed during compaction of the fill against the wall of a trench containing a vertical joint. When the water level rose in the joint, the saturated soil fell out leaving a cavity in the fill adjacent to the joint. The gradually increasing water pressure acting on the walls of this cavity was then able to cause hydraulic fracturing across the key trench.

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.

POSSIBLE TRIGGER MECHANISMS LEADING TO FAILURE

In 1977, the triggering mechanisms considered most likely by the Investigating Panels were

(1) Flow of water through the grout curtain just below the grout cap, leading to erosion of soil on the base of the key trench, and

(2) Hydraulic fracturing or differential settlement in the key trench fill leading to cracking across the fill and resulting soil erosion.

As a result of the investigations previously described four other possibilities may be added to this list:

(1) Hydraulic separation between the key trench fill and the base of the trench permitting water to flow, with accompanying erosion, from an upstream open joint along the base of the trench, over the grout cap and into a downstream joint as illustrated in Fig. 20. This possibility exists because the hydrostatic pressure of the water along the base of the trench on the upstream side is approximately equal to the computed downward pressure of the wetted soil on the rock surface.

(2) Seepage through the key trench fill, with accompanying erosion, from an open joint upstream, over the grout cap and into a downstream joint as shown in Fig. 21. At the time of failure, the hydraulic gradient along such a flow path was probably of the order of 7 to 10. This is higher than the hydraulic gradient which is known to have caused piping and a large cavity in the core of the East Branch Dam in 1957 as a result of seepage through a steep-walled key trench similar to that at Teton Dam (Fetzer, 1977).

(3) Seepage through the soil near the base of the key trench, facilitated by sloughing of wetted fill into open joints, thereby progressively increasing the hydraulic gradient as illustrated in Fig. 22.

