Some Consideration of the Teton Dam Failure

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ティートンダムの欠壊に関する考察

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ABSTRACT

Recent disastrous failure of Teton dam created big reverberation in the world and has given many significant precepts for dam engineers. In order not to give rise such an unfortunate accident again, engineers should make efforts to conduct close examination of possible aspects that could have contributed to the failure and to make the best use of their findings for future activities of design and construction. The authors have had an oppotunity to visit and inspect the damsite in Oct., 1976 as member of the investigation party organized by the Governor of Gifu prefecture. This paper was made as a report of the authors' findings on the cause of the Teton dam failure and submitted in Dec., 1976 to Gifu prefecture. The present investigation is based on some contributed materials and analytical studies by the finite element method, and discussions are given principally on the possibility of crack generation and the hydraulic fracturing potential of the dam. As one of investigation groups in U. S. A. reached final conclusicn recently, its view on the cause of the failure is also presented in this paper.

INTRODUCTION

Teton dam which was designed by the U. S. Department of the Interior Bureau of Reclamation failed on June 5, 1976. As the Bureau of Reclamation has achieved an international reputation on design and construction of fill-type dams, the news on the failure created big reverberation in the world and gave shock not only to dam engineers but also to the people who are now living downstream region of dams.

Immediately following the failure of the dam, two panels were established in order to investigate and study on the cause of the failure. One of these, the Teton Dam Failure Interior Review Group, was organized by the U. S. Department of the Interior, composed of six representatives of selected Federal agencies. The other, the Independent Panel to Review Cause of Teton Dam Failure, which was charged by the Secretary of the U. S. Department of the Interior and the Governor of the State of Idaho, is composed of ten members of scholors and dam specialists headed by Chadwick, W. L..

As of December, 1976, Interior Review Group has submitted interim reports two times (Teton Dam Failure Interior Review Group, 1976a, 1976b). In the second report dated Oct. 21, 1976, the group listed up six possible causes ot the failure based on its investigations and available informations. The Independent Panel, on the other hand, submitted the final report to the U. S. Department of the Interior and State of Idaho on December 31, 1976 (Independent Panel to Review Cause of Teton Dam Failure, 1976). In this report the Panel identified many possible causes of the failure and examined each of them in detail through extensive investigations and studies. The Panel then concluded finally that the failure resulted from piping, a process by which embankment material is eroded internally and transported by water flowing through some channel in the embankment sections.

The authors have been much concerned with the failure of Teton dam since it happened. In this paper, some theoretical examinations are given on the cause of the Teton dam failure on the basis of contributed materials concerning the design of the dam, construction records and laboratory and field test results on embankment materials. Numerical analyses are performed by the finite element method and discussions are given principally from viewpoints of crack generation and hydraulic fracturing in fill-type dams.

DESCRIPTION OF TETON DAM

Following description is refered to the materials presented by the Teton Dam Failure Interior Review Group (1976a), Aberle (1976) and U. S. B. R. (1970).

(1) Project and Design

Teton dam is located on the Teton river, a branch of the Snake river, about 50km northeast of Idaho Falls, Idaho. The project is a multipurpose dam with a total capacity of 3.5×10^{8} m^3. The dam is a multizoned earth embankment rising about 120m above the base foundation. Construction of the dam was completed in November, 1975. Storage was begun in early October, 1975 and continued until the failure on June 5, 1976.

Type of Dam	Multizoned Earth Fill
Purpose	Multipurpose (Flood Control, Power Generation, Irrigation, etc.)
Height	93m (+31m Cut-Off Trench)
Crest Length	930m
Fill Volume	7,260,000m^3
Reservoir Capacity	350x$10^6$$m^3$ (effective 240x$10^6$$m^3$)
Spillway Capacity	500 m^3/sec.
Project	U.S. Bureau of Reclamation

Table—1 Features of Teton Dam

Some features of Teton dam are described in Table-1, and plan view and maximum cross section of the dam are shown in Figs. 1 and 2, respectively.

