

REPORT TO
U.S. DEPARTMENT OF THE INTERIOR AND STATE OF IDAHO
ON
FAILURE OF TETON DAM

BY
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

DECEMBER 1976

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Robert B. Jansen, Executive Director

IDAHO FALLS, IDAHO

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UNITED STATES DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

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December 31, 1976

Honorable Thomas S. Kleppe
Secretary
U.S. Department of the Interior
Washington, D.C.

Honorable Cecil D. Andrus
Governor
State of Idaho
Boise, Idaho

Gentlemen:

Immediately following the failure of the Teton Dam on June 5, 1976, you joined in ordering an investigation and established for this purpose the Independent Panel to Review Cause of Teton Dam Failure. The Panel has completed its charge and herewith submits a report on its findings.

Pursuant to your instructions, the Panel has conducted a comprehensive study of the failure, including review of planning, design, construction, and operation of the dam and reservoir. Extensive field exploration, laboratory testing, and data analysis have been accomplished.

In briefest summary, the Panel concludes (1) that the dam failed by internal erosion (piping) of the core of the dam deep in the right foundation key trench, with the eroded soil particles finding exits through channels in and along the interface of the dam with the highly pervious abutment rock and talus, to points at the right groin of the dam, (2) that the exit avenues were destroyed and removed by the outrush of reservoir water, (3) that openings existed through inadequately sealed rock joints, and may have developed through cracks in the core zone in the key trench, (4) that, once started, piping progressed rapidly through the main body

Letter to Honorable Thomas S. Kleppe and Honorable Cecil D. Andrus

of the dam and quickly led to complete failure, (5) that the design of the dam did not adequately take into account the foundation conditions and the characteristics of the soil used for filling the key trench, and (6) that construction activities conformed to the actual design in all significant aspects except scheduling.

The Panel is hopeful that its findings will not only shed light on the Teton Dam failure, but will also assist in the design of safe dams at other sites.

Respectfully submitted,

Arthur Casagrande
Arthur Casagrande

Wallace L. Chadwick
Wallace L. Chadwick, Chairman

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Munson W. Dowd
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SUMMARY AND CONCLUSIONS

The Independent Panel to Review Cause of Teton Dam Failure has completed its task, as charged by the Secretary of the United States Department of the Interior and the Governor of the State of Idaho in letters from Secretary Kleppe, dated June 11, 23, and 30, 1976. The Panel submits its report herewith. These pages present a summary and conclusions.

Teton Dam failed on June 5, 1976, when the reservoir was at El. 5301.7, 3.3 ft below the spillway sill. Although downstream warnings are believed to have been timely, deaths of 14 persons and property damage estimated variously from 400 million to one billion dollars have been attributed to the failure.

Construction of Teton Dam was authorized on September 7, 1964, by Public Law 88-583. The dam is situated on the Teton River, three miles northeast of Newdale, Idaho. Prior to 1963, the proposed dam was known as Fremont Dam.

Teton Dam and its reservoir were principal features of the Teton Basin Project, a multipurpose project embodying flood control, power generation, and supplemental irrigation water supply. The dam was a central-core zoned earthfill structure, with a height of 305 ft above the riverbed and 405 ft above the lowest point in the foundation. Provisions for seepage control included a key trench in the foundation rock above El. 5100 and a cutoff trench to foundation rock below that elevation. A grout curtain extended below these trenches.

Investigations of site possibilities for a dam on the Teton River commenced as early as 1904 and continued at various times until bids for construction of Teton Dam were invited on July 22, 1971. A construction contract was awarded on December 13, 1971. The embankment was topped out November 26, 1975. Filling of the reservoir commenced October 3, 1975, and continued until the failure on June 5, 1976.

The Panel's approach to its assignment has been to:

- (1) obtain, analyze and evaluate all relevant information which could be obtained in document form from the United States Bureau of Reclamation, the United States Geological Survey, the construction contractor, and any other available and knowledgeable source regarding the regional and site geology, pre-siting investigations, siting decisions, pre-design investigations, design, contract specifications and drawings; construction practices, progress and inspections; in-progress changes, if any; pre-failure operation; mechanism of failure, including sworn eyewitness accounts; and actions of respective authorities during and immediately following failure;
- (2) supplement the documentary information by such further inquiry, including public hearings, as became necessary;
- (3) make (a) detailed studies of the post-failure condition of the dam, its auxiliary structures and its foundation, by inspection, dissection and subsurface drilling; (b) special tests on foundation materials; (c) detailed geologic maps and joint surveys; (d) tests of remnant materials; (e) detailed stress analyses; (f) studies of photographs for comparison of post-failure conditions with pre-construction and construction conditions; (g) measurements of post-failure geodetic positions of surface and subsurface points, as determined before failure and before filling of the reservoir;
- (4) contract with various organizations for special studies required by the Panel;

- (5) evaluate relevant data in order to sort out those of greatest significance in determining cause;
- (6) complete a report of the results of the foregoing activities prior to January 1, 1977.

The approach was initiated by telegrams, dated June 11 and June 14, 1976, to the Director, Engineering and Construction, U.S. Bureau of Reclamation, Denver, and by setting the Panel's first working session and its inspection of conditions at the site for the week of June 28-July 2, 1976. The telegrams requested information concerning (1) site geology in plan and sections with any test results on foundation materials; (2) site exploration with detail of drill logs, exploration trenches, borrow materials and tests; (3) grout records in detail showing non-average takes by location and depths, the patterns used and records of any interconnections; (4) foundation preparation showing both before and after conditions; (5) design memoranda for embankment, spillway, diversion structures and outlets; (6) basic drawings and technical specifications; (7) any outside report regarding the site or designs; (8) construction history of borrow pits, material preparation placement, progress, inspection, in-place tests; (9) seepage measurements or observations; (10) eyewitness accounts on progress of failure; (11) hydrology of the site; (12) seismicity of the site; (13) drain designs and drainage observations; (14) post-failure changes in spillway or auxiliary outlet structures; (15) any changes in precise level or horizontal control survey points; (16) changes in topography up and downstream; (17) photos of the foundation as approved at the start of embankment placement, particularly in the key trench and the cutoff trench; (18) record of any seeps or springs in the cutoff and core contact area; and (19) records of cofferdam seepage and pumpage from the foundation area.

Prior to the Panel's convening for its first session, the Department of the Interior had recorded sworn testimony of 37 eyewitness observers of pre-failure and during-failure conditions, of whom 14 were Bureau of Reclamation staff and employees, 13 were employees of the construction contractor, and 10 were from the general public. In parallel with these eyewitness accounts, there became available several excellent photographic sequences in still and later in motion picture form. In order to supplement these eyewitness accounts with any available observations of failure-related, but pre-failure conditions, a public call was issued, and two public hearings were held in Idaho Falls on July 21, 1976.

During its first working session, the need for professional staff and technical and administrative support was recognized. To fill this need, the services of Robert B. Jansen, as Executive Director, were secured through the cooperation of the Governors of Idaho and California. Also, the services of Clifford Cortright, Staff Engineer, and Larry James, Staff Geologist, and Frank Sherman, also a staff geologist, were secured within a few days of Mr. Jansen's appointment. Through the excellent cooperation of the Office of the Secretary, Department of the Interior, supporting properties, services, technicians and administrative assistance have been made available to the Panel through various bureaus of the Department.

Because of the importance of determining existing embankment and foundation conditions, the Panel early addressed the Director, Design and Construction, USBR, Denver, requesting specific work on the right abutment to permit detailed examination of the remnant there, and excavation to uncover both the auxiliary outlet tunnel for internal inspection and the site of the large, lower spring observed early on June 5, 1976.

Response was prompt, and on July 16, 1976, the Bureau of Reclamation awarded its Contract No. DC-7232 to Gibbons and Reed, Salt Lake City, to cover the required work. Actual dissection of the right remnant of the dam started July 26, 1976. This excavation proceeded expeditiously, in five-foot vertical increments, to El. 5200, with trenching in each incremental level to allow taking of samples as

well as inspection of the core remnant for any evidence of water channeling, or cracking, and of the manner in which the key trench was excavated, sealed, filled and compacted.

The Bureau's response to the Panel's request for records, data and descriptions was also prompt. A large volume of information was furnished. Many of these records have been supplemented by others furnished to the Panel's staff at the site in response to oral and written requests.

Further information was sought on the manner in which the grout curtains were closed and in which the core was built into the key trench. This information was desired both from the Bureau of Reclamation as designers and construction engineers of the dam, and of the contractor, who implemented that construction. Accordingly, a questionnaire was directed concurrently to the Director, Design and Construction, USBR, and to the Chief Executive Office of Morrison-Knudsen, as the sponsoring member of the constructing contractor, Morrison-Knudsen-Kiewit.

The USBR response was quite complete. The contractor's response is in two parts. One is from the prime contractor per se, and the other is from the grouting sub-contractor, McCabe Bros., Inc. The prime contractor's answer was rather general.

Staff investigations started immediately upon appointment of the various staff members. Their efforts have been interested, diligent, competent, and tireless. They have greatly expedited the completion of the Panel's task and the compilation of its report. The Panel met in Denver on June 28 and 29 to organize and initiate its inquiry through information presentations by the Bureau of Reclamation. A site inspection was made on June 30. Information meetings were held with the Bureau engineers at the site during the following day. Working sessions were continued in Denver on July 2. The Panel met again in working session in Idaho Falls August 3 through 5, again October 5 through 7, November 1 through 3, and December 7 through 10. Between its working sessions, individual Panel members worked with the staff, and independently on assignments from the Panel. Frequent individual visits were made to the site exploratory work.

Careful study was made of all eyewitness accounts of their observations prior to the breach. All available photographs of the failure events were studied and arranged in chronologic sequence. All available relevant documentary records have been reviewed for significant content. Continuous professional examination was conducted of all trenching in the right abutment embankment remnant. Detailed mapping of the bedrock joints and fractures in and adjacent to the right abutment key trench was conducted between Stas. 11+00 and 16+00. Laboratory testing of undisturbed samples of Zone 1 (core) material was carried out. Subsurface water loss tests were conducted at many locations near the centerline right abutment grout curtain. Surface ponding tests were conducted at the key trench invert at prominent joints crossing the invert. Hydraulic fracturing tests were made in drill holes in the left abutment core remnant. Analytic studies were made to assess the stress conditions on sections of the embankment and key trench in the zone of failure.

The Panel's conclusions are summarized below:

1. The records show that the pre-design site selection and geological studies were appropriate and extensive. The pilot grouting program carried out in 1969 forecast the difficulties to be experienced in construction of the final grout curtain.
2. The design followed USBR practices, developed over a period of many years from experience with other Bureau projects, but without sufficient consideration of the effects of differing and unusually difficult geological conditions at the Teton Damsite. Every embankment can be said to

have its own personality requiring individual design consideration and construction treatment. Treatment of such individualities produces most of the continuing advances in dam design and construction technology.

3. The volcanic rocks at the Teton Damsite are highly permeable and moderately to intensely jointed. Water was therefore free to move with almost equal ease in most directions, except locally where the joints had been effectively grouted. Thus during reservoir filling, water was able to move rapidly to the foundation of the dam. Open joints existed in the upstream and downstream faces of the right abutment key trench, providing potential conduits for ingress or egress of water.

4. The wind-deposited nonplastic to slightly plastic clayey silts used for the core and key trench fill are highly erodible. The Panel considers that the use of this material adjacent to the heavily jointed rock of the abutment was a major factor contributing to the failure.

5. Construction of the project was carried out by competent contractors under formal contracts administered in accord with well-accepted practices. Controversy between the contractors and Bureau of Reclamation officials which might have affected the quality of the work seems not to have occurred. Construction activities conformed to the actual design in all significant aspects except scheduling.

6. One construction condition which affected the Bureau's ability to control the rate of filling of the reservoir was the delay that occurred in completion of the river outlet works. However, the Panel believes that the conditions which caused the piping and consequent failure of the dam were not materially affected by the fact that the reservoir was filled at a more rapid rate than had been originally planned. A slower rate of filling would have delayed the failure but, in the judgment of the Panel, a similar failure would have occurred at some later date.

7. The records show that great effort was devoted to constructing a grout curtain of high quality, and the Panel considers that the resulting curtain was not inferior to many that have been considered acceptable on other projects. Nevertheless, the Panel's on-site tests and other field investigations showed that the rock immediately under the grout cap, at least in the vicinity of Stas. 13+00 to 15+00, was not adequately sealed, and that additional unsealed openings may have existed at depth in the same locality. The leakage beneath the grout cap was capable of initiating piping in the key trench fill, leading to the formation of an erosion tunnel across the base of the fill. The Panel considers that too much was expected of the grout curtain, and that the design should have provided measures to render the inevitable leakage harmless.

8. The geometry of the key trenches, with their steep sides, was influential in causing transverse arching that reduced the stresses in the fill near the base of the trenches and favored the development of cracks that would open channels through the erodible fill. Arching in the longitudinal direction, due to irregularities in the base of the key trenches, and arching adjacent to minor irregularities and overhangs, undoubtedly added to the reduction of stress.

9. Stress calculations by the finite element method indicated that, at the base of the key trench near Stas. 14+00 and 15+00, the arching was great enough that the water pressure could have exceeded the sum of the lateral stresses in the impervious fill and the tensile strength of the fill material. Thus, cracking by hydraulic fracturing was a theoretical possibility and may have led to flow of water in the base of the key trench between Stas. 14+00 and 15+00, and erosion of the key trench fill.

10. Close examination of the interior of the auxiliary outlet tunnel showed no distress of any kind such as would be expected had the right abutment, through which the tunnel passes, been subjected to significant settlement or other structural change. Geodetic resurveys showed only minor surface movements as a result of reservoir filling and emptying. Accordingly, differential movements of the foundation are not considered to have contributed to the failure.

11. The Panel found no evidence that seismicity was a factor in failure of the dam.

12. The dam and its foundations were not instrumented sufficiently to enable the Project Construction Engineer and his forces to be informed fully of the changing conditions in the embankment and its abutments.

13. Following its first working session, the Panel reported that it then seemed apparent 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 section. That conclusion remains valid. The Panel's investigations since that time have been directed particularly to determining the most probable manner in which such piping erosion started. The Panel believes that two mechanisms are suspect. Either could have worked alone or both could have worked together. 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 near Sta. 14+00 and the consequent development of an erosion tunnel across the base of the key trench fill. The other is cracking caused by differential strains or hydraulic fracturing of the core material filling the key trench. This cracking would also result in channels through the key trench fill which would permit rapid internal erosion.

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.

14. 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.

The difficult conditions of the site called for basing the design on the most unfavorable assumptions compatible with the geologic conditions concerning the behavior of the water and its possible effect on the embankment. Instead of placing so much dependence on the key trenches and grout curtain, measures should have been developed to render harmless whatever water did pass, irrespective of the reasons.

In final summary, under difficult conditions that called for the best judgment and experience of the engineering profession, an unfortunate choice of design measures together with less than conventional precautions was taken to ensure the adequate functioning of the Teton Dam, and these circumstances ultimately led to its failure.