and

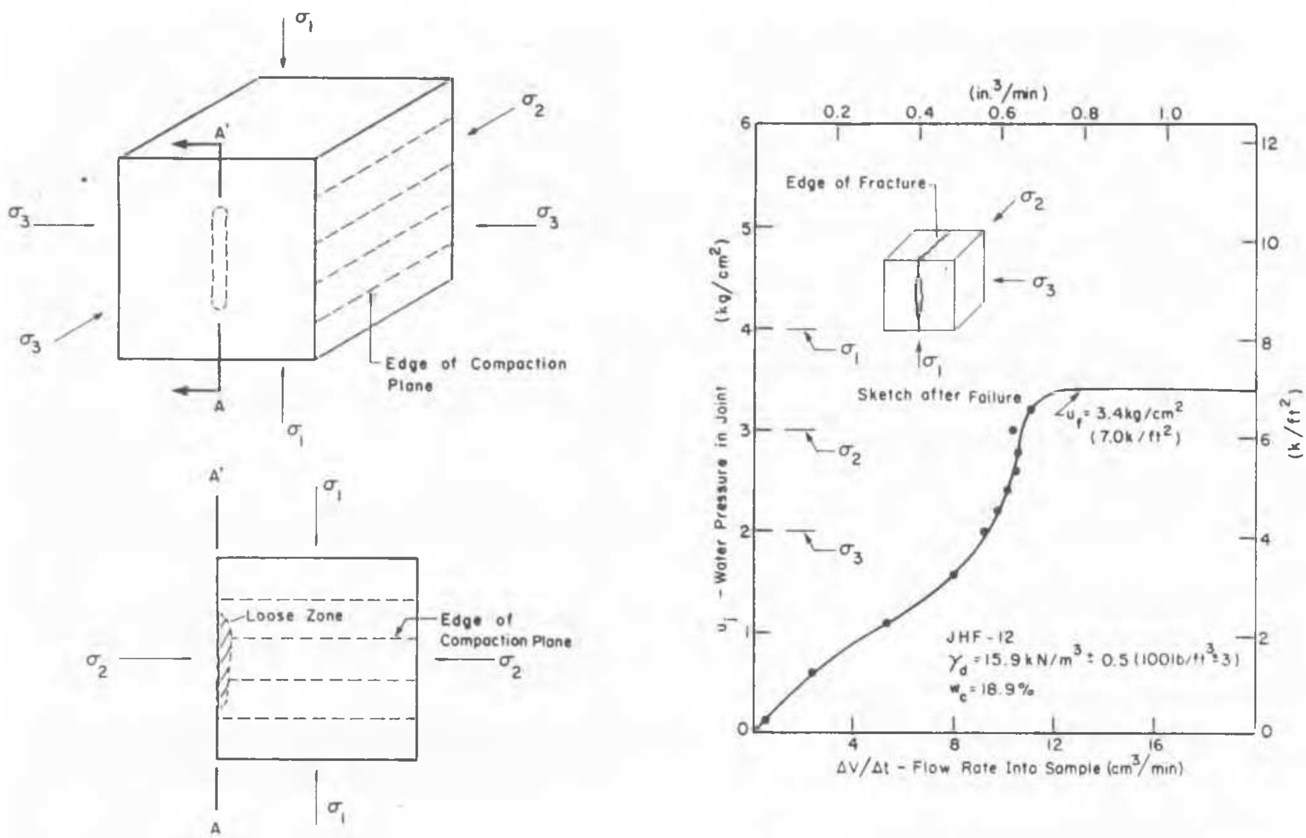

Fig. 17 Test Conditions and Results for Fracture Tests on Soil Compacted Adjacent to Slot