(2) Topography and Geology

A longitudinal section along the axis is shown in Fig. 3. The river bed is at about El. 1530m and its width is about 220m. Both abutments consist of steep rock walls extending to El. 1620m, with slopes ranging from 1 : 1.0 to 1 : 1.5. Foundation rock is welded, rhyolite ash-flow tuff, which is about 1.9million years old. Inhigher elevations

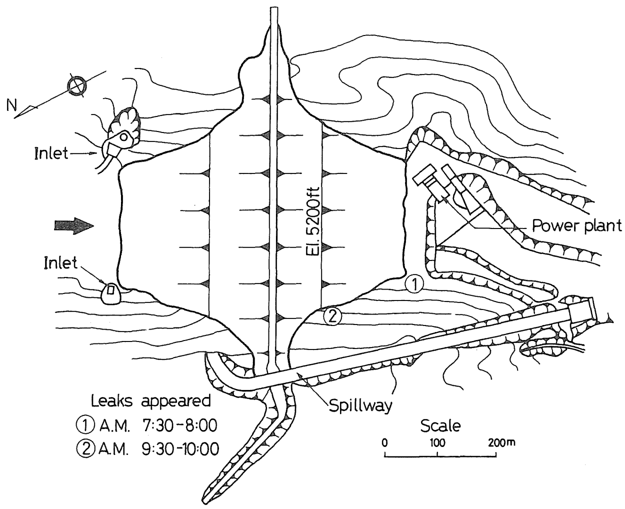

Fig. 1 Plan View of Teton Dam

above El. 1556m (5100ft), both valley walls are covered with exceedingly weathered and open jointed rocks reaching about 20m deep.

(3) Foundation Treatment

For the purpose of seepage control at the core contact area, foundation excavation was done for the base foundation and both abutment rocks. A key trench about 20m deep with 1 : 0.5 side slopes was installed in each

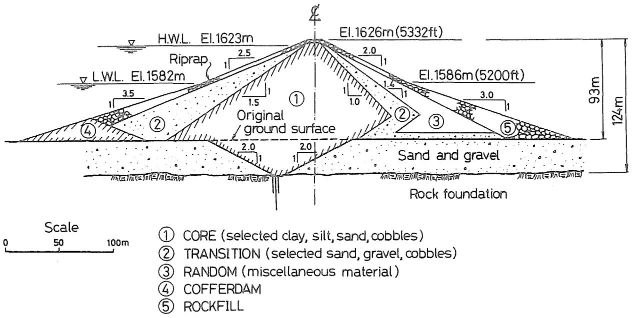

Fig. 2 Cross Section of Teton Dam

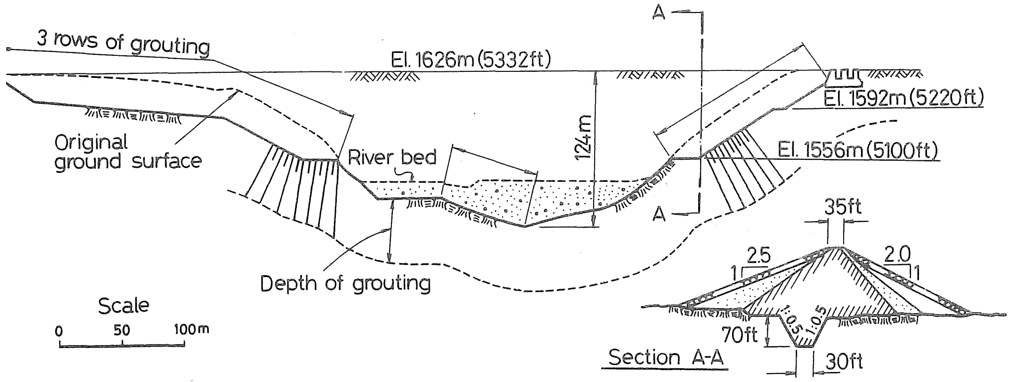

Fig. 3 Longitudinal Section Along the Axis

abutment rock above El. 1556m, as shown in Section A-A in Fig. 3. Below that elevation, a cut-off trench was excavated to foundation bedrock, reaching about 30m in maximum depth. The foundation was grouted below these trenches extending from 60m to 80m in depth. In higher elevations above El. 1556m and in the section between stations 19+90 and 23+90 where foundation rock is highly fractured and jointed, three rows of grouting was needed as shown in Fig. 3, while the grout curtain in other areas consisted of two rows of grout holes.

(4) Embankment Materials

Type of fill materials used in the embankment are indicated in Fig. 2. Physical properties and placement conditions of Zone 1 (core) material are summarized in Table-2 and Fig. 4. In Fig. 4, an average gradation of Zone 1 material is compared with grain size distributions of materials which are susceptible to cracking, presented by Sherard et al. (1963). The bottom column in Table-2 exhibits the difference of settlements before and after saturation of materials in oedometer tests. According to

Specific Gravity (Gs)		2.65
Containing Soil Particle	Silt	60-70%
	Clay	10-20%
Atterberg Limit	w_L	25-30%
	Ip	N.P.
Classification		ML
Compacted Moisture Content		1.0-2.5% Dry of Opt.
Compacted Density	γ_t	1.85-1.95 (g/cm³)
	γ_d	1.50-1.65 (g/cm³)
Compressive Strain due to Saturation		0.25-0.75%

Table—2 Teton Zone 1 (Core) Material the test results on undisturbed samples, more than 95% of them showed the increase in settlement on wetting, ranging from 0.25% to 0.75%.