FOREWORD

As a basis for this report, the Independent Panel to Review Cause of Teton Dam Failure read and evaluated a large volume of documents, records, and data, the larger part of which was obtained from the United States Bureau of Reclamation, the designing and constructing agency for the Teton Dam. Additionally, the Panel carried out numerous independent inquiries; and through contracts, administered by the Department of the Interior, conducted detailed exploratory excavations in the right bank remnant of embankment which survived the failure. All of that remnant was dissected, seeking evidences of cause of failure.

Exploratory core drilling was done for the Panel, seeking better to evaluate subsurface foundation conditions and adequacy of foundation grouting. Independent geological mapping was carried out, particularly of the bedrock foundation joint systems. Physical testing was done on undisturbed samples of fill material from the remnant of the dam on the right abutment. Using several laboratories, additional tests were made of the characteristics of the foundation materials. Analytical studies were made to permit estimating the stresses within the embankment. The results of some of the investigations carried out for a separate investigative unit identified as the U.S. Department of the Interior Teton Dam Failure Review Group, under the chairmanship of Mr. Dennis Sachs, have been supplied to the Independent Panel, and have been carefully evaluated. Also the results of the field and laboratory studies conducted by the Independent Panel were made available to the Interior Group.

An effort was made by the Independent Panel to evaluate all available, relevant information. To all of this information the Panel has applied its best professional judgment, and it is satisfied that its conclusions regarding the cause of failure are sound. However, to permit others to reach their own judgments concerning the Panel's findings, the Panel has attempted to list and reference, as much as has been practicably possible, the principal source information upon which it relied in making its judgments.

The Panel is grateful to the Secretary of the Interior and to his office, and to the Governor of Idaho and his office, for their support and understanding throughout the Panel's review. The administrative counsel and aid received from the Secretary's office has maximized the time available for the technical and analytical work of the Panel. The cooperation which the Panel received from all levels of the Bureau of Reclamation has been of great assistance.

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ABBREVIATIONS

acre-ft	acre-foot
a.m.	midnight till noon
AOW	auxiliary outlet works
avg	average
cfs	cubic feet per second
cm	centimeter
cu ft	cubic foot
cu in.	cubic inch
cu yd	cubic yard
DH	drill hole
diam	diameter
El.	elevation (always in feet)
Fig.	Figure
ft	foot
gpm	gallons per minute
in.	inch
kip	1,000 pounds
km	kilometer
ksf	kips per square foot
kw	kilowatt
lb	pound
max	maximum
min	minimum
M.M.	Modified Mercalli scale of earthquake intensities
NP	non-plastic
PI	plasticity index
p.m.	noon to midnight
pcf	pounds per cubic foot
psf	pounds per square foot
psi	pounds per square inch
ROW	river outlet works
rpm	revolutions per minute
sec	second
sq ft	square foot
Sta.	Station
T	ton
yr	year

TERMINOLOGY

c'	Effective stress cohesion parameter
\mathcal{C}	Centerline
CH) GP) GW) MH) ML)	Soil classification according to Unified Soil Classification Chart. (See Fig. 3-7, "Earth Manual," USBR.)
DH-651AB	Numerals designate drill hole number. DH-651A indicates that DH-651 was abandoned and a new start undertaken. DH-651B designates the third start at the same drill site. DH-651AB refers to all three holes. Thus the log DH-651AB is a composite log containing information obtained in the course of drilling DH-651, DH-651A and DH-651B.
E	Modulus of deformation
ϵ	Unit strain
E_s	Secant modulus of elasticity
g	Acceleration of gravity
k	Coefficient of permeability
K	Modulus number
K_o	Ratio of horizontal principal stress (σ_3) to vertical principal stress (σ_1).
k_{av}	Average coefficient of permeability
lineament	A line on an aerial photo that is structurally controlled. The term is widely applied to lines representing sedimentary beds, faults, joints, and rock boundaries.
Left, i.e., left abutment, left bank, etc.	Refers to that side of the river channel to the viewer's left when he is facing downstream.
μ	Poisson's ratio
M_L	Earthquake magnitude based on measurement of the amplitude of surface (Love) waves

TERMINOLOGY (cont.)

Nx drill hole	A 2.98-in. diam hole drilled or cored in rock. Similarly, Bx, Ax, and Ex holes are respectively 2.38 in., 1.89 in., and 1.485-in. diam.
s_x	Standard deviation
$\tan \phi'$	Effective stress friction parameter
2:1	Slope designation, horizontal units to vertical units

ACKNOWLEDGEMENTS

The Panel gratefully acknowledges the fine help it has received from many individuals during its review. Accepting the risk that some who should be mentioned may be overlooked, special thanks are due Secretary Thomas S. Kleppe for his own and his staff's support and assistance, including Under Secretary Kent Frizzel; Executive Assistant Loren J. Rivard; Assistant Secretary Albert L. Zapanta; Assistant Secretary Jack O. Horton; Deputy Assistant Secretary Richard R. Hite; Deputy Assistant Secretary Dennis N. Sachs; Justin P. Patterson, Assistant Solicitor for Procurement; and John R. Little, Jr., Regional Solicitor in Denver.

The Panel also is very grateful to Governor Cecil Andrus for his aid, particularly in talking with Governor Edmund G. Brown, Jr., of California to obtain Mr. Jansen's leave of absence to assist with the review. Panel member Keith Higginson has helpfully maintained a continuing contact with Governor Andrus and his office.

The interest and capable assistance of James F. Kelly, Director, Office of Administration and Management Policy; James E. Johnson, Chief, Procurement and Grants; William S. Opdyke, Procurement Analyst, Department of the Interior; and M.S. Greenlee, Chief, Property and Purchasing Branch, Engineering and Research Center, Denver, have been especially helpful by freeing the Panel of many contract and administrative tasks which it otherwise would have been required to perform personally.

The fine support of the Bonneville Power Administration, the National Park Service, including the Division of Graphic Systems, the Bureau of Indian Affairs, the Government Printing Office, and the United States Geological Survey in supplying technical and administrative people and services has been greatly appreciated. These people responded to the Panel's need with interest and diligence.

The Bureau of Reclamation responded to the Panel's many needs for data and information promptly and with candor and interest. Special thanks are due Harold G. Arthur, Donald J. Duck, Sammie D. Guy, and others in the Bureau's Denver organization. Sam Rey from Grand Coulee and D.D. McClure from Boise were especially effective in getting the office set up and supplied in Idaho Falls and at the damsite. Thayne O'Brien continued that fine service.

At the damsite the help and assistance of Robert R. Robison, Project Construction Engineer, have been candid and fully responsive. This cooperation has been duplicated by all of the Bureau's on-site staff, particularly by Pete Aberle, Stanley White, Dan Magleby, Ralph Mulliner, and Ken Hoyt. The cooperation of Brent Carter of the Regional Office was appreciated.

The work of Gibbons and Reed under Superintendent George Tackett in carrying out the dissection and other work on the right abutment was capable, responsive, and interested.

Important laboratory and analytical investigations in support of the Panel's work were also carried out by Northern Testing Laboratories, Geo-Testing, Inc., J.M. Duncan and G. Jaworski of the University of California at Berkeley, R.E. Bieber of Dynamic Analysis Corporation, K. Arulanandan of the University of California at Davis, the U.S. Bureau of Reclamation laboratories at Denver and at the damsite, and the Waterways Experiment Station at Vicksburg.

ExhibiGraphics Group was especially helpful in its expeditious construction of the model of the dam.

This report would not be in being, however, except for the special competence and untiring dedication of Robert B. Jansen, Executive Director, and other professional staff, namely Clifford J. Cortright, Staff Engineer, Laurence B. James, Staff Geologist, and Frank B. Sherman, geologist. Jacque Steele well coordinated the Idaho Falls office and kept an immense number of drafts of texts flowing smoothly under considerable pressure.

CHAPTER 1 INTRODUCTION

Construction of Teton Dam was authorized on September 7, 1964, by Public Law 88-583. The dam is situated on the Teton River, three miles northeast of Newdale, Idaho, as shown on Fig. 1-1. Prior to 1963, the proposed Teton Dam was known as Fremont Dam, and some records remain under that name.

The Teton Dam and Reservoir are the principal features of the Teton Basin Project, a multipurpose project, which when completed was to serve the objectives of flood control, power generation, recreation, and supplemental irrigation water supply for 111,250 acres of farm land in the Upper Snake River Valley. Appurtenant features of the dam are (1) a 16000 kw generating and pumping plant on the left bank, (2) river outlet works and a gate chamber shaft on the left bank, (3) auxiliary outlet works and an access shaft in the right bank, (4) a three-gate chute-type spillway on the right bank, and (5) the 72-in. Enterprise-East Teton Feeder Pipeline and Canal Outlet Works Control Structure on the left bank. In this report, right and left designations are made looking downstream. The reservoir had a total capacity of 288,000 acre-ft. It extended 17 miles upstream, and had a surface area of 2100 acres.

The dam was a central-core zoned earthfill structure, whose maximum section rose 305 ft above the original valley floor, and 405 ft above the lowest point in the foundation. The plan and cross section of the dam are shown in Fig. 1-2, from the construction contract documents. Embankment details are shown in Fig. 1-3, from the contract documents. This drawing (Fig. 1-3) includes the foundation design grouting patterns. Details of auxiliary outlet works, river outlet works, and spillway are provided in Figs. 1-4, 1-5, and 1-6, respectively. The crest elevation before camber was 5332, and its length was about 3,100 ft. The volume of the embankment was about 10 million cu yds.

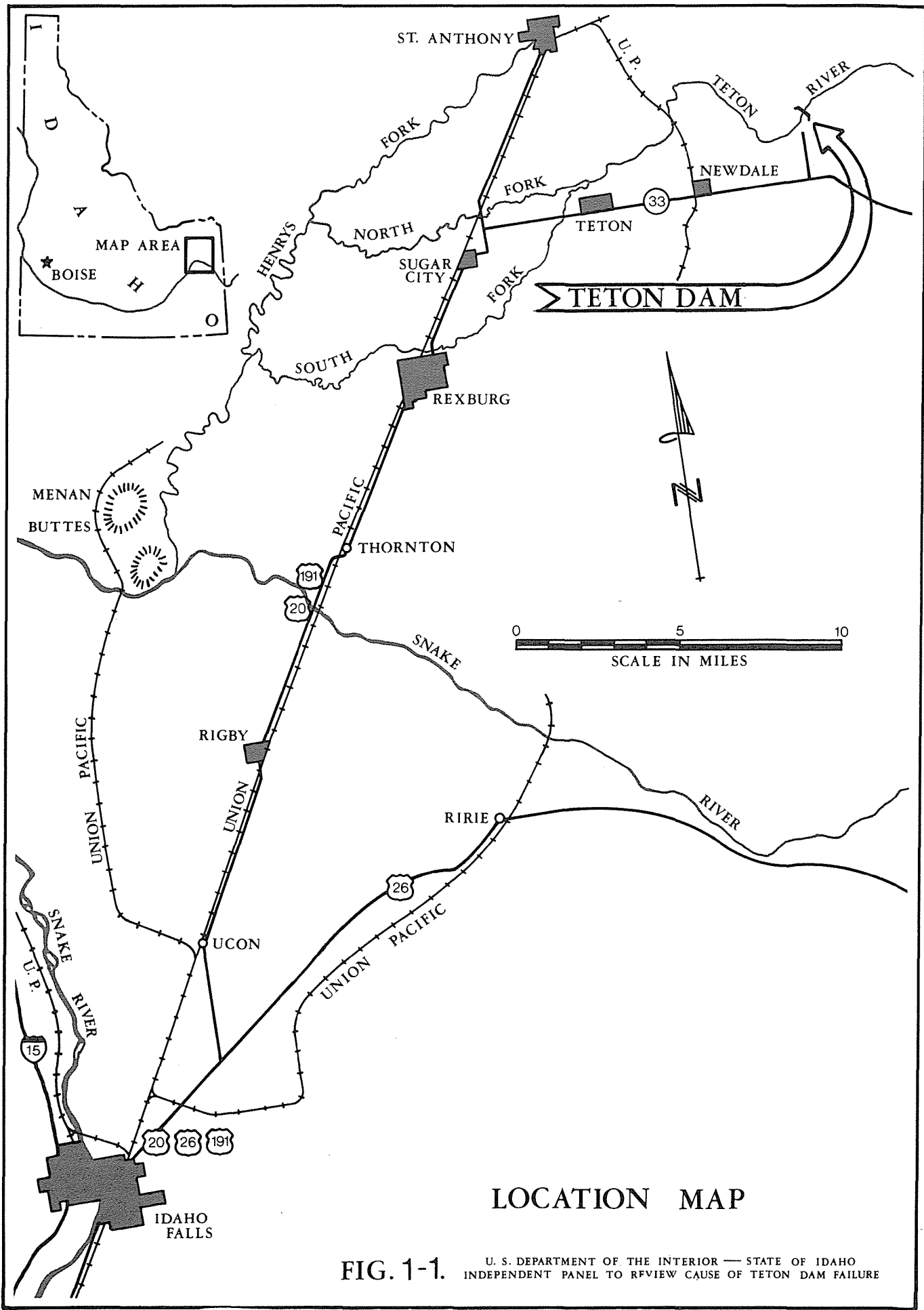
At the damsite the Teton River occupies a steep-walled canyon incised in rhyolitic ash-flow tuff, a hard rock derived from distant volcanoes now long extinct. Extensive joints are common in this rock and are particularly numerous near the surface of the abutment. An important feature in the dam is a key trench excavated through this highly jointed surficial layer and later backfilled with embankment to provide a barrier to reservoir leakage.

The construction contract was awarded December 13, 1971, and work commenced in February 1972. The embankment was topped out November 26, 1975. Closure for storage occurred October 3, 1975.

The dam failed on June 5, 1976, when the reservoir water level was at El. 5301.7. That level was 22.6 ft below the maximum water level and 3.3 ft below the spillway sill.