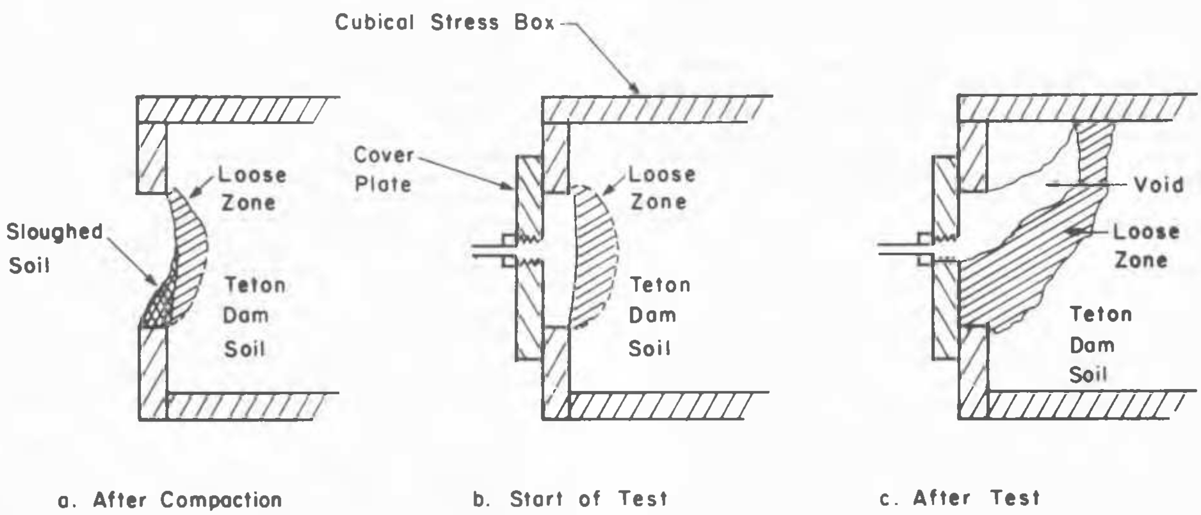

Fig. 18 Sloughing of Soil Around Simulated Rock Joint

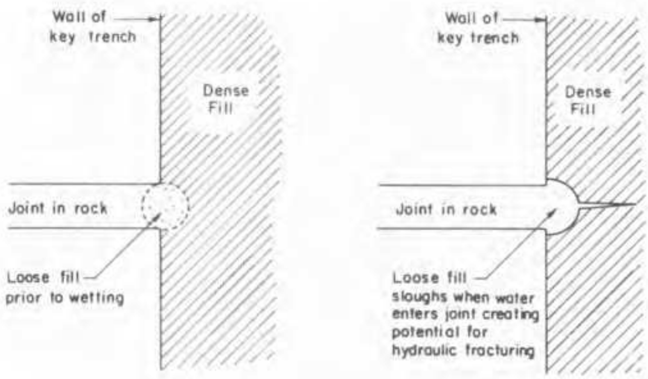

Fig. 19 Conceptual Mechanism of Hydraulic Fracturing in Key Trench Fill

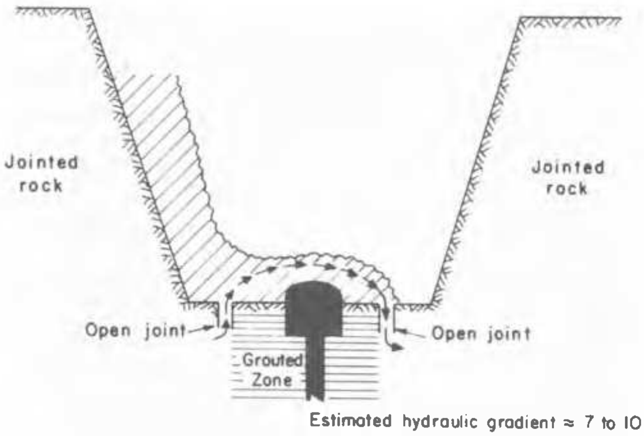

Fig. 21 Piping Due to Seepage Across Base of Key Trench

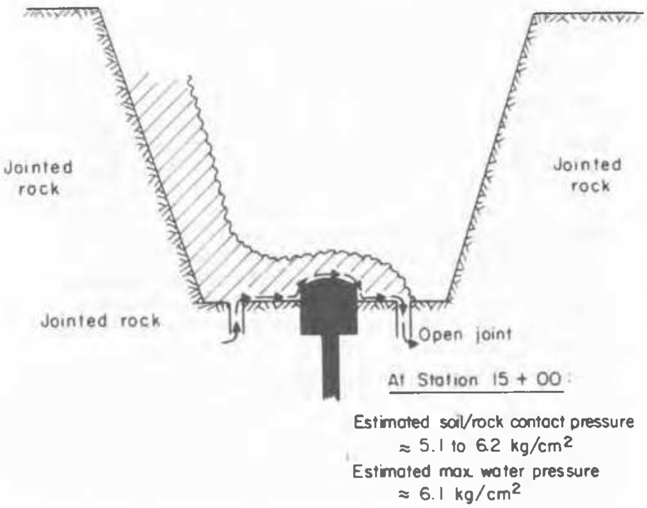

Fig. 20 Hydraulic Separation at Base of Key Trench

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

All of the failure mechanisms postulated above could have been prevented if adequate protection of the Zone 1 fill against internal erosion had been provided in the original design. Such protection would have involved appropriate treatment of the rock to ensure that all joints in contact with the core material were sealed, together with the provision of a suitable filter system for the highly erodible fill. By these means erosion could have been prevented and failure could not have occurred. Thus while the new investigations indicate additional modes of failure to those proposed in 1976 and 1977 they do not change the basic conclusion of the report of the Independent Panel that:

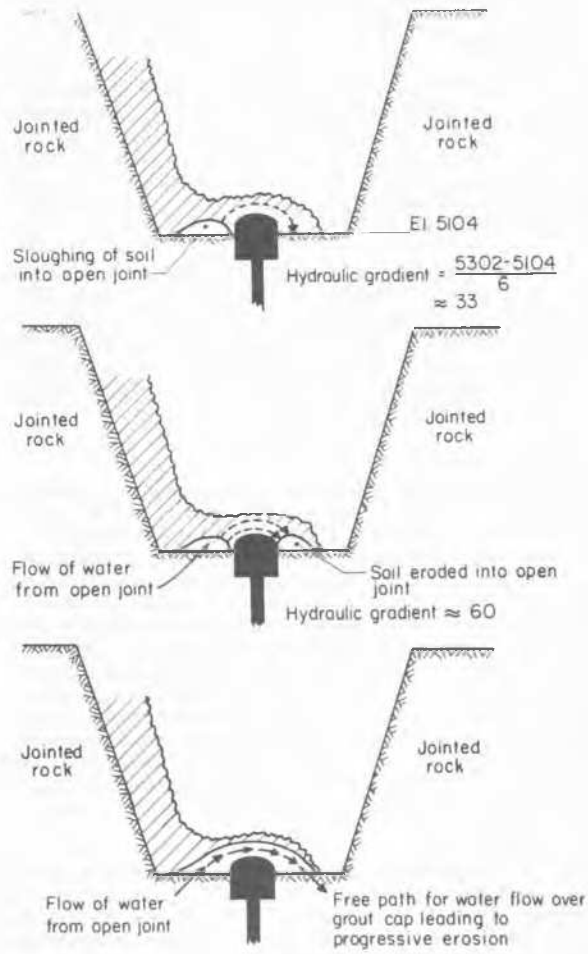

Fig. 22 Piping Due to Seepage and Sloughing Over Base of Key Trench

"The fundamental cause of failure may be regarded as a combination of geological factors and design decisions that, taken together, permitted the failure to develop. The principal geologic factors were (1) the numerous open joints in the abutment rocks, and (2) the scarcity of more suitable materials for the impervious zone of the dam than the highly erodible and brittle windblown soils. The design decisions included among others (1) complete dependence for seepage control on a combination of deep key trenches filled with windblown soils and a grout curtain; (2) selection of a geometrical configuration for the key trench that encouraged arching, cracking and hydraulic fracturing in the brittle and erodible backfill; (3) reliance on special compaction of the impervious materials as the only protection against piping and erosion of the material along and into the open joints, except some of the widest joints on the face of the abutments downstream of the key trench where concrete infilling was used; and (4) inadequate provisions for collection and safe discharge of seepage or leakage which inevitably would occur through the foundation rock and cutoff systems."

After investigations extending over a period of several years the Interior Review Group arrived at a similar conclusion. Citing their 1977 finding that "the dam failed as a result of inadequate protection of Zone 1 impervious core material from internal erosion," the Review Group concluded (1980):

"None of the findings resulting from this continued investigation have changed this primary conclusion. A safe dam could have been constructed if the designers had provided a defensive design with proper embankment filtration and drainage, and appropriate surface treatment."

Probable Mechanism of Failure

While any of the phenomena discussed in the preceding pages could have been the triggering mechanism by which a flow path was created through the impervious core and key trench fill, the basic cause of the failure was clearly progressive erosion of the highly erodible Zone 1 fill material. No matter what caused the initial flow, the flow path would have to develop into a continuous pipe through the embankment in order to lead to the massive seepage which developed in one or two hours just prior to complete failure and which, through accompanying erosion, led to the breaching of the embankment. It is of interest therefore to speculate on the manner in which this transition might have developed.

Playing a key role in this aspect of the failure were undoubtedly the specific nature of the joint systems in the rock between Stations 14+00 and 15+00 on the right abutment and the highly erodible nature of the Zone 1 fill. As observed in the field, there were a number of open joints in the rock in the vicinity of Station 14+00. Also of interest was a 25 ft thick layer of highly pervious talus or slope wash covering the right abutment on the downstream side of the embankment.

A section along the general path of seepage both as evidenced by the field studies and by the observed backward path of erosion towards a whirlpool, which developed in the reservoir as the failure approached, is shown in Fig. 23. The overall progression of piping leading to the failure might thus be visualized as follows (Seed et al., 1976):

"Several days before the final failure, leakage through the key trench fed water at a slowly increasing rate into a number of diagonal joint systems; a portion of this flow entered the joints directly, and a portion entered via the overlying highly fractured rhyolite and talus above El. 5200. As the joint systems began to fill with water, aided by water flow around the end of the right abutment key trench fill, quiet discharges of water occurred several days before the actual failure. Some of the discharges emerged along the base of the canyon wall downstream from the dam and some moved as subsurface flows into the contact zone of talus and heavily jointed rock beneath the Zone 2 and Zone 5 portions of downstream part of the embankment.