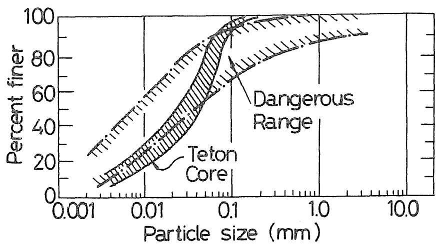

Fig. 4 Range of Cracky Material (after Sherard etal., 1963)

PANELS' VIEW ON THE CAUSE OF THE FAILURE

As stated before, the investigation and study on the cause of the Teton dam failure have been undertaken by two panels. One of these, the Teton Dam Failure Interior Review Group, stated in its first interim report that the dam failure had been a result of internal erosion (Teton Dam Failure Interior Review Group, 1976a). The group also identified several design and construction aspects which might have contributed to the development of internal erosion and the rapid failure. These are, a) extremely open-jointed and pervious nature of the abutment rock, b) brittleness and erodibility of Zone 1 material, c) narrow key trench bottom, d) the difficulty of thorough sealing of cracks in the rock in contact with Zone 1, e) the difficulty of adequate compaction of Zone 1 material against the key trench and abutment, f) grouting the upper layers of rock under the spillway structure in lieu of extending the key trench through this reach, g) the geometry of the steep abutment with a deep, narrow key trench.

On the basis of field observations at failure, the group has drawn an opinion that either one or a few of the above-listed aspects could have induced internal erosion and resulted in subsequent failure. They have then pointed out following six items as most possible aspects (Teton Dam Failure Interior Review Group, 1976b).

1. Cracking or hydraulic fracturing of Zone 1 material.
2. Piping along the interface between the Zone 1 material and rock foundation.
3. Flow through the grout curtain.
4. Flow bypassing the grout curtain.
5. Cracking due to foundation settlement.
6. Cracking due to hydraulic uplift.

The above six causes are ranked based on the group's judgement of probability of occurrence.

The Independent Panel, on the other hand, submitted the final report on December, 1976 and stated that the dam failed by internal erosion (piping) of the core of the dam deep in the right foundation key trench (Independent Panel to Review Cause of Teton Dam Failure, 1976). The Panel's detailed considerations and conclusions on the cause of the failure are summarized in the following.

1. The pre-design site selection and geological studies were appropriate and extensive.
2. The design did not adequately consider the effects of differing and unusually different geological conditions at the Teton damsite.
3. The volcanic rocks at the damsite are highly permeable and intensely jointed. Water was therefore free to move in most directions except where the joints had been effectively grouted.

4. Nonplastic to slightly plastic clayey silts used for the core and key trench fill are highly erodible: the use of this material adjacent to the heavily jointed rock of the abutment was a major factor contributing to the failure.
5. Construction activities conformed to the actual design in all significant aspects except scheduling.
6. Rapid rate of filling of the reservoir was not a factor contributing to the failure.
7. Grout curtain was not inferior to that have been acceptable on other projects. Nevertheless, field observations showed that the rock immediately under the grout cap was not adequately sealed. Too much was expected of the grout curtain and the design should have provided measures to render the inevitable leakage harmless.
8. The geology of the key trenches, with their steep sides, was influential in causing transverse arching.
9. Stress calculations by the finite element method indicated that the arching was great enough at the base of the key trench between Stas. 14+00 and 15+00 and cracking by hydraulic fracturing was a theoretical possibility in that area.
10. Differential movements of the foundation are not considered to have contributed to the failure.
11. The dam was not instrumented sufficiently to enable engineers to be informed fully of the changing conditions in the dam.

The Panel finally concluded that the failure had resulted from piping and believed that two mechanisms were suspect. One is the flow of water against the highly erodible and unprotected key trench filling, through joints in the unsealed rock immediately beneath the grout cap and the consequent development of an erosion tunnel. The other is cracking caused by differential strains or hydraulic fracturing of the core material filling the key trench.