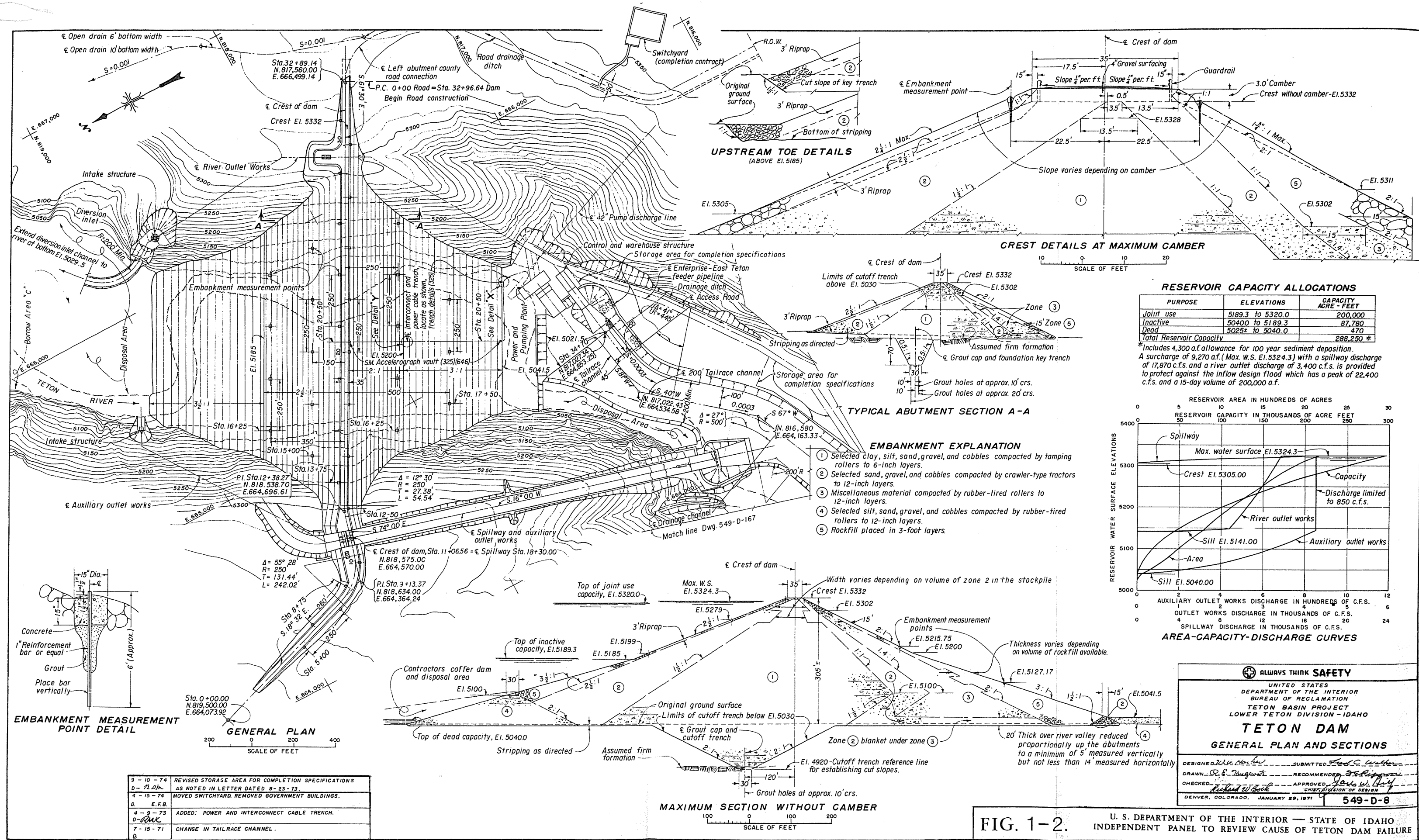
The inundation downstream was disastrous, and the loss of the lives of 14 persons has been associated directly or indirectly with the failure. Property damage has been estimated at various amounts from \$1 billion downward. The current best estimate seems to be \$400 million. The area which was inundated has been shown on maps by the Corps of Engineers, U.S. Army, and by the United States Geological Survey. The outlines mapped by the latter are shown on Fig. 1-7.

Immediately following the Teton Dam failure, the Governor of Idaho and the Secretary of the Interior agreed on the need for an independent engineering and geological review of the cause of that failure.



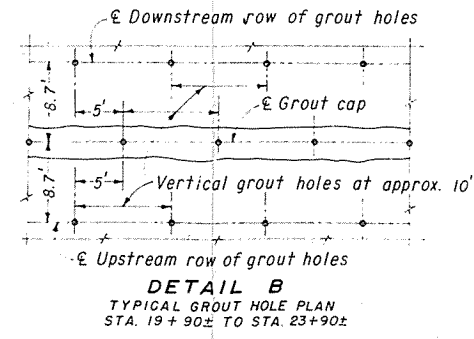
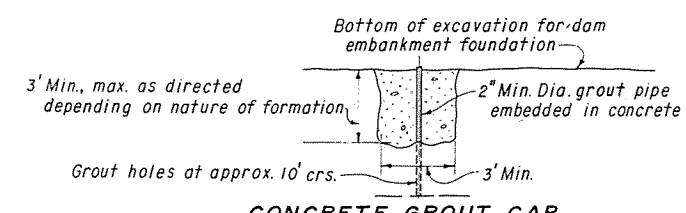
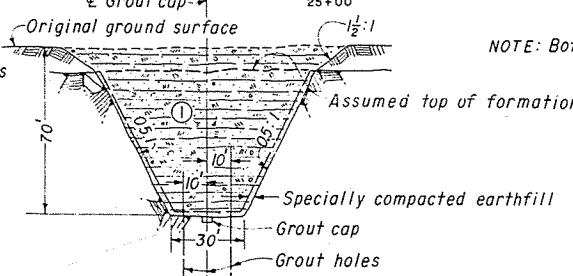
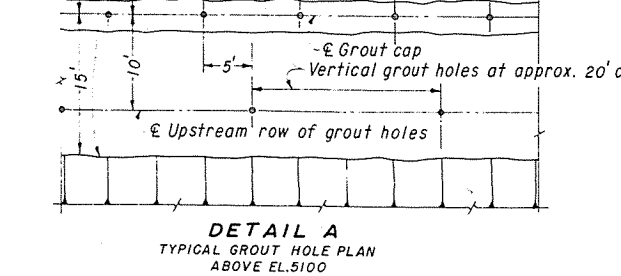
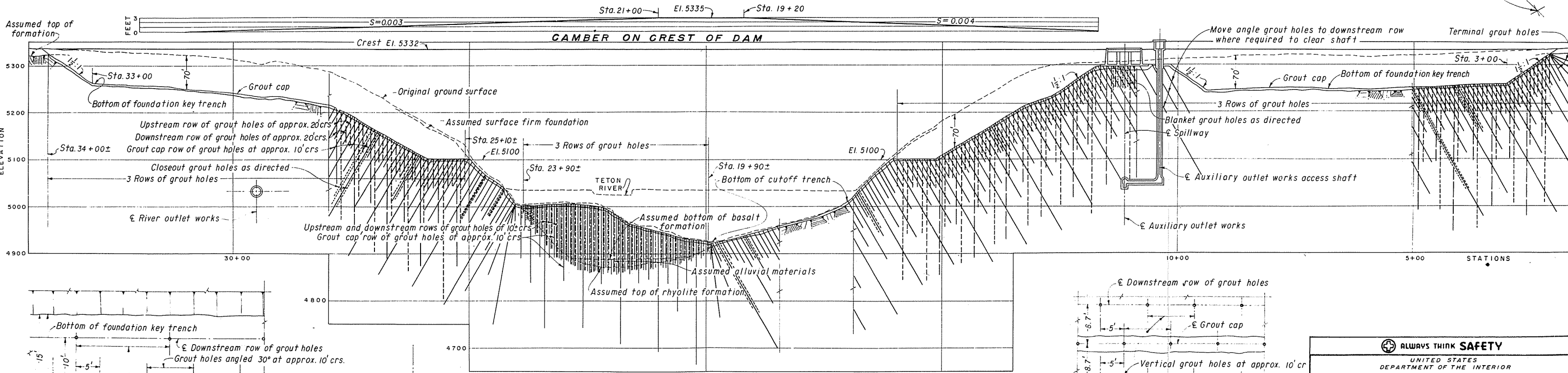
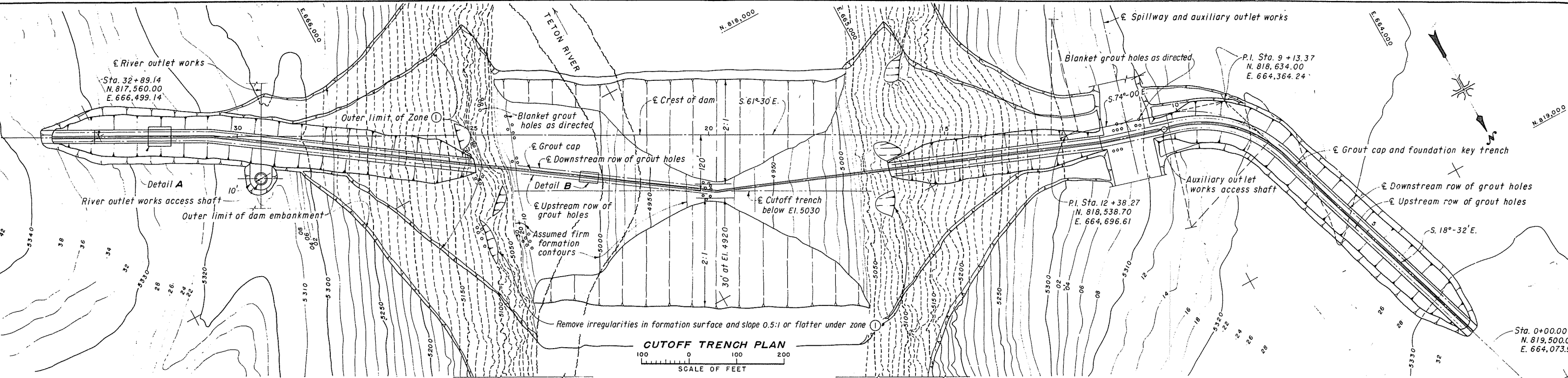
LOCATION MAP

FIG. 1-1. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



9-10-74	REVISED STORAGE AREA FOR COMPLETION SPECIFICATIONS AS NOTED IN LETTER DATED 8-23-73.
0-12-74	
4-15-74	MOVED SWITCHYARD. REMOVED GOVERNMENT BUILDINGS.
D.	E.F.B.
4-9-73	ADDED: POWER AND INTERCONNECT CABLE TRENCH.
0-12-73	
7-15-71	CHANGE IN TAILTRACE CHANNEL.
D.	

U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
 INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



PROFILE ON CREST OF DAM
NOTE: Bottom of foundation excavation is projected from grout cap.

DETAIL B
TYPICAL GROUT HOLE PLAN
STA. 19+90± TO STA. 23+90±

ALWAYS THINK SAFETY

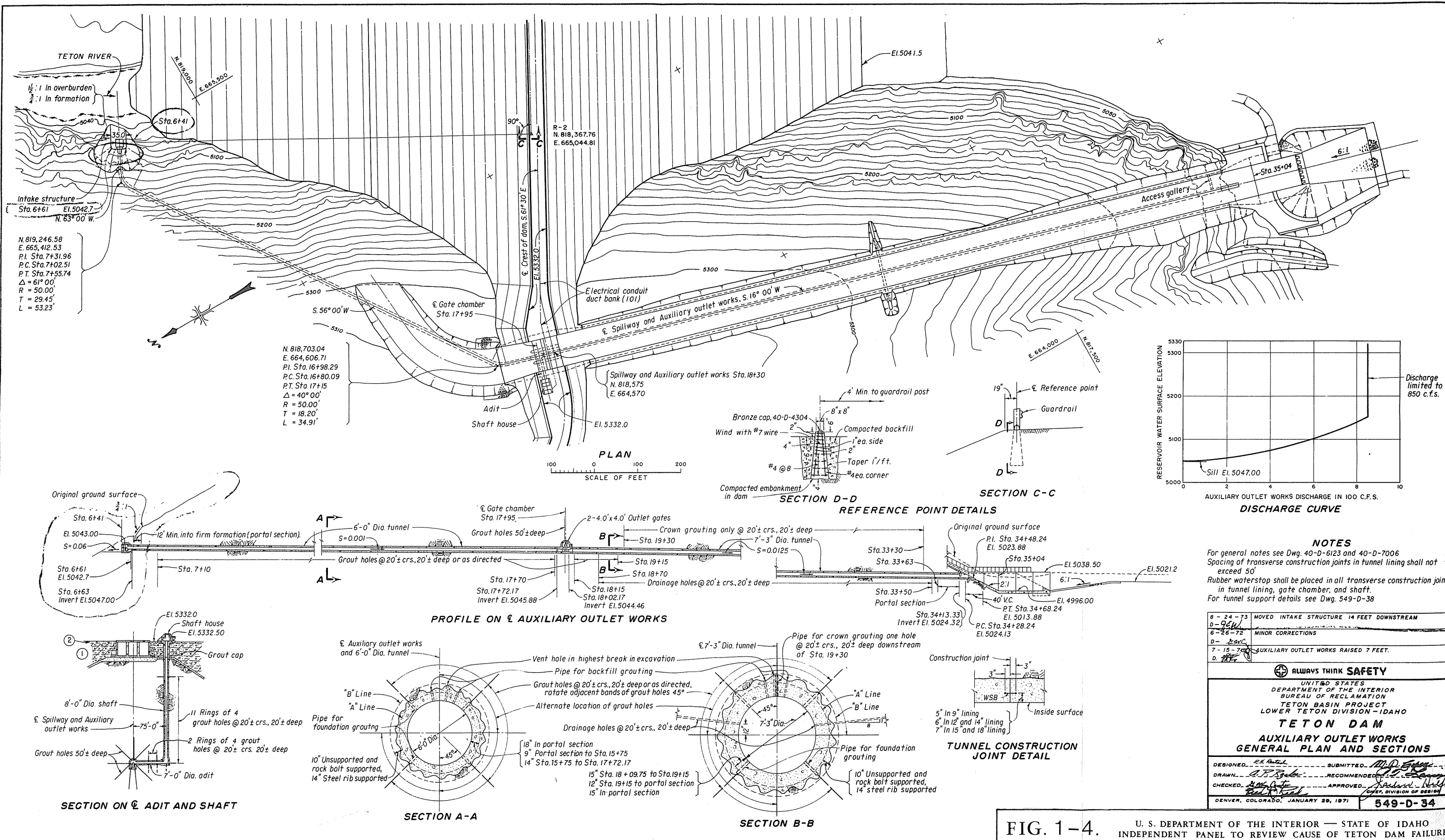
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TETON BASIN PROJECT
LOWER TETON DIVISION - IDAHO

TETON DAM
EMBANKMENT DETAILS

DESIGNED: *[Signature]* SUBMITTED: *[Signature]*
DRAWN: *[Signature]* RECOMMENDED: *[Signature]*
CHECKED: *[Signature]* APPROVED: *[Signature]*
CHIEF DIVISION OF DESIGN

DENVER, COLORADO, JANUARY 22, 1971 **549-D-9**

FIG. 1-3. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



NOTES

For general notes see Dwg. 40-D-6123 and 40-D-7006
 Spacing of transverse construction joints in tunnel lining shall not exceed 50'
 Rubber waterstop shall be placed in all transverse construction joints in tunnel lining, gate chamber, and shaft.
 For tunnel support details see Dwg. 549-D-38