"Thus the critical escape route for leakage was the multitude of partially filled void spaces in the loose slabby rock just beneath the Zone 1 fill downstream from the key trench. Significantly, materials partially filling void spaces in this zone of rock would be unaffected by overburden pressures from the overlying fill because of the sheltering action of the loose rock structure. Accordingly, the leakage conveyed to this medium by flow across the key trench at Station 14+00 and thence flowing downward and to the left towards Station 15+00, found not only an almost free exit in the near-surface rock but also escaped in channels that were of such size that they could easily convey soil particles eroded from the core of the dam. Thus of paramount importance was the possibility for leakage flows occurring immediately along the core-to-rock interface to loosen and erode the compacted silt from Zone 1. Although the fill was probably well-compacted, those parts of the fill beneath minor overhangs would inevitably be sheltered from overburden pressures and thus locally vulnerable to erosion.

"In this way the initial seepage probably eroded a small channel along the base of the dam, both upstream and downstream with the seepage flowing under the Zone 2 material, down the talus on the upper part of the right abutment and finally emerging as the leak at the toe of the dam on the morning of the failure.

"As the flow continued, further erosion along the base of the dam and a resulting concentration of flow in this area, led to a rapid increase in the size of the eroded channel. At this stage water probably began to emerge at the contact of the embankment with the underlying rock at about El. 5190 to 5200.

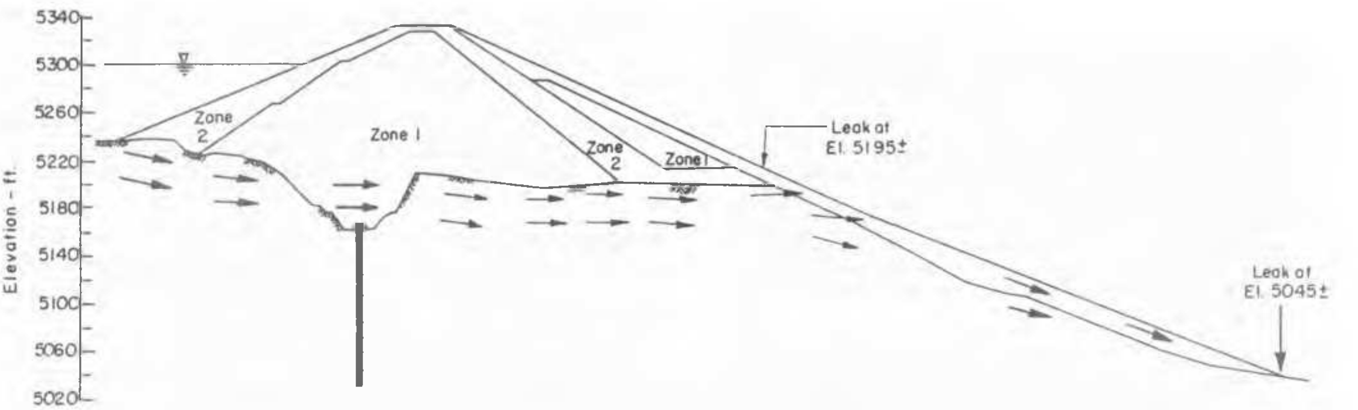

Fig. 23 Probable Path of Water in Early Stages of Leakage

"Progressive erosion led to continued increase in the size of the channel along the base of the dam, and perhaps some erosion of the soil above Zone 2 until finally the water pressure was sufficiently great to break suddenly and violently through the Zone 2 fill and erupt on the face of the dam.

"Beyond this point the progressive formation of sinkholes, both upstream and downstream, provided an ever-accelerating mechanism for internal erosion, finally leading to complete breaching of the dam."