FINITE ELEMENT ANALYSIS

In examining the cause of the Teton dam failure, it should be attached great importance to the fact that considerably wide flat benches were left on both abutments in the excavation of key trenches, and that low plastic brittle material which is susceptible to cracking was used in the core zone. Because it has been pointed out since old times that such benches, cliffs or overhangs are sufficient to cause large differential settlement at the core contact area and subsequent cracking and progressive erosion have often resulted in serious damages in several dams such as Stockton Creek dam (Fig 5 : Sherard, 1973). In the following, the authors give some discussions on the Teton dam failure on the basis of the finite element analysis. Analysis is made to evaluate stress and strain conditions within a longitudinal section of the dam, and examinations are presented on the cracking potential and hydraulic fracturing of the dam.

The profile of the dam was divided into triangular finite elements, as shown in Fig. 6. In this case, only the right section of the embankment in which the breach had occured was given attention and idealized as a half of a

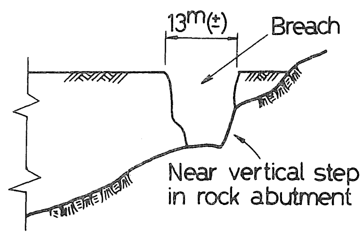

Fig. 5 Breach of Stockton Creek Dam (after Sherard, 1973)

symmetric embankment. Two flat benches were left in the excavation of the key trench of the right abutment: i. e., one is about 34m below and the other is about 70m below the crest of the dam, respectively. Note that the lower bench is comparatively wider than the upper and an abrupt change in slope inclination is seen at this location.

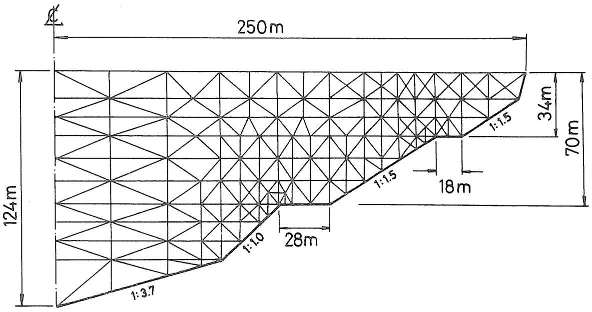

Fig. 6 Finite Element Idealization of Teton Dam Embankment

To evaluate first post-construction deformation condition in the embankment, a single lift linear analysis was made considering the dam be constructed at a stretch. Applicability of single lift solutions to the problem of crack generation in fill-type dams has already been discussed by Narita (1976). Fig. 7 shows contours of major principal strain ε_1 obtained in the analysis (tension is positive), in which numerical values indicated give actual strains in percents if they are multiplied by the dimensionless quantity $\gamma H/E$ for the dam under consideration. It is recognized that concentration fields of tensile strain appear not only at the crest of the dam but in the vicinity of both benches, and that the maximum tensile strain developed near the lower bench is about two times as large as others. The extent of the tension zone which is defined by the horizontal stress distribution along the crest is also presented in the figure. It is seen that the tension zone extends up to the point just above the left edge of the lower bench.

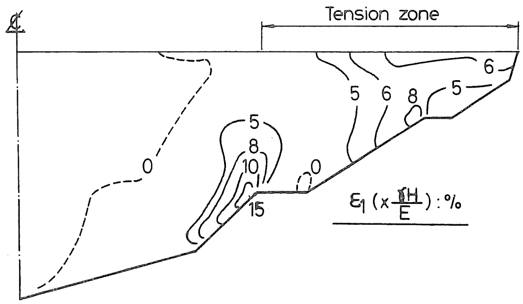

Fig. 7 Contours of Major Principal Strain After Construction

Non-linear incremental loading analysis was then carried out to assess the stress and strain conditions in the embankment during construction. Analytical procedure adopted here is that proposed by Kulhawy et al. (1969), which employs hyperbolic stress-strain relationships. Hyerbolic stress-strain and strength parameters used in the present computation are listed in Table-3. These were determined refering to the results of drained triaxial compression tests on saturated samples of Zone 1 material (U. S. B. R., 1970). The material unit weight was taken as 1.9 t/m³ in this computation (Table-2).

K	470
n	0.12
c' (kg/cm^2)	0.015
ϕ' (degree)	35.0
R_f	0.80
G	0.35
F	0.17
d	3.80

Table— 3 Stress-Strain Parameters

Figs. 8 and 9 show the distributions of internal stress and strain in the state just at the end of construction. In Fig. 8, contours of major principal strain ε_1 show similar concentration fields of tensile strain on the left side of both benches as is indicated in Fig. 7. In the distribution of major and minor principal stresses (Fig. 9), the principal stress ratio σ_1/σ_3 becomes larger near both benches, resulting in concentration of shear stresses. In the top portion of the embankment, tensile stresses develop in the shaded elements: their stress levels are represented in kg/cm². It is recognized that comparatively large tensile stress develops at the core contact area where the inclination of slope is locally steep due to the proximity to the wall of spillway structure.