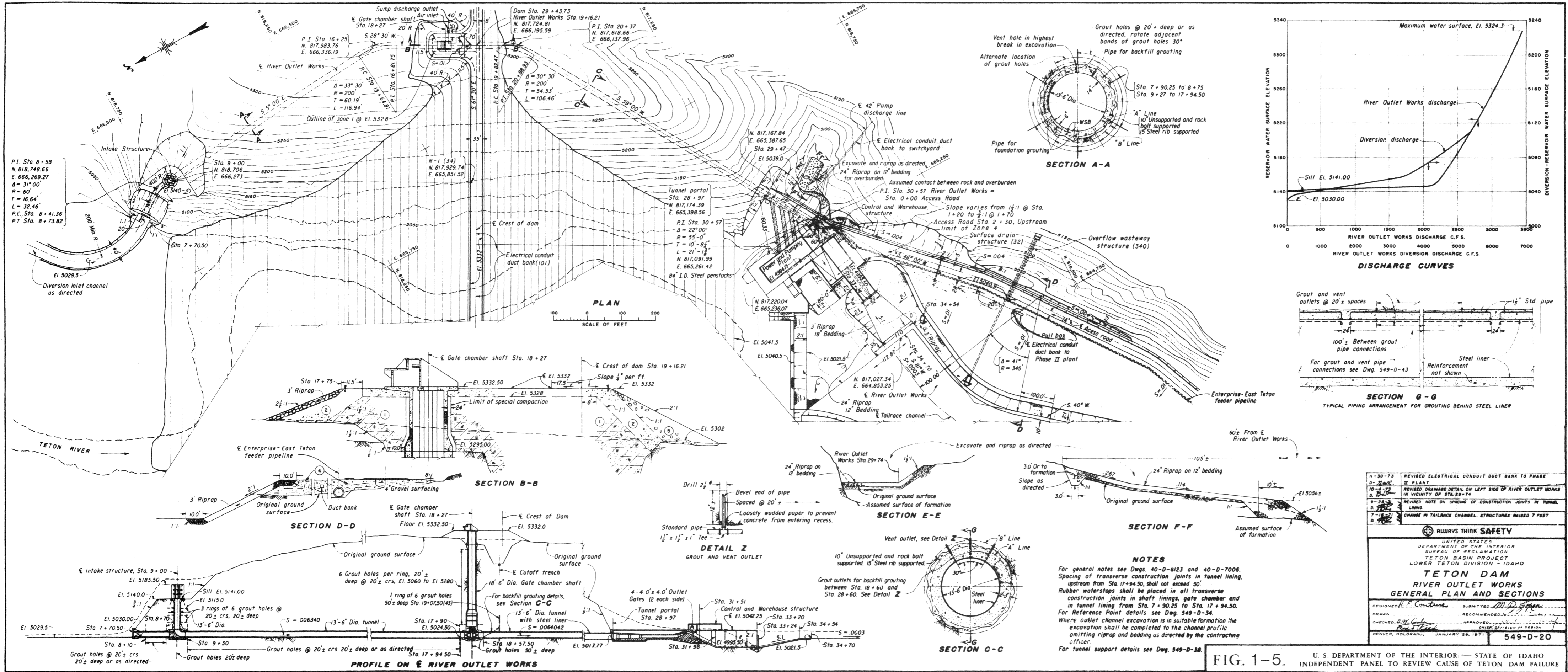
8-24-73	MOVED INTAKE STRUCTURE 14 FEET DOWNSTREAM
D-9-26-72	MINOR CORRECTIONS
6-26-72	
D-2-26-72	
7-15-72	AUXILIARY OUTLET WORKS RAISED 7 FEET.
D-1-26-72	

ALWAYS THINK SAFETY

UNITED STATES
 DEPARTMENT OF THE INTERIOR
 BUREAU OF RECLAMATION
 TETON BASIN PROJECT
 LOWER TETON DIVISION - IDAHO
TETON DAM
AUXILIARY OUTLET WORKS
GENERAL PLAN AND SECTIONS

DESIGNED: *E.K. Smith* SUBMITTED: *M.D. Egan*
 DRAWN: *A.T. Rye* RECOMMENDED: *E.L. ...*
 CHECKED: *...* APPROVED: *...*
 DIVISION OF DESIGN
 DENVER, COLORADO, JANUARY 29, 1971 **549-D-34**

FIG. 1-4. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
 INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



11-30-73	REVISED ELECTRICAL CONDUIT DUCT BANK TO PHASE II PLANT.
10-4-73	REVISED DRAINAGE DETAIL ON LEFT SIDE OF RIVER OUTLET WORKS IN VICINITY OF STA. 28+74.
9-28-73	REVISED NOTE ON SPACING OF CONSTRUCTION JOINTS IN TUNNEL LINING.
7-18-73	CHANGE IN TAILRACE CHANNEL STRUCTURES RAISED 7 FEET.

ALWAYS THINK SAFETY

UNITED STATES DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TETON BASIN PROJECT
LOWER TETON DIVISION - IDAHO

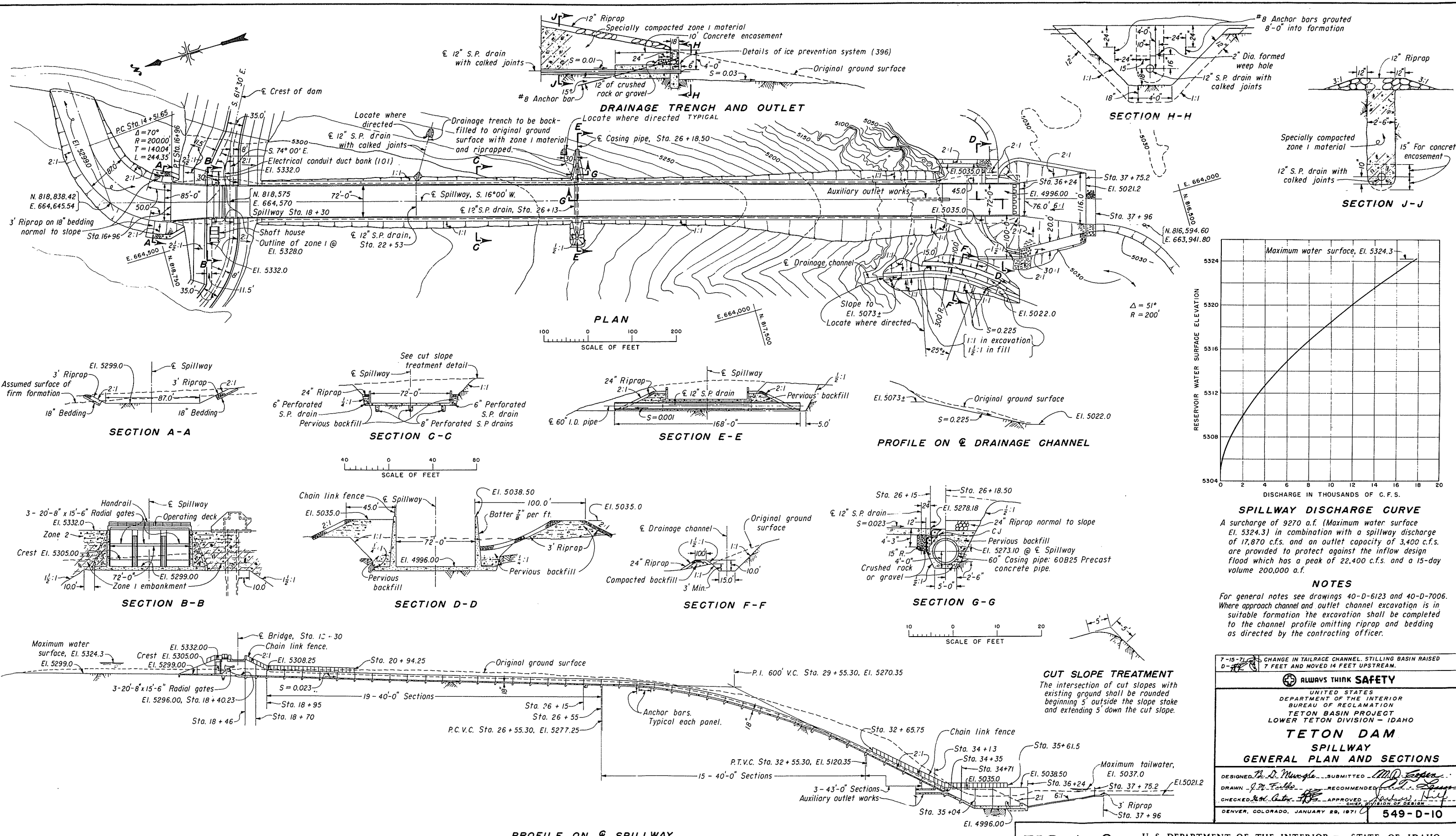
TETON DAM RIVER OUTLET WORKS GENERAL PLAN AND SECTIONS

DESIGNED: *[Signature]* SUBMITTED: *[Signature]*
DRAWN: *[Signature]* RECOMMENDED: *[Signature]*
CHECKED: *[Signature]* APPROVED: *[Signature]*
DENVER, COLORADO, JANUARY 29, 1971

NOTES

For general notes see Dwg. 40-D-6123 and 40-D-7006. Spacing of transverse construction joints in tunnel lining, upstream from Sta. 17+94.50, shall not exceed 50'. Rubber waterstops shall be placed in all transverse construction joints in shaft linings, gate chamber and in tunnel lining from Sta. 7+90.25 to Sta. 17+94.50. For Reference Point details see Dwg. 549-D-34. Where outlet channel excavation is in suitable formation the excavation shall be completed to the channel profile omitting riprap and bedding as directed by the contracting officer. For tunnel support details see Dwg. 549-D-38.

FIG. 1-5. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



SPILLWAY DISCHARGE CURVE
 A surcharge of 9270 a.f. (Maximum water surface El. 5324.3) in combination with a spillway discharge of 17,870 c.f.s. and an outlet capacity of 3,400 c.f.s. are provided to protect against the inflow design flood which has a peak of 22,400 c.f.s. and a 15-day volume 200,000 a.f.

NOTES
 For general notes see drawings 40-D-6123 and 40-D-7006. Where approach channel and outlet channel excavation is in suitable formation the excavation shall be completed to the channel profile omitting riprap and bedding as directed by the contracting officer.

7-15-71 CHANGE IN TAILRACE CHANNEL. STILLING BASIN RAISED 7 FEET AND MOVED 14 FEET UPSTREAM.

ALWAYS THINK SAFETY

UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION TETON BASIN PROJECT LOWER TETON DIVISION - IDAHO

TETON DAM SPILLWAY GENERAL PLAN AND SECTIONS

DESIGNED *P. D. Mungle* SUBMITTED *M. D. Fisher*
 DRAWN *J. P. Fiddle* RECOMMENDED *R. J. Lewis*
 CHECKED *S. H. Carter* APPROVED *W. C. ...*
 DENVER, COLORADO, JANUARY 28, 1971

FIG. 1-6. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

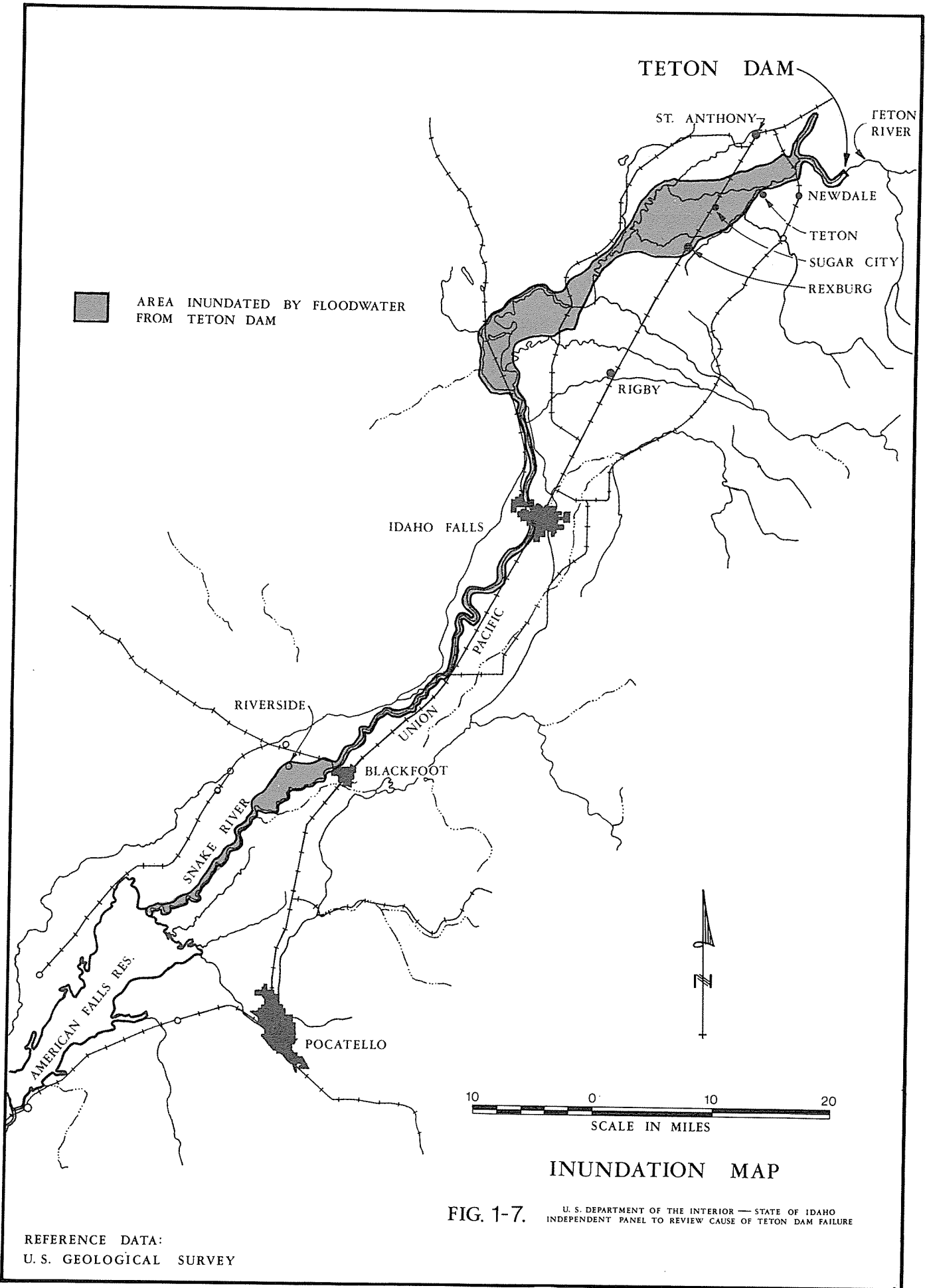


FIG. 1-7.

U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Candidates for a panel of review were considered from lists suggested by the National Academy of Engineering, the National Academy of Science, and other organizations.

The selection of the Panel members was confirmed in appointment letters signed by the Secretary on June 11, June 23, and June 30, 1976. Copies of these letters are included in Appendix B. The June 30 letter states the Panel's charge.

As a result of the various consultations, the Independent Panel to Review Cause of Teton Dam Failure was established with membership as follows:

WALLACE L. CHADWICK, Chairman, former president of the American Society of Civil Engineers, and a consultant on water projects and dam safety.

ARTHUR CASAGRANDE, Professor Emeritus at Harvard University and engineering consultant on dams and foundations.

HOWARD A. COOMBS, Professor Emeritus of Geology at the University of Washington, and consulting geologist on dams and power projects.

MUNSON W. DOWD, Chief Engineer of the Metropolitan Water District of Southern California.

E. MONTFORD FUCIK, Chairman of the Board, Harza Engineering Company, and designer and consultant on major dams.

R. KEITH HIGGINSON, Director of the Department of Water Resources, State of Idaho.

THOMAS M. LEPS, consulting engineer on major water projects and a member of the consulting board retained by the State of California to investigate the failure of the Baldwin Hills Dam in southern California in 1963.