Lessons from the Failure Investigation

The investigations of the cause of failure of Teton Dam provide many lessons, some of which are apparent from the previous discussion. Each reviewer will undoubtedly make his own list of such lessons but the writers list is presented below:

1. It is important to recognize how quickly a dam failure may occur due to internal erosion and piping of erodible construction materials. For this reason it is essential to fill the reservoir slowly under fully-controlled conditions and to have available a means for lowering the water level rapidly (e.g. a low-level outlet) if problems develop.

2. The problem of foundation and abutment treatment for high embankment dams on rock foundations remains one of the most critical aspects of dam design. If the contact surfaces between the impervious core and the jointed rock at the Teton site had been appropriately sealed and a filter layer had been provided to prevent movement of core material into any voids that may have inadvertently remained unsealed, the piping which led to failure of the dam could not have occurred. Sealing of the core-foundation contact and the provision of adequate filter and drainage systems are essential elements of all earth dams.

3. The principle of multiple lines of defense, long advocated by Arthur Casagrande, should never be neglected since there are many unknown circumstances which may arise during construction, such as the use of unexpected types of fill in the borrow areas at the Teton site, which can jeopardize the best designs.

4. While every effort should be made to ensure that an earth dam is built in accordance with the design specifications, materials and conditions not anticipated by the designer may arise during construction which will lead to the incorporation into the embankment of materials of lower quality than those envisaged by the designer. This possibility should be recognized in design and provisions made both to minimize the possibility and also to ensure the safety of the dam in spite of such occurrences.

5. It is the opinion of all investigating panels that the occurrence of the main "wet seam" at Teton Dam could not be related to the failure which ultimately occurred. However under other conditions, such as the occurrence of the design earthquake, the unknown presence of the wet seam could have been the trigger mechanism leading to a major slide whose occurrence would have been a source of mystery to the designers and to the profession as a whole. It is essential that detailed construction records be kept on fill material placed in order that all aspects of embankment performance may be fully understood.

6. No matter how successful a design agency or group may be it is extremely desirable that designs of major dams be reviewed by an independent group of engineers to ensure that no possible design deficiency has been overlooked.

7. Instrumentation designed to monitor the performance of earth dams should be incorporated in all major structures so that any evidence of malfunctioning can be detected at an early stage and remedial action taken to prevent failure.

8. It is virtually impossible to provide a tight grout curtain in highly jointed rock with a single row of grout holes and it is equally difficult to seal all rock defects near the rock surface, no matter how carefully and skillfully the grouting procedures may be performed.

9. It is essential that the designer of an earth dam should remain in close contact with the construction of the dam throughout the entire period of construction so that unanticipated conditions may be recognized and the design modified, as appropriate, to mitigate any hazards which the new conditions may introduce.

10. Abrupt changes in geometric configuration or material stiffnesses in an embankment dam can lead to stress distributions which will greatly facilitate the occurrence of hydraulic separation or hydraulic fracturing. Such abrupt changes should be avoided.

11. While low stresses facilitate the occurrence of hydraulic fracturing, this phenomena can only occur if there are discontinuities present in the soil which will permit the development of tensile stresses in the soil. Such discontinuities include existing cracks in the soil, zones of loose soil adjacent to rock joints, cavities and voids in the embankment soil, and irregular zones of high permeability embedded within less pervious materials.

CONCLUSIONS

Designing and constructing earth dams is one of the most challenging tasks a geotechnical engineer can undertake, and all of us involved in this type of work on a large scale can count ourselves fortunate if we escape with minor mishaps as the only result of our activities and decisions. Indeed it has been said that

> "Earth dam engineering is the art of moulding materials we do not fully understand into shapes we cannot precisely analyze so as to withstand forces we cannot always assess, in such a way that we never-the-less produce safe and economical structures."

We do this mainly by building redundancy into the structures through the use of multiple lines of defense. Unfortunately this was not accomplished in the design of Teton Dam and a failure ensued. It is hoped that we can all benefit from the lessons provided by this failure.

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