Fig. 10 shows the distribution of the ratio "R" of minor principal stress σ_3 to the hydrostatic pressure $\gamma\omega h$ at the element under consideration, in which the reservoir is assumed to be fully impounded. The ratio R is expressed in percentage and every element is marked according to the range of R as it exists.

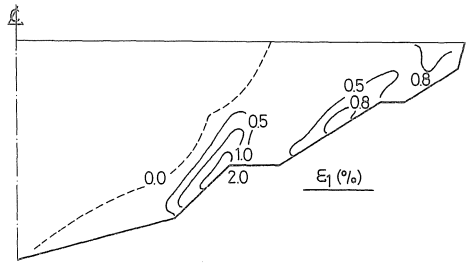

Fig. 8 Countours of Major Principal Strain During Construction

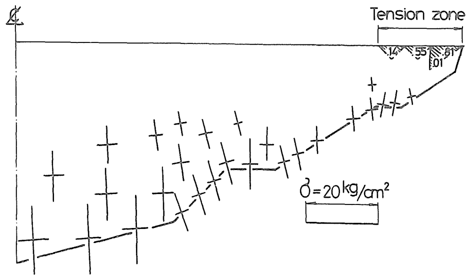

Fig. 9 Distribution of Major and Minor Principal Stresses

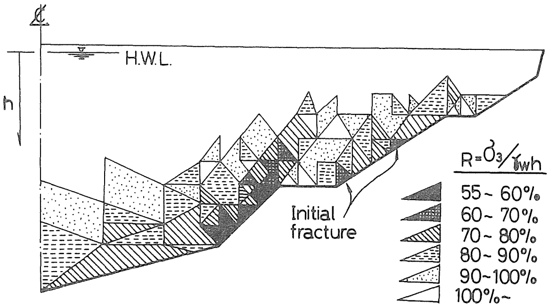

Fig. 10 Distribution of "R"

EXAMINATION OF CRACKING POTENTIAL

Tensile strains which may have developed during and after construction are roughly estimated at a few representative portions of the embankment, as shown in Table-4. In which the value of the tensile strain during construction is that of the maximum strain in each concentration field presented in Fig. 8. On the other hand, the tensile strain after construction can be predicted in the following manner.

According to the construction records of Teton dam, incremental strains which developed when undisturbed samples were saturated in oedometer tests show about 0.5% on the average (Table-2). This incremental strain can be considered to be an average vertical strain w/H of the embankment which develops at first filling of the reservoir. The crest settlement given by the single lift linear analysis showed $w=0.285\gamma H/E^2$ at the central axis of the embankment, and therefore results in $w/H=0.285\gamma H/E$. The value of the dimensionless parameter $\gamma H/E$ is then given as 1.75×10^{-2} by equating w/H to 0.5% and tensile strains are calculated applying this value to Fig. 7, as indicated in Table-4.

	Crest	Upper Bench	Lower Bench
During Construction	0.31	0.95	2.45
After Construction	0.18	0.13	0.26
Total	0.49	1.08	2.71

Table— 4 Tensile Strains

The possibility of crack generation in the top portion of the dam can be assessed by comparing the predicted strain with the cracking strain from pure-tension test. Concerning with the top surface of the dam, tensile strain of about 0.18% is recognized only after construction. On the other hand, though pure-tension test has not been conducted on the Teton dam material, it can be easily presumed according to the results of beam tests (Leonards and Narain, 1963) that such low or nonplastic materials as Zone 1 material would have the cracking strain from 0.1% to 0.2%. It has been reported that cracks were actually detected on the crest surface of the dam near the left abutmet. It is then of interest to note that the above fact is supported with a sufficient reliance by the comparison of these strains, while the predicted strain was given by a rough estimation having assumed the reservoir be fully impounded at a time.