RALPH B. PECK, consulting engineer, Professor Emeritus of Foundation Engineering, University of Illinois at Urbana, and recipient of the National Medal of Science from President Ford in 1975.

H. BOLTON SEED, Professor of Civil Engineering at University of California at Berkeley, a member of the California Seismic Safety Commission, and a consultant on seismic design of embankment dams and on nuclear power plants.

Organizational work started with telegraphic requests to the Director, Design and Construction, United States Bureau of Reclamation, for specific information and data. These requests were dated June 11 and June 14, 1976. Copies are included in Appendix B. Subsequently, the office of the Secretary of the Interior, working with the Panel Chairman, undertook a detailed plan for administering the Panel's work. Several alternatives were considered, and the respective advantages and disadvantages weighed. In the interest of best preserving complete independence for the Panel, a contract plan of organization and administration was accepted wherein the Panel Chairman became a prime contractor, and the other panel members and professional staff became subcontractors, with the contract obligation to prepare a series of interim and progress reports, and a main report.

The intent of the "no fee" contract was contained in a letter dated July 16, 1976. The definitive contract was also dated July 16, 1976, but was finalized on October 4, 1976. However, because of the urgency of its task, the Panel actively pursued its purpose from the time of its appointment.

On June 30, 1976, the Panel was able to retain as its Executive Director, Robert B. Jansen, a civil engineer who was formerly Chief, Division of Safety of Dams, and Deputy Director of Water Resources in California. Mr. Jansen served as chairman of the State Engineering Board of Inquiry in the Baldwin Hills Dam failure investigation in 1963 and is on leave of absence through the cooperation of the Governor of California.

On July 3, 1976, selection of the principal members of the Panel staff was completed by the assignment of consulting engineer Clifford J. Cortright as Staff Engineer, and consulting geologist Laurence B. James as Staff Geologist. Mr. Cortright is former chief of the Division of Safety of Dams, and of the Division of Design and Construction of the California Department of Water Resources. Mr. James was until recently Chief Engineering Geologist for the California Department of Water Resources.

Other members of the Panel staff, on special assignment from their regular government positions, were Frank B. Sherman, geologist from the Idaho Department of Water Resources; technicians Romeo Singson, Thomas E. Chinn, and Doug Burkhard from the National Park Service; technicians Dwight P. Berger and Clifford L. Cole, draftsman Robert E. McDonald, and office assistant Jacque J. Steele from the Bonneville Power Administration; and draftsmen John Carillo and Robert M. Nilchee from the Bureau of Indian Affairs. In addition, the services of geologist Gerald Maughan were obtained from Northern Testing Laboratories for a short period.

The Panel's contract required that a final report be completed by the Panel by December 31, 1976. The Panel was also requested to make a preliminary report as of August 1, 1976, and progress reports as of the first of each following month, until the final report was made.

During the month of June, 1976, Secretary of the Interior Thomas S. Kleppe appointed Dennis N. Sachs, Deputy Assistant Secretary for Land and Water Resources, as Chairman of an Interior Department Teton Dam Failure Group, composed of Federal employees, and also named Mr. Sachs as liaison with the Independent Panel to Review Cause of Teton Dam Failure. Throughout the program of the Panel, close coordination was maintained with Mr. Sachs to assure that unnecessary duplication of field and laboratory effort was avoided, while preserving the independence of the investigating organizations.

To conduct its investigation most effectively, the Independent Panel established offices in Idaho Falls and at the Teton damsite. Data collection and technical analyses were accomplished at both locations. Daily overseeing of field investigative work was done by Panel staff at the dam, while general management of the program was provided in Idaho Falls.

CHAPTER 2
CHRONOLOGY OF FAILURE AND USBR REACTIONS
(Panel Charges Nos. 11 and 13)

The record shows that the first indication of grave difficulties at Teton Dam was observed at about 7:00 a.m. on Saturday, June 5, 1976, and that the breach of the dam was complete before noon of that day. Because of the dramatic events during the final hours preceding that breach, an excellent photographic record, both still and cinematic, exists of the various stages of heavy erosion, of progress of that erosion, of the final failure itself, and of post-failure flows. Because the related events were not so dramatic, records of the symptoms of erosion and of the erosion itself during the time prior to the major erosion are minimal. However, sworn witnesses who were questioned by special agents of the Department of the Interior reported observations of some of these early events. Although timing is approximate, being based on individual memories, the Panel has pieced together the following sequence. For locations, refer to Fig. 2-1. A section along the observed line of failure is shown in Fig. 2-2. The record of eyewitness accounts is included in Appendix C.

JUNE 3, 1976 – OBSERVATION, SMALL SEEPS DOWNSTREAM OF DAM

From testimony of Peter P. Aberle, Field Engineer, USBR, Teton Dam Project:

Starting on about June 3, 1976, I observed small springs in the right abutment downstream from the toe of the dam. These springs were clear water and did not appear to be serious in nature, but warranted monitoring by visual observation as frequently as routine inspections of the entire operation at the dam.

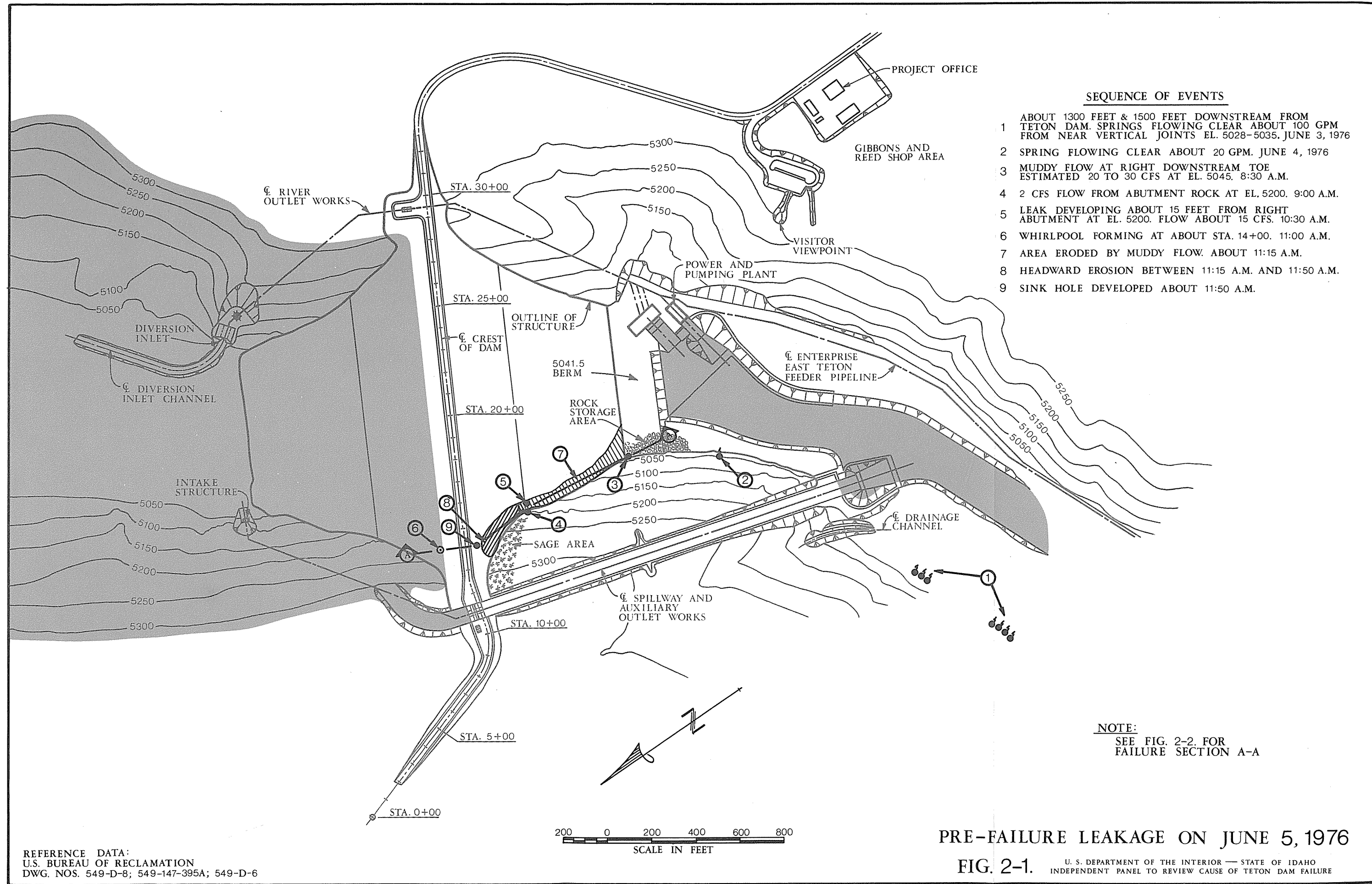
From testimony of Harry Parks, Supervisory Surveying Technician, Teton Dam Project:

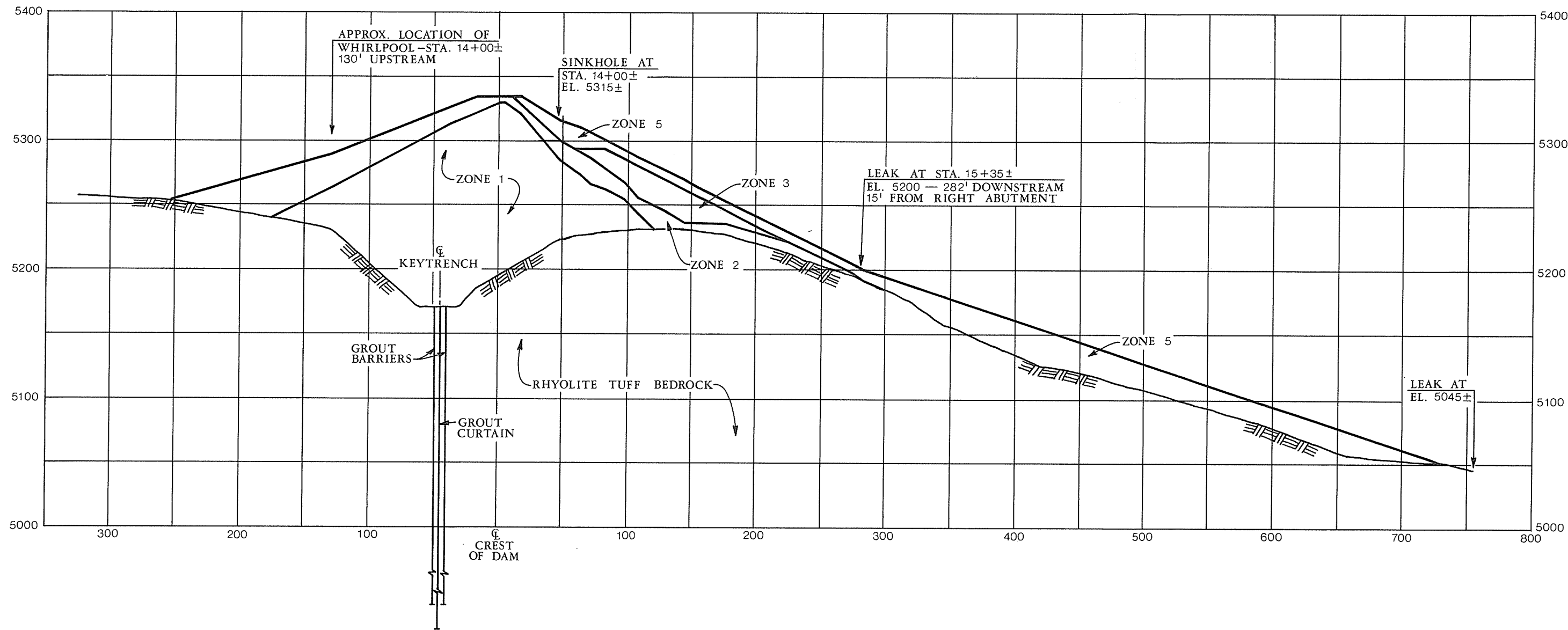
About June 3, 1976, I observed a small stream of water appearing along the bottom of the waste area about 1400 feet downstream from the toe of the dam. I was on the top of the south rim when I observed this water and so I could not say at this time whether the water was clear, muddy, etc. I was aware that Robison and Aberle were watching the flow on at least one occasion.

From testimony of Robert R. Robison, Project Construction Engineer:

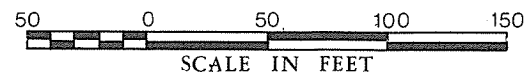
While there were rumors as early as April 1976 that there were leaks at the dam, there is no basis to these rumors, because there were no leaks.

On June 3, 1976, several small seeps in the rhyolite (volcanic rock) appeared about 1400 to 2000 feet downstream from the toe of the dam in the north abutment wall. The water was clear and all of these seeps totaled about 100 gallons of water per minute. This was felt to be a good sign because the dam was being filled and it indicated the water table gradient was acting in a normal manner. The water was clean enough to drink and if there had been a problem the water would have been turbid. I felt the area should be monitored by sight inspections and other mechanical means, the latter of which were never put into effect. I took pictures of the seepage [Figs. 2-3 and 2-4] and reported the matter to the E&R Center, Bureau of Reclamation, Denver, Colorado.





NOTE:
SEE FIG. 2-1 FOR LOCATION



SECTION A-A
ALONG APPROXIMATE PATH OF FAILURE

FIG. 2-2. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

REFERENCE DATA:
U.S. BUREAU OF RECLAMATION
DWG. NO. 549-D-8



Fig. 2-3. North canyon wall about 1300 feet downstream from Teton Dam. Clear water from several small seeps flowing about 60 gpm. June 3, 1976..



Fig. 2-4. North canyon wall about 1500 feet downstream from Teton Dam. Clear water flowing from rhyolite about 40 gpm. June 3, 1976.

From testimony of Kenneth C. Hoyt, Construction Inspector, Teton Dam Project:

Before June 5, I saw seepage in the bottom beyond the toe of the dam. This seepage was visible for about two or three days prior to June 5, and was 150 feet downstream of the toe of the dam. I never saw the seepage clearly, do not know the condition or volume. It was a slight flow and was of no great concern to me as it appeared rather natural.

No other records have been found by the Panel of leaks or seepage on June 3 or earlier, although public inquiries were made and replies were invited.

JUNE 4, 1976 – FURTHER SMALL SEEPS DOWNSTREAM

From testimony of Wilburn H. Andrew, Mechanical Engineer, Teton Dam Project:

At 9:00 a.m. on Friday, June 4, 1976, Stites and I walked around the right abutment (north side) area at the toe of the dam for the purpose of looking for leaks. We were doing this because one or two spring leaks had developed further down the stream in the abutment wall about the day before. We did not see any leaks around the toe of the dam or any where on the downstream face of the dam.

From testimony of Dick R. Berry, Survey Technician, Teton Dam Project:

On June 4, 1976 I recall seeing seepage near the right abutment wall below the toe of the dam. The water was clear and not really running – just settlement. There were no leakages or seeps at the dam.