Taking notice of areas near the flat benches, the tensile strain which is to be compared with the cracking strain in experiments is considered to be the sum of strains during and after construction. As both major and minor principal stresses are in compression in these areas, failure strains resulted from triaxial extension tests would be appropriate for the cracking strain to be compared. In this sense, the authors performed extension tests on weathered mudstone material and the Teton Zone 1 material, as shown in Figs. 11 and 12, respectively. The extension characteristics of the weathered mudstone was investigated by giving variety to the confining pressure σ_3 and compared with the results of pure-tension test (Fig. 11). In extension tests, the lateral pressure σ_1 was increased incrementally up to the failure maintaining the vertical pressure σ_3 be constant. The point denoted by a symbol (▽) on every stress-strain curve signifies the failure state of the specimen where deviator stress reaches maximum value. Beyond this point, the extension strain increases without the increase in deviator stress and the specimen is finally constricted and cut off. The failure strain defined by this critical state shows somewhat constant increase with the increase in applied confining pressure, in the range from 5% to 10%. Note that the cracking strain in pure-tension test on the same sample is 10 to 20 times as small as failure strains in extension tests.

Fig. 12 shows the results of triaxial extension and compression tests on the Teton Zone 1 material. Drained tests were performed on specimens both in unsaturated and saturated states with a confining pressure $\sigma_3 = 0.5$ kg/cm² : the confining

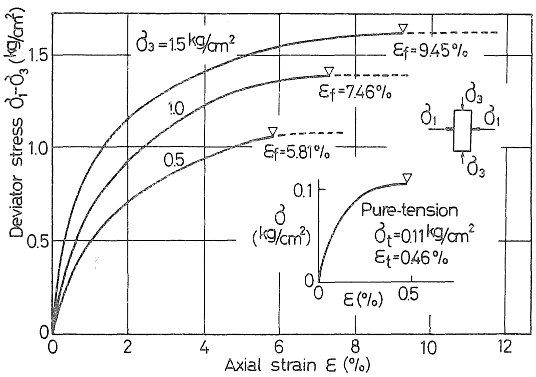

Fig. 11 Triaxial Extension Test (Weathered Mudstone: UU-test)

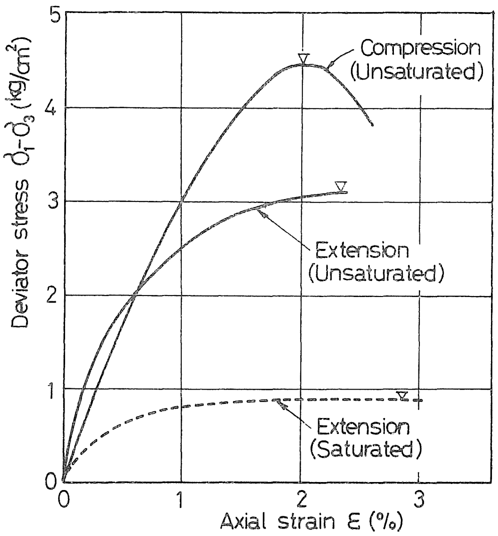

Fig. 12 Core Material of Teton Dam (CD-test, $\sigma_3 = 0.5$kg/cm²)

pressure was restricted to this extent for the limit of loading capacity of the apparatus. It is noticed that failure strains are almost the same magnitude both in extension and compression tests, but that the failure strength in extension tests on saturated specimen decreases to be around one-third of that on unsaturated specimen. This suggests that the examination of the cracking potential not only from a strain condition but from a stress condition would be required for areas under a state of compressive stresses.

Giving attention now only to the strain condition, the sum total of the predicted strain indicates 1.08% near the upper bench and 2.71% near the lower one(Table-4). On the other hand, the failure strain in extension tests is in the range from 2.4% to 2.9% at $\sigma_3=0.5$ kg/cm^2, as shown in Fig. 12. Hence, considering that a confining pressure $\sigma_3=0.5$ kg/cm^2 is less consistent with the stress level near the benches, and that both failure stress and strain showed an increase with increasing confining pressure (Fig. 11), cracks would not be expected to occure in such areas where the overburden pressure is considerably high as 34m or 70m in height.

EXAMINATION OF HYDRAULIC FRACTURING

When an embankment deformation is accompanied by the differential settlement, minor principal stress σ_3 tends to decrease locally. If σ_3 decreases sufficiently being zero or negative (tensile) and if major principal stress σ_1 is less than the unconfined compressive strength, a possible state of cracking appears and cracks may open in further deformation. Even though no cracks were found, reservior water would open existing closed cracks or create new cracks when σ_3 decreases beyond the state of equilibrium with the hydrostatic pressure. If cracks were created, water would permeate into them and make them wider, resulting in progressive erosion and concentrated leaks through them. This kind of phenomenon in which the reservoir water pressure is closely connected with the dam failure has been refered to in general "hydraulic fracturing" (Sherard, 1973).

From a practical point of view, it can be considered that the possible state of hydraulic fracturing arises when the total minor principal stress acting on a certain plane becomes small enough as compared to the hydrostatic pressure (to make the problem simple, the tensile strength which is to be added to the minor principal stress in judgement is regarded as zero in the present examination). However, the above stress state does not necessarily provide the critical (initiating) condition of hydraulic fracturing. Because fracture may be hard to occur when protective facilities such as filter and drainage zones are provided adequately against erosion of soil particles, and so is it when major principal stress is much greater than the unconfined compressive strength.

For the purpose of examining the hydraulic fracturing potential at Teton dam, minor principal stress σ_3 is compared with the hydrostatic pressnre $\gamma\omega h$ in Fig. 10 for every element within the longitudinal section of the dam. The distribution of R indicates the most dangerous region along the slope on the left side of the lower bench, showing 50% to 70% of R over considerably wide extent. It also shows lower values of R extending over the left portion of the upper bench. In such a narrow V-shaped key trench as in Teton dam, arching action is necessarily anticipated to occur. As the present analysis has not paid special attention to the arching effects, some considerations are supplemented on them as follows.

Taking note now of only stress conditions, criteria for hydraulic fracturing can be written in practice as

$$\bar{\sigma}_3 < P_w \ , \quad \bar{\sigma}_1 = \frac{1}{K_0}\bar{\sigma}_3 < P_w \cdots\cdots (1)$$

in which K_0 is the coefficient of earth pressure at rest, and P_W is the hydrostatic pressure (Fig. 13). Major and minor principal stresses denoted by $\bar{\sigma}_1$ and $\bar{\sigma}_3$ in Eq. (1) are those containing the effects of transverse arching, and these are related individually to σ_1 and σ_3 obtained in the present computation as

$$\bar{\sigma}_3 = \alpha\sigma_3 \ , \quad \bar{\sigma}_1 = \beta\sigma_1 = \frac{\beta}{K_0}\sigma_3 \cdots\cdots (2)$$

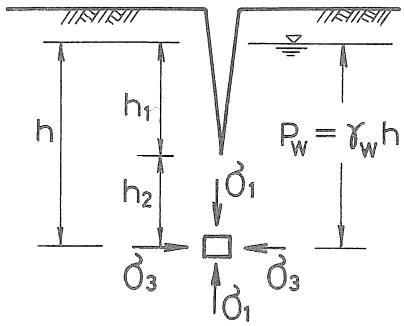

Fig. 13 Stress State

where α and β are coefficients of stress reduction due to the arching effects. In view of the analytical results on other fill dams and on ditch conduits (Narita, 1975), it can be presumed that the reduction of the overburden pressure due to arching amounts to 30% to 50% in case of the Teton dam key trench: that is, $\beta = 0.5 \sim 0.7$. This estimation is really compatible with the results of the finite element stress analyses on cross sections of the Teton dam embankment (seed et al., 1976), in which maximum stress reduction of 40% and 60% is recognized in the key trench, respectively, before and after saturation of the fill. Thus, for the V-shaped key trench in higher elevations above El. 5100ft, the latter criterion in Eq. (1) yields

$$\sigma_3/P_w < \frac{K_0}{\beta} = (1.5 \sim 2.0)K_0 \cdots\cdots (3)$$

Using $\phi = 35°$ (Table-3) and $K_0 = 1 - \sin\phi = 0.43$, this becomes

$$\sigma_3/P_w < 60\% \sim 80\% \cdots\cdots (4)$$

On the other hand, for comparatively flat foundation below El. 5100ft, arching action is supposed to be less influential on the overburden pressure. When stress reduction of around 15% to 20% is assumed for this part, the criterion becomes

$$\sigma_3/P_w < 50\% \cdots\cdots (5)$$

As stated before, fracture does not always occur even when σ_3 satisfies the former criterion in Eq. (1). It is well understood, however, that much more severe situation of hydraulic fracturing would come out if σ_3 also satisfied Eq. (4) or Eq. (5), because these are essentially equivalent to the criterion for major principal stress. When applying these criteria to the distribution of R in Fig. 10 and examining the fracture process of Teton dam, it can be considered that the hydraulic fracture might have developed first along the abutment slope between the two benches (rather in the vicinity of the lower bench), and have showed progressive upward enlargement to the upper bench. This inference shows comparatively good correpondence to the situation of the Teton dam failure reported by the

U. S. B. R.: that is, concentrated leaks through the dam had appeared first in the vicinity of the point at El. 5200ft no the downstream slope between the stations No. 14 and No. 15 (Fig. 1).