From testimony of Stephen Elenberger, Construction Inspector, Teton Dam Project:

On Friday, June 4, 1976, I was working the 4:00 p.m. to 12:30 a.m. shift at the dam. Up until dark, which occurred at about 9:00 p.m. or shortly thereafter I made several observations of both the downstream side and the upstream reservoir. I had been alerted to pay particular attention for possible leaks because there were small spring like areas of water on the north side of the canyon well below the toe of the dam. These springs were clear water and had been visible for two or three days.

Until darkness I did not see any sign of a leak in the toe of the dam at the north or right abutment at about 100 feet from the top of the dam near the north or right abutment. The entire downstream face of the dam showed no sign of any problems. I also did not see anything unusual in the reservoir or upstream side of the dam. There was no sign of a whirlpool.

From testimony of Clifford Felkins, Surveying Aide, Teton Dam Project:

On Friday, June 4, I noticed for the first time some wetness in the waste area near the right abutment wall of the dam. There was no water flow, just wetness.

From testimony of Robert R. Robison:

On June 4, 1976, a small seepage occurred about halfway between the toe of the dam and the end of the spillway along the north abutment. This flow was approximately 20

gallons per minute and I had no concern because the water was clear. I checked this leak at about 4:30 p.m. on June 4 before leaving the dam and determined that there was no problem. At this time I also observed the entire downstream face of the dam and observed nothing unusual. I also observed that there was nothing unusual on the upstream reservoir side of the dam.

The record contains no other statements of observations of seepage prior to June 5. As of darkness on June 4, seepage had been observed from springs 1400 to 2000 feet below the toe of the dam in two groups with a reported total flow of 100 gpm and from a small spring midway on the right side, between the toe of the dam and the spillway. This latter flow was reported to be 20 gpm. Apparently these flows were not measured.

JUNE 5, 1976 – FIRST OBSERVATIONS – LEAKS AT ELS. 5045 AND 5200

The first record of observations of leakage on June 5 begins at 7:00 a.m. Testimony regarding observations prior to 8:00 a.m. follows:

From testimony of Clifford Felkins:

On Saturday, June 5, 1976, I arrived at the dam at about 7:00 a.m. . . . On June 5, the first thing that I saw connected with the later events of the dam collapse was a water flow coming from the toe of the dam. It was a steady flow of water, but I cannot estimate the volume. To the best of my recollection the water flow was clear. I noticed this flow while I was standing across the river on the canyon wall from the spillway. I was with Harry Parks and we came to the survey office . . . and reported the leak to Jan Ringel. This was about 8:15-8:30 a.m.

From testimony of Dick R. Berry:

On Saturday, June 5, 1976, I arrived at the Project office before 7:00 a.m. Harry Park's Volkswagen was in the parking lot . . . I had no watch with me at work on that date.

At 7:20 a.m., on June 5, I left the Project Office and drove down the upper south rim road to check three site [sight] rods. . . . While checking the site [sight] rods I saw a small seepage on the north side downstream face of the dam, right at the abutment and dam joint. This was approximately one-third of the way up the dam, but not as high as the change in slope. There was slight erosion, slow flow of water, but I do not recall it being muddy. The seepage appeared to be almost new. I returned to the office and Harry Parks, who was in the crew, reported the seepage to Jan Ringel about 7:35 a.m.

From testimony of Myra H. Ferber, Survey Technician, Teton Dam Project:

On Saturday, June 5, 1976, I reported to work at the Dam at 7:00 a.m. for the purpose of doing scheduled survey work. At about 7:30 a.m. Harry Parks, Richard Berry, Clifford Felkins, all surveyors, and myself, proceeded downstream from the dam on the south or left canyon wall to check sitings [sightings] While checking the sitings [sightings] we saw a small leakage about 100 feet below the top of the dam near the right abutment on the downstream face of the dam. The water was flowing down the face of the dam and washing away fill at the toe of the dam. We then proceeded to the office and reported the leak to Jan Ringel.

From testimony of Harry Parks:

On Saturday, June 5, 1976, I arrived at the project office a couple of minutes before 7:00 a.m. . . . We left the office about 7:35 a.m. . . and traveled down the south rim road downstream for the purpose of checking survey sights in order to perform a survey on the spillway on the north side of the dam. At about 7:50 a.m. a member of the survey party noticed water seepage. I then observed the water which was running out of the toe of the dam at about 50 feet from the north abutment wall ["ponding" on the berm at El. 5041.5]. I cannot estimate the volume but it was barely what could be called a stream at all. The water appeared muddy, but this may have been caused by the material over which it was flowing. We drove back to the office and I reported the water leakage about 8:00 a.m. to Jan Ringel.

From testimony of Jan Ringel, Civil Engineer, Teton Dam Project:

On Saturday, June 5, 1976, I arrived at work at 7:00 a.m. I had two survey crews working. . . . Mr. Parks checked the staffs for the spillway control on the south side of the dam opposite the spillway. They were on the canyon rim and noticed the lower leak on the dam near the toe at about 5,041.5 elevation. At about 7:30 a.m. Parks reported sightings to me. I drove down to the powerhouse and walked over to the leak. The water was muddy. The water was running between the rocks on the right abutment and not through the dam. I estimate the water flow to be about 20-30 cfs at this time. I did not detect any increase at that time.

The only other noticeable thing at this time was some springs at the base of the dam against the abutment – 200 feet below the other. This had been there for one or two days previous. This was clear water running at about 10 gallons per minute. Mr. Aberle and Mr. Robison had previously checked this.

During a conference on October 29 requested by the Panel staff with Messrs. Robison, Aberle, Ringel, Parks, Isaacson, and Rogers, Mr. Ringel supplemented the foregoing sworn testimony by stating that he first examined the leakage at El. 5045 and that he noticed that water which had been flowing down the right groin during the night of June 4, or early in the morning of June 5, had eroded a shallow channel that had not been there at 9:00 p.m. on the preceding night. He said that there was no water in this channel when first observed during the morning of June 5 but that it was damp in places (see letter of October 31, 1976 from Robert B. Jansen to Panel Members in Appendix B).

From testimony of Harold F. Adams, Mechanic, Gibbons and Reed Company, Teton Dam Project:

On Saturday, June 5, 1976, I arrived at Gibbons and Reed yard behind Bureau office at 7:00 a.m. to work on equipment. As I drove in I saw a small trickle of water on downstream slope of dam against the north abutment and about 100 feet from top of dam. About 30 feet out there was a wet spot.

From testimony of David Burch, Mechanic, Gibbons and Reed Company, Teton Dam Project:

I arrived for work at 7:00 a.m. on June 5, 1976. As I was driving up the canyon to the G-R shop I noticed a seepage down the north side of the dam. The seepage was slight and started at about the 5200 level near the change of the slope and ran down the abutment wall towards the toe of the dam. You could not actually see water running – just the

dampness. I could not tell if the water was clear or muddy because it was just dampness. I mentioned to some of my co-workers that the dam was leaking.

From testimony of Perry W. Ogden, Mechanic, Gibbons and Reed Company, Teton Dam Project:

On Saturday, June 5, 1976 . . . I arrived at shop at 7:00 a.m., went right to our shop area. I was out of view of most of dam, but could see top part. Shortly after I arrived, Dave Burch told me there was a wet spot on the downstream side of dam. I walked over to the visitor's viewpoint on south rim and saw a wet spot at about 100 feet from top of dam against abutment. No flowing water — just a wet spot.

It seems reasonable to conclude that the seeps at El. 5045 and El. 5200 were both active as early as 7:00 a.m. The record does not permit determining which was activated first. Neither appears to have existed at 9:00 p.m. on June 4.

JUNE 5, 1976 — 8:00 A.M. TO 10:00 A.M. — DEVELOPMENT OF ADDITIONAL LEAKAGE AT ELS. 5045 AND 5200

From testimony of Peter P. Aberle:

Between 8:20 a.m. and 8:30 a.m. on Saturday, June 5, 1976, I received a call from Jan Ringel at my home and he told me of a leak at the right abutment toe area of the dam. Ringel estimated the leak to be about 20 to 30 sec. ft. I asked my wife to call Mr. Robison and I left for the dam. I drove directly to the powerhouse area and briefly inspected the leak from the left side abutment area. I noted that the water was muddy and estimated the volume to be the same as that given me by Ringel. I do not believe the water was running long because there was very little erosion in the gravel at the toe of the dam.

At approximately 9:00 a.m. I went to the project office and met Mr. Robison and Jan Ringel. Mr. Robison and I walked out on the top of the dam and walked down the downstream face of the dam to a leak located at the 5200 feet elevation, near the right abutment wall. The water in this leak was running at about 2 sec. ft. and was only very slightly turbid. The leak appeared to be coming from the abutment rock. [*] The leak at the toe of the dam was running turbid water from the abutment rock at an estimated volume of 40 to 50 sec. ft.

From testimony of Robert R. Robison:

I . . . arrived at the Reclamation Office at about 9:00 a.m. Aberle and I drove to the downstream toe of the dam and I observed a major leak at the downstream toe at the right abutment at about 5045 elevation. The water was flowing at about 50 cubic feet per second, was moderately turbid and was coming from the abutment rock. [*] This was

*During the October 29 conference with the Panel staff, Mr. Robison stated that the flow first observed at El. 5045 was from the talus, not formation rock. Mr. Robison was asked whether the talus at the toe of the right canyon wall could have carried appreciable flow without such flow being apparent on the surface. He replied that such a condition was entirely possible. He said that in retrospect he believes that the leak seen issuing from the abutment at El. 5200 on June 5 was also from talus, as well as the seepage discovered between the toe of the dam and the spillway stilling basin on June 4.

not connected to the other seepages mentioned above. I felt this seepage was coming straight out of the abutment rock and not through the dam.

I also saw another leak at about 5200 elevation in the junction of the dam embankment and the right abutment. The water was slightly turbid and issuing from the rock at about 2 cubic feet per second. The water from this leakage was not flowing at a great enough volume to even reach the toe of the dam.

At about 10:00 a.m. I observed a large leak developing about 15 feet from the right abutment in the dam embankment at an approximate elevation of 5200. This leak was on the downstream face of the dam and was adjacent to the smaller leak at the same elevation. At first the flow of water was about 15 cubic feet per second and it gradually increased in size. The water was turbid.

During the October 29 meeting with Panel staff, Mr. Robison said that from his vantage point looking directly into the hole at El. 5200, it was a tunnel about 6 ft in diameter running roughly perpendicular to the dam axis and extending back into the embankment for about 35 ft, as far as he could see.

From testimony of Dick R. Berry:

We then started work on the spillway at about 8:30 a.m. Just before we went into the spillway I saw a wet area at the end of the sage area just off the abutment on the downstream face of the dam. I do not recall this being running water, just a wet area.

From testimony of Myra H. Ferber:

At about 8:30 a.m. we checked the water elevation in the reservoir on the upstream side of the dam. The water elevation was 5301+ feet and I did not notice anything unusual about the reservoir water — specifically there was no indication of a whirlpool.

From testimony of Jan R. Ringel:

At about 8:50 a.m. Mr. Aberle and Mr. Robison arrived at the dam. I briefed them lightly and we drove over the top of the dam to the right abutment. At this time Mr. Robison and Mr. Aberle walked down the downstream face of the dam to look at the leak. I drove the pickup around the rim road to meet them at the bottom. When I arrived, I walked directly to the right abutment. I stopped momentarily at the powerhouse and took some pictures of the leak. . . . [Figs. 2-5 and 2-6]

From testimony of David Burch:

At about 9:30 a.m. I noticed a wet spot appear on the north side of the face of the dam. This spot was about 100 feet from the abutment and probably 125 feet from the top of the dam. The damp spot appeared to be about 3 or 4 feet in diameter from my viewpoint at the trailer. There was not any water flowing from the damp spot at that time.

At 10:00 a.m. I observed water coming from the above described spot. The water was coming at a steady flow and was muddy.



Fig. 2-5. Muddy flow at about El. 5045 at right downstream toe estimated 20 to 30 cfs (9,000 to 13,500 gpm) 8:30 a.m. June 5, 1976.



Fig. 2-6. Close-up of leak shown in Fig. 2-5. Muddy flow through rock on right abutment at about El. 5045. 8:30 a.m. June 5, 1976.

From testimony of Jerry Dursteler, Master Mechanic, Gibbons and Reed Company, Teton Dam Project:

On Saturday, June 5, 1976, Perry Ogden and I arrived at the company yard behind the Reclamation offices at about 10:00 a.m. . . . When at the office, I heard water running. I drove downstream from the dam on the upper south rim road to look at the spillway and to see if water was flowing over it. I saw wetness on the downstream face of the dam and seepage against abutment wall. This was about at the slope change in the dam. I cannot be more specific. The water was muddy, but was merely a light stream. I went back to my truck. By then the wet spot had started flowing. This was a very small flow. I returned to my office and told Adams and Burch there was a problem. The three of us walked behind the Reclamation offices on the south side of the dam to look at the dam. The leakage had increased considerably and started eroding a hole. This was about 10:15 a.m.

The foregoing testimony, covering the period from 8:00 a.m. to 10:00 a.m., includes the history of the leak which developed a short distance south and about at the same elevation as the 2 cfs spring observed earlier. One observer (Berry) noted it as a wet spot at 8:30 a.m. At 9:30 a.m. David Burch reports seeing a wet spot appear on the north side of the face of the dam 100 ft from the abutment and 125 ft below the crest. He estimated the spot to be 3 or 4 ft in diameter. He stated that at 10:00 a.m. water was flowing from that spot. Dursteler reported development of flow from a wet spot at about the same time followed by increasing flow.

Beginning at about 10:00 a.m., the record of leaks expands both in eyewitness statements and in photographs taken by several photographers. These records clearly show the leakage at El. 5045. Although the outflow at that level was not measured, the eyewitness accounts show an increase after the first observations by Parks at about 7:50 a.m. Ringel reported 20-30 cfs to Aberle at about 8:30 a.m. Aberle later concurred in that estimate. Subsequently, Aberle and Robison estimated 40 to 50 cfs.

The records clearly show the development of leakage near the abutment at El. 5200, first the small 2 cfs flow from the abutment followed about 10:30 a.m. by the 15 cfs flow from the embankment (Figs. 2-7 and 2-8). Later, following a progressive upward erosion of the original embankment leak, a sinkhole or crater developed just below the crest above the 15 cfs flow at El. 5200. Successive photographs show this development clearly; also the upward erosion from El. 5200 toward the sinkhole (Figs. 2-9 through 2-16).

Although it is spectacularly shown in the photographs, little notice of this development seems to have been taken by people on-site. The following quotations seem to be the only testimony as to its observation.

From testimony of Clifford Felkins:

I do not recall the time when we first observed the upper water seepage. We were standing near the top of the dam in the spillway and observed the second hole beginning to form just as we were coming out of the spillway. We were leaving the spillway on the instruction of Pete Aberle who told us to get out. I did not actually see any water come out of the upper hole because the dam caved in and the two holes became one large one. The water that came through was muddy. I cannot estimate the volume but it was a lot of gallons. The volume increased very rapidly.



Fig. 2-7. View from top of embankment toward leak at El. 5200 near right abutment. Approximately 10:30 a.m. June 5, 1976.