SUMMARY AND CONCLUSIONS

Based on the above discussions, following remarks can be drawn on the cause of the Teton dam failure.

As indirect causes of the failure, the abutment configuration is the most possible aspect to be considered. That is, one is a couple of benches left on the right abutment, which might have been sufficient to cause large differential settlements at the core contact area. Another is the V-shaped key trench with steep side walls excavated in higher elevations above El. 5100ft, in which reduction of the overburden pressure due to arching of soil could have led the dam in a potentially dangerous state of hydraulic fracturing. Hence, following two are most probable aspects as direct causes of the failure.

(1) Several vertical transverse cracks opened on the top surface of the dam due to the differential settlement. Cracks extended over wide area between the two benches and reached deeply as the settlement developed.

(2) Minor principal stress decreased enough, espacially near the benches, as the differential settlement increased and arching action proceeded when the material was wetted. Critical state of hydraulic fracturing appeared in the V-shaped key trench.

Thus, following circumstances can be supposed on the failure process.

For the parts near the two benches, rather severe strain concentration and stress reduction had been experienced due to the differential settlement during construction (Fig. 8). As the dam was completed and the reservoir began to rise in comparatively high speed, settlements developed rapidly due to the saturation of fill materials, reaching about 0.5% on the average. Such post- construction settlements were also accompanied by strain concentration on and around the benches, and the sum total of maximum tensile strains during and after construction may have reached about 1 to 3 percents (Table-4).

In the course of storage of the reservoir and of development of the differential settlement, several cracks might have opened in some part where the tensile strain exceeded the cracking strain of the material, and hydraulic fracture might have occured where minor principal stress decreased enough beyond the hydrostatic pressure. Through existed cracks or newly created cracks, concentrated leaks developed and percolated water flew into downstream abutment rock. As the abutment foundation consisted of fissured and open jointed rocks, fairly large volume of core material must have been eroded and flown into open cracks. Interior cracks of the core were then enlarged gradually due to erosion of soil particles, and concentrated leaks still more developed correspondingly. In proportion as leaks and erosion progress, percolated water flew out from the point at El. 5200ft on the downstream slope as well as flew in foundation rock, and induced erosion and piping of downstream part of the embankment. In succession of piping, large volume of fill materials were carried away from the embankment and it extended gradually upwards to the top of the dam, and the fill collapsed finally.

Supposing that Teton dam followed the above-mentioned failure process, the authors cannot but point out some defects on the design of the dam: for example,

(1) It has often been reported that rapid differential settlement sometimes develops when fill materials compacted in dry of optimum are saturated by filling of a reservoir and it is very contributive to some damages like cracking and piping. Despite of such concerns involved, Teton dam embankment was not one with sufficient structural resistance against them and, furthermore, most of the fill was constructed of materials with low plasticity, erodible and sufficient to cause large differential settlement on wetting.

(2) Deep key trenches with steep side walls and a couple of flat benches were left at foundation excavation. These are most influential factors to cause large differential settlement and arching action locally.

Concerning the structural defect in the item (1), filter zones serving drainage facilities as well should have been provided in the interior of the embankment and also at the core-abutment interface, for example, as shown in Fig. 14. It can be considered much more desirable and effective to make the core zone narrower to be adjusted to the key trench and be intercepted with filter zones, as shown in Fig. 15.

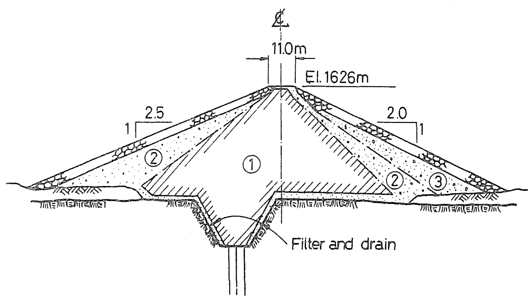

Fig. 14 Filter and Drainage Facilities Interfacing Abutment

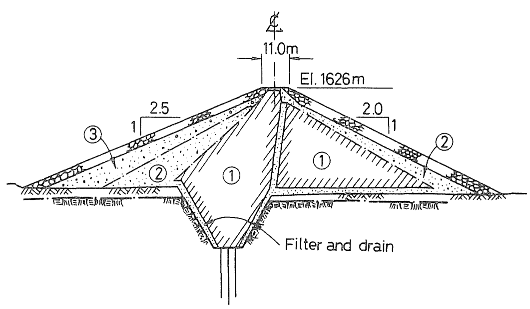

Fig. 15 Desirable Dam Type

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