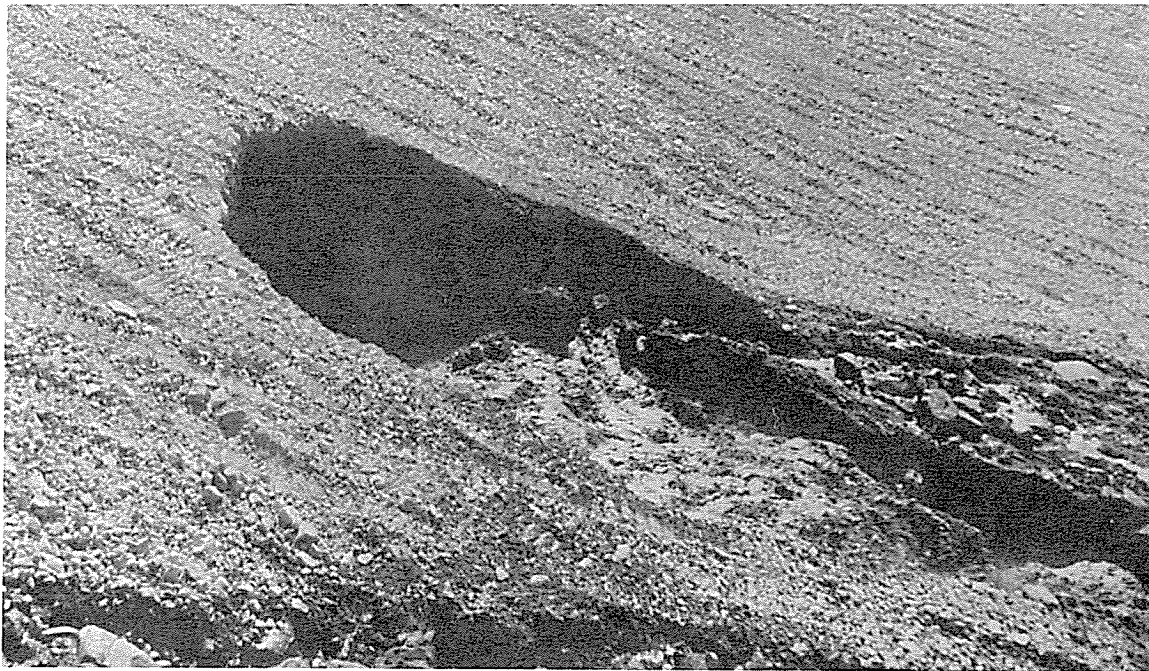


Fig. 2-8. Close-up of leak shown in Fig. 2-7. Turbid flow about 15 cfs. Approximately 10:30 a.m. June 5, 1976.



Fig. 2-9. Flow increasing. Dozers sent to fill hole at El. 5200. About 10:45 a.m. June 5, 1976.



Fig. 2-10. Dozers lost in hole. About 11:20 a.m. June 5, 1976.



Fig. 2-11. Approximately 11:30 a.m. June 5, 1976.



Fig. 2-12. Second hole in face of dam. A few minutes after 11:30 a.m. June 5, 1976.



Fig. 2-13. About 11:50 a.m. June 5, 1976.



Fig. 2-14. Dam crest breaching. 11:55 a.m. June 5, 1976.



Fig. 2-15. Early afternoon June 5, 1976.



Fig. 2-16. Late Afternoon June 5, 1976.

From testimony of Jerry Lynn Walker, Superintendent, Gibbons and Reed Company, Teton Dam Project:

While I was standing on the visitor's observation point and after the two M-K dozers were lost a crack developed above the hole. The crack was in the shape of a semi-circle with the arc at the top; was about 30 feet above the hole; and I would estimate that it may have been as much as 100 feet in total length. The earth started sluffing down from the crack towards the hole and caused an offset in the earth on the face of the dam as it sank. As the earth fell in a small hole developed above the crack. I would estimate this was about 10 to 15 feet above the crack and was initially six or seven feet in diameter. I then left the visitor's observation point and drove to the top of the dam. I would estimate that I reached the top of the dam at about 11:40 a.m.

DEVELOPMENT OF UPSTREAM WHIRLPOOL

Aberle reports a "loud burst of water" at a time which he now estimates at 10:30 a.m.* From his testimony, that burst was coincident with development of the leak at El. 5200, 15 ft to the left of the right abutment. Aberle also reported that he observed a whirlpool in the reservoir surface upstream at Sta. 13+00 (about 150 ft from the spillway) and about 10 to 15 ft into the water from the riprap. He estimated the whirlpool to have been 0.5 ft in diameter when first observed. Although he reports the whirlpool to have formed in clear water, he also reported "I noticed that the water along the right bank was turbid about 150 feet upstream from the dam and about 15 to 20 feet out from the edge of the abutment. This turbid water was first noted at 9:30 a.m. by me before the whirlpool started and was thought to be turbid due to wave action."

From testimony of Alvin J. Heintz, Construction Inspector, Teton Dam Project:

As I was talking to Aberle we noticed a small whirlpool forming in the reservoir on the upstream side of the dam. The whirlpool was about two feet in diameter, close to the north or right abutment and about 10 to 15 feet out from the dam. . . .

I remained on the top of the dam near the north end and helped direct two dozers pushing riprap into the whirlpool. While working I saw the downstream flow of water increase in volume and the whirlpool increase in size.

From testimony of Charles L. Entwisle, Construction Inspector, Teton Dam Project:

As I approached the north or right side a small whirlpool about 10 feet from the upstream face of the dam just off the right abutment was forming in the reservoir. The time of this was about 10:50 a.m. The whirlpool was about two feet in diameter and the vortex eye was about six inches. It appeared to be stationary, but grew in size as I watched it.

From testimony of Jan Ringel:

At approximately 10:50 a.m. a whirlpool developed on the upstream face of the dam. This was at the right of the dam about 15 to 20 feet away from the dam.

*Revised from 10:00 a.m. to 10:30 a.m., as discussed during October 29 conference with Panel staff.

Robison reported:

At about 11:00 a.m. I saw a whirlpool developing on the upstream side of the dam in the reservoir at about 10 to 15 feet into the water from the face of the dam and less than 100 feet from the abutment wall. I had looked for a whirlpool at about 10:30 a.m. and had not seen one. The whirlpool was approximately six feet in diameter, was stationary, and appeared to be increasing in size. The water on the reservoir side was clear.

The approximate elevation of the whirlpool was 5295. I would estimate that at this time the volume of water going through the upper leak on the downstream face of the dam was 100 cubic feet per second. . . .

When I noted the whirlpool developed at about 11:00 a.m. I realized there was imminent danger of the dam collapsing. From this time on there were numerous people making telephone calls alerting people in the area of the danger.

In their meeting with Panel representatives on October 29, Robison and Aberle agreed that the whirlpool was probably not as close to the abutment as they previously had estimated. They said that it may have been as far out as Stas. 13+70 or 13+80.

From testimony of Alfred D. Stites, Construction Inspector, Teton Dam Project:

I arrived at the top of the dam at about 10:40 a.m. and within three or four minutes I noticed a whirlpool forming in the reservoir on the upstream side of the dam about 22 feet into the water from the face of the dam. The whirlpool was approximately 1½ feet in diameter at the outset, briefly got smaller, and then began increasing in size. The water in the area of the whirlpool appeared to be slightly muddy.

David Burch reported:

I had started pushing riprap from the face of the dam towards a whirlpool or funnel which had developed on the reservoir side of the dam shortly after 11:00. The whirlpool was directly across from the spot where the hole appeared on the downstream face of the dam. When I first saw the whirlpool, it was very small, maybe a foot across and was very muddy and it was surrounded by clear water. I saw no other mud on the upstream side. The water on the reservoir side was very calm. There was very little wind. The whirlpool was about 20 feet out from the upstream face of the dam and about 100 feet from the north abutment. We tried by using the riprap to build a ramp to the whirlpool but never succeeded.

Sometime after 10:15 a.m. Dursteler started taking pictures of the downstream canyon walls and some of the face of the dam. I took pictures from the visitor's viewpoint, downstream rim and from the Morrison and Knudsen yard.

Ogden reported:

Burch arrived with a dozer and the two of us crossed the dam and started pushing riprap into whirlpool. This probably about 10:00 a.m. or so. Whirlpool developed at this time about 4 feet in diameter.

Walker testified:

By the time I had arrived at the dam at 10:30 a.m. two D-8 dozers . . . had been dispatched to the top of the dam to work on the upstream face and push riprap into the whirlpool which had developed.

John P. Bellegante, Excavation Superintendent, Morrison-Knudsen and Kiewit, Teton Dam Project, testified:

Others had found whirlpool on upstream face and were directing dozers to push riprap into whirlpool area. Whirlpool was about 18 inches in diameter near the north abutment wall about 15 feet from upstream face of the dam. I did not notice it getting bigger.

Jay N. Calderwood, General Excavation Foreman, Morrison-Knudsen and Kiewit, Teton Dam Project, testified:

I . . . worked on pushing riprap into the whirlpool, which was on the upstream side about 12 feet to 14 feet in water near the right abutment, not far out. The whirlpool was about 20 feet to 30 feet in circumference and 5 feet to 6 feet in depth. It continued to get larger.

From testimony of David O. Daley, Equipment Operator, Morrison-Knudsen and Kiewit, Teton Dam Project:

. . . I operated a Gibbons and Reed dozer trying to fill in the whirlpool on the upstream reservoir side of the dam.

The whirlpool was about 30 feet out into water and about 20 feet in circumference. The pool was rather close to the north wall.

Vincent M. Poxleitner, Jr., Project Engineer, Morrison-Knudsen and Kiewit, Teton Dam Project, testified:

By the time I got to the top of dam, whirlpool had developed on upstream side of dam. I cannot give times. The whirlpool about 25 feet from upstream face of dam and 75 feet from right abutment. About 3-1/3 feet to 4 feet in diameter.

Henry L. Bauer, resident farmer on the north side of Teton Canyon, reported:

Time approximately 11:15 a.m. to 11:30 a.m. I saw a truck dump material on the upper face of the dam as I approached. I noticed a whirlpool 8 feet across against abutment and face of dam. Large commotion and muddy water. Water away from whirlpool was semi-clear. Then a large part of time [dam] — 20 feet wide and 20 feet high sluffed off into the whirlpool — one big chunk. This created extra commotion in whirlpool and boiled up more. In a matter of one minute the top section of the dam dropped and the dam had collapsed. I never looked at my watch. . . .

REACTION OF BUREAU OF RECLAMATION PERSONNEL TO THE EMERGENCY

Pinpointing of the beginning of the period of emergency at the Teton Dam and Reservoir would be difficult. There is no question in retrospect that the sequence of events observed on the morning of Saturday, June 5, 1976, indicated a rapidly developing emergency. Developments during earlier days of that week undoubtedly have some significance and may have pointed, though more subtly, to adverse circumstances threatening the integrity of the dam. To assess the performance of Bureau of Reclamation personnel, therefore, events and conditions during the week prior to failure must be examined.

Project employees knew of the probability that flows in the Teton River would exceed, and had exceeded, the capacity of the completed auxiliary outlet works and that completion of the construction of the unfinished river outlet works was necessary to assure control of reservoir filling.

Project personnel had been carefully reviewing runoff forecasts for the watershed and foresaw that the reservoir would fill at rates greater than the 1-ft-per-day criterion originally prescribed. In view of the delay in the completion of the river outlet and to obtain benefit from greater generating head, they asked for relaxation of that requirement.

The trends of rising water in the observation wells were being monitored. Frequency of readings was about once a week until the spring of 1976, when it was increased to about twice a week. Inspectors were on the alert for signs of seepage at the dam or in the canyon downstream. They made daily inspections of these areas.

During the first week in June, 1976, the Project Construction Engineer gave oral instructions to designated field people to be on the alert. He made personal inspections of the dam and its environs. He took photographs of the seepage discovered on June 3, 1976 and dispatched a report on the situation to the Engineering and Research Center in Denver. Again on June 4, he examined additional seepage which had appeared downstream from the dam. He judged it not to be dangerous because he found the water to be clear. On June 5 at 8:30 a.m., when he was notified by telephone of the leakage from the dam itself, he left his home immediately and arrived at the project office at about 9:00 a.m. He went to the downstream toe of the embankment and examined the leakage. Within half an hour after his arrival, he entered discussions with the Project Manager for the contractor to determine remedial measures. At that point he judged the situation to be critical but believed that the leakage could be controlled, since it appeared to be coming from the abutment rather than from the dam.

He made telephone calls to the Bureau of Reclamation Regional Office in Boise and to the Engineering and Research Center in Denver to alert them to the situation. He considered notification of residents downstream; but since he did not believe that an emergency situation was then imminent and did not want to cause a panic he decided against such notification.

As the leak at El. 5200 turned into a hole with a "loud burst" at about 10:30 a.m., he ran to the project office and at about 10:43 a.m. notified the Sheriff's Offices of Madison and Fremont Counties of the hazard and advised them to alert the citizens of potential flooding. The Project Construction Engineer did not hesitate in notifying the citizenry of the hazard at that time. Power supply and communication to the project facilities were interrupted at 11:57 a.m.

At 12:10 p.m., he left the damsite to go to Rexburg to place telephone calls to Bureau of Reclamation officials in Boise and Washington, D.C., notifying them of the collapse of the dam.

On the day of the failure, there was no schedule of work shifts for Bureau of Reclamation employees that would have required personnel at the dam on a 24-hour-a-day basis.

In general, Bureau of Reclamation personnel appeared to have been dutifully responsive during the emergency on June 5, 1976. Supervisory Surveying Technician Harry Parks, for example, spotted a small seepage at the right groin of the dam at a time reported between 7:25 and 7:50 a.m. and immediately saw that it was reported to his supervisors. Without delay, those supervisors went to the leak and assessed the situation. Other observations were made and responded to as the morning went on.

Jan R. Ringel, civil engineer at the project, was one of the supervisors who received the first report of leakage. After evaluating the situation, he telephoned the field engineer. Within half an hour both the field engineer and the project construction engineer had arrived at the dam.

USBR personnel acted promptly and responsibly throughout the emergency to protect the public and the project. They directed the contractor to mobilize all possible equipment and they took initial steps to open the river outlet. Efforts to close the erosion conduit at El. 5200 by pushing embankment material into the downstream exit were futile. Likewise, efforts to close the upstream entrance by pushing riprap into the opening were foredoomed because of inability to move physically a sufficient mass of properly graded material into the opening fast enough to abate the rapid erosion. Any possible success of such an effort would have required several thousand cubic yards of readily available stockpiled material and almost instant mobilization of a considerable fleet of loaders, dump trucks and bulldozers. Neither the material nor the equipment was available.

The concern of USBR personnel for the people downstream was apparent. They sounded the warning as soon as failure could be foreseen.

CHAPTER 3 PANEL INVESTIGATIONS

The Panel's approach to its assignment to review the cause of Teton Dam failure has been to:

- (1) obtain, analyze and evaluate all relevant information which could be obtained from the United States Bureau of Reclamation, the United States Geological Survey, the construction contractor, and any other available and knowledgeable source regarding the regional and site geology, pre-siting investigations, siting decisions, pre-design investigations, design, contract specifications and drawings, construction practices, progress and inspections, in-progress changes, if any, pre-failure operation, mechanism of failure, including sworn eyewitness accounts, and actions of respective authorities during and immediately after the failure;
- (2) supplement documentary information, as it was received, by supplemental inquiry, including public hearings, written requests and responses, and oral inquiries where appropriate;
- (3) make (a) detailed study of the post-failure condition of the dam, its auxiliary structures and its foundation, by inspections, dissection, subsurface drilling; (b) special tests of foundation materials; (c) detailed geologic maps and joint surveys; (d) tests of remnant materials; (e) detailed stress analyses; (f) studies of photographs for comparison of post-failure conditions with pre-construction and construction conditions; (g) measurements of post-failure geodetic positions of surface and subsurface points, to compare with data available before filling of the reservoir;
- (4) evaluate relevant data in order to sort out those of greatest significance in determining cause;
- (5) report the results of the foregoing activities to the extent possible by completion of all of the investigation work involved by December 31, 1976.

This approach was initiated by telegrams, dated June 11 and June 14, 1976, to Director, Design and Construction, U.S. Bureau of Reclamation, Denver, and by setting the Panel's first working session and inspection of conditions at the site for the week of June 28-July 2, 1976. The telegrams requested a data book, available if possible before the meeting, to present (1) site geology in plan and sections with any test results on foundation materials, (2) site exploration with detail of drill logs, exploration trenches, borrow materials and tests, (3) grout records in detail showing non-average takes by location and depths, patterns used and record of any interconnections, (4) foundation preparation showing both before and after conditions, (5) design memoranda for embankment, spillway, diversion structures and outlets, (6) basic drawings and technical specifications, (7) any outside reports re site or designs, (8) construction history of borrow pits, hauling, placement, progress, inspection, in-place tests, (9) any seepage measurements or observations, (10) eyewitness accounts on progress of failure, (11) hydrology, (12) seismicity, (13) drain designs and drainage observations, (14) any changes in spillway or auxiliary outlet structures, (15) any changes in precise level or horizontal control survey points, (16) changes in topography upstream and downstream, (17) photos of foundation as approved at start of embankment, particularly in the cutoff trench, (18) record of any seeps or springs in the cutoff and core contact area, and (19) record of cofferdam seepage and pumpage from the foundation area.

Prior to the Panel's convening for its first session, the Department of the Interior had recorded sworn testimony of 37 eyewitness observers of pre-failure and during-failure conditions. Of those 37

persons, 14 were Bureau of Reclamation staff and employees, 13 were employees of the construction contractor, and 10 were from the general public. In parallel with these eyewitness accounts, there became available several excellent photographic sequences in still and later in motion picture form. In order to supplement these eyewitness accounts with any available observations of failure-related but pre-failure conditions, a public call was issued, and two public hearings were held in Idaho Falls on July 21, 1976.

All members of the Panel were present for the June 28-July 2 working sessions, and for the June 30-July 1 site inspections. An interim report, covering the Panel's activities up to July 2, was forwarded to the Secretary of the Interior and the Governor of Idaho on July 2, 1976. The full text of that report is attached in Appendix B.

During this first working session, the need for professional staff and technical and administrative support was recognized. To fill the professional need, the services of Robert B. Jansen, as Executive Director, were secured through the cooperation of the Governors of Idaho and California. Also, the services of Clifford J. Cortright, Staff Engineer, and Laurence B. James, Staff Geologist, were secured within a few days of Mr. Jansen's appointment. Soon thereafter, geologist Frank B. Sherman joined the professional staff. Through the excellent cooperation of the Office of the Secretary, Department of the Interior, supporting properties, services, and technical and administrative assistance have been made available to the Panel through various bureaus of the Department.

Simultaneously with its July 2 report the Panel addressed the Director, Design and Construction, USBR, Denver, saying:

The following activities represent the Panel's highest priority and are recommended for immediate implementation. It should be recognized that additional activities will be proposed in the coming months.

1. The remnant of the right-abutment keyway fill to the left of the spillway should be excavated to permit inspection of conditions below Elevation 5301. Down to Elevation 5301 the remnant can be removed in any manner that will not disturb the material below. Below Elevation 5301 the remnant can be removed in any stages and by any means, provided that a width of undisturbed material remains with a minimum horizontal thickness of five feet on each side and a minimum vertical distance of ten feet above the bottom of the original trench. The material within the five-foot envelope on each side should be removed by hand, where directed by the Panel's representative, as required to permit appropriate sampling to allow description of conditions of soil, rock, and any joint treatment disclosed by the excavation, to allow observation of any indications of piping or other defects. The bottom ten feet should be removed in two lifts. These lifts should be preceded by excavating trenches at places selected by the representative of the Panel to a depth of five feet with appropriate sampling and observation.
2. Any debris remaining on the face of the central part of the abutment, especially where the grout cap remains intact, should be carefully cleaned to permit detailed inspection.
3. The area of the lower spring (50 cfs) should be exposed. Any original material still in place should be left undisturbed. The details of jointing of the rock in this area should be carefully examined.

4. All steps necessary to assure safety at the remaining left section of the dam can be carried out promptly.

5. In order to provide some quantitative evaluation of permeability in the rocks in the right abutment, detailed studies should be made on enlarged photographs of representative areas of each joint type near the keyway.

Total footage of open joints per unit of area (e.g., one square yard) should be determined by direct measurements on enlargements of the photos, using a reliable scale with which a grid system is drawn on the enlargement.

The details of this survey, including best lighting (either direct sun during the forenoon or on a cloudy day) should be developed in a pilot program.

6. An item of prime importance is the nature of the joint system in the right abutment on either side of the keyway. Particularly important is the identification of major, throughgoing joints on the downstream side of the keyway that might provide access of water to the embankment.

Primary and secondary joint systems should be plotted on a new topographic map. Symbols may be used to indicate wide and continuous joints in contrast to the numerous, smaller joints. Any evidence of springs or watercourses along or through the joints should be indicated on the joint map.

Response was prompt, and on July 16, 1976, the Bureau of Reclamation awarded its Contract No. DC-7232 to Gibbons and Reed, Salt Lake City, to do the requested excavation. Notice to proceed was issued July 23, 1976, and mobilization started on July 23. Actual removal of the right remnant of the dam started July 26, 1976. This excavation proceeded expeditiously, by five-foot working levels, or platforms, to El. 5200, with trenching of each platform in accord with the Panel's need to inspect the core remnant for any evidence of water channeling or cracking and of the manner in which the key trench was excavated, sealed and filled.

A large volume of information was furnished, with oral perspective and explanations, during the Panel's sessions June 28, 29, and 30. A list of the related exhibits is contained in Appendix A. Many of these records have been supplemented by others furnished to the Panel's staff at the site, and by oral and other written requests.

Further information was desired on the manner in which the grout curtains were closed and in which the core was built into the key trench. This information was desired both from the Bureau of Reclamation as designers and constructors of the dam, and of the contractor who implemented that construction. Accordingly, on August 18, 1976, a questionnaire was directed concurrently to the Director, Design and Construction, USBR, and to the Chief Executive Officer of Morrison-Knudsen, as the sponsoring member of the construction contractor, Morrison-Knudsen-Kiewit, requesting descriptions of: (Refer Appendix B)

a. The manner in which axial grout distribution and closure were assured when the up and downstream grout travel was relatively unlimited. Details of any doubts over the effectiveness of this axial distribution in any particular location along the three grout curtains between Station 18+00 and Station 2+00 will be helpful. Likewise, details of difficulties in obtaining assurance of axial closure at any stations or grout holes along this same stretch of curtain will be helpful.

b. The manner in which the key trench between Station 18+00 and Station 2+00 was prepared to receive the first embankment material. Compare the way in which this trench was prepared with "broom clean." If there were differences in clean-up between particular stations, because of weather, or any other cause, please describe such differences in detail.

c. The manner in which any fissures or open joints in the key trench walls and floor were sealed between Station 18+00 and Station 2+00; that is, the manner in which, and the places where, slush grouting, dental concrete, gunite, or shotcrete may have been used, also the extent to which such sealing was general. Were any joints left unsealed and, if so, where? If known, please indicate the particular stations, if any.

d. The method of material selection, preparation, placement and compaction, in the key trench, of the "specially compacted earthfill" shown in the cross section marked "Foundation Key Trench" on USBR Drawing 549-D-9. If special difficulties were encountered in selection, preparation, placement or compaction at any points along the length from Station 18+00 to Station 2+00, please describe each.

e. The method of material selection, preparation, placement and compaction in the key trench between Station 18+00 and Station 2+00 of the core material. If special difficulties were encountered in selection, preparation, placement or compaction at any points along the length from Station 18+00 to Station 2+00, please describe each.

f. The manner in which the contact area under the core of the dam outside of the key trench was prepared to receive the first core material. If special difficulties were encountered at any location along the length of dam between Station 18+00 and Station 2+00, please describe.

g. The manner in which core material was selected, prepared, placed, and compacted outside of the key trench between Station 18+00 and Station 2+00. If special difficulties were encountered, please describe in detail by specific location.

h. Similarities and significant differences in the appearance of the walls and floor of the key trenches in the right and left abutments.

The answers of the addressees are contained in full in Appendix B. The USBR response is quite detailed. The contractor's response is in two parts. One is from the contractor per se, and the other is from the grouting subcontractor, McCabe Bros., Inc. The prime contractor's answer was rather general and not sufficiently responsive.

The full Panel met in technical working session again on August 2-5, 1976. At that time the excavation was well started. That work was inspected. Because of the need to know the physical properties of the in-place materials of the right remnant and the subsurface conditions under that remnant, the Panel appended to its progress report a list of additional physical work, analyses and tests required to be conducted. The report included the following statements:

A. Purpose

In its report of July 2, 1976, the Panel listed five potential causes of the piping failure of Teton Dam, and on the same date, in a letter to the Director, Design and Construction of the Bureau of Reclamation, listed items of highest priority recommended for action by

the Bureau to provide data for choosing among the potential causes. In its deliberations during its meeting of August 3-5, the Panel concluded that the field evidence virtually excludes massive seepage around the end of the grout curtain as a likely cause. Accordingly, the following detailed program was developed to aid in discriminating among the other four hypothetical causes, namely whether the massive seepage or piping took place (1) through the grout curtain, (2) through the core at the core-to-rock contact, (3) through the core above the base of the keyway core-to-rock contact, or (4) through a crack in the core. The program is in part a particularization of the work recommended on July 2, and in part a supplement to that work.

B. Investigation of Bottom of Key Trench and Grout Curtain

The purpose of the program is twofold: first, to determine if any cracks encountered in the rock in the bottom of the key trench, either up- or downstream, are open enough to permit flows of water through them; and second, to test the watertightness of the grout curtain under the grout cap and under the spillway. The section of the key trench to be tested extends from Station 12+50 to 14+50.

To test the water-carrying characteristics of cracks in the bottom of the key trench, it is proposed to pond water over selected cracks and observe the drop in the level of ponds. Each pond can be formed by placing a dike of stiff mortar on the low side of the crack, high enough to produce a depth of water of about 6 inches over the crack. Visual observation of the loss of water will permit a rough idea of whether the crack is relatively open or tight. At open cracks, an approximate measurement should be made of the outflow per linear foot of crack per minute. It is suggested that the wider cracks be tested first, and then the narrower ones.

Tests should be made both upstream and downstream of the grout cap. It is envisioned that between 10 and 20 representative cracks should be tested in the proposed section. The cracks tested should be distributed throughout the length of the section. If most of the cracks leak substantially, additional tests might be made to verify the conclusion that most cracks would transmit water easily.

To test the watertightness of the grout curtain, it is proposed to drill through the grout cap and the spillway crest into the rock below, and to watertest these holes. The holes should preferably be of Ax size and cores should be obtained from each hole to permit observation of any grout that may fill cracks in the rock. The holes through the grout cap should be drilled to a depth of 10 feet below the bottom of the grout cap, watertested, drilled 10 feet more and tested again. If pressure is used, it should not exceed 10 psi at the collar. The rate of flow in each stage of the hole should be recorded. If the second stage of any hole shows large leakage, a third 10-foot stage should be drilled and tested.

It is suggested that tests be carried out on the centerline of the grout curtain approximately at Stations 12+65, 13+05, and 13+40. At each station, three holes should be drilled, one vertical, one inclined $22\frac{1}{2}^{\circ}$ from the vertical toward the abutment, and one inclined 45° into the abutment. At each location, three holes should be drilled, in each stage, before starting the water testing.

It is also suggested that holes be drilled at about the center of each of the three spillway bays. Three holes should be drilled at each location, one vertical, one at an angle of 30° toward the river, and one at an angle of 30° away from the river. The holes through the

spillway crest should be drilled and watertested in three stages of 25 feet each, so that the grout curtain will be tested to the depth of the adjacent key trenches.

If large water takes are observed at any location, additional holes should be drilled on each side to determine the extent of the open zone.

C. Investigation of Key-Trench Fill

As the key trench fill on the right abutment is excavated in accordance with the Panel's recommendation of July 2, detailed studies should be made of the variations in the degree of compaction of the fill material by penetration tests, and samples should be taken for investigation of erosion resistance, stress-strain characteristics, and such other purposes as may become desirable as the investigation proceeds. The specific studies are as follows:

1. Field Investigations and Routine Laboratory Tests

a. Observations and Sampling in Trenches

Immediately upon completion of excavation of an approximately 30-foot long section of exploratory trench, the following observations and sampling should be performed:

With a shovel or spade, make a fresh exposure by removing a vertical slice at least one inch thick, at locations spaced approximately 7 to 8 feet. In this fresh exposure make a rapid survey of variations in consistency along a vertical line, using a screwdriver or other convenient hand tool; also examine variations in types of materials; then perform penetration tests with the Proctor Needle on several representative layers, to define the entire range of strengths, with special attention to the weakest layers or lenses. For the penetration tests on the weakest materials, it will probably be necessary to use the largest diameter "point." Prepare a log of all observations and penetration tests, including thickness of representative layers.

To facilitate recording the logs, it will be desirable to develop a simple classification system which should be based on the BR test data of the Zone 1 fill and on initial experience in surveying the trenches.

b. Sampling

(1) **Hand-cut block samples.** Samples, usually about 8 inches square and about 12 inches high, should be taken of representative materials, but with particular emphasis on the weakest materials. Usually three such samples should be taken at each location, side by side, of material that is essentially similar.

Each sample should be wrapped in Saran wrap, or similar plastic film, and then covered with at least a 1/4-inch thick layer of microcrystalline wax by dipping several times into the wax melted to the correct temperature. (Do not overheat the wax, which would change its properties.)

Use a grade of wax as used in soils laboratories for such purposes. Then place a clearly written identifying label on one side of the sample and again wrap in one layer of plastic film, taking care to place the film smoothly over the label to ensure that it can be read easily.

(2) **A Bag Sample** should be taken at each location where block samples are taken and placed in a plastic bag which is closed tight. Usually about 10 lb. will be sufficient.

(3) **Storage of Samples** should be in a shed with appropriate shelves to provide space for samples taken from an estimated 100 locations and equipped with a humidifier (to maintain humidity at greater than 80% relative humidity) and heated in winter to a temperature above 40°F.

c. Observation of Features That May be Related to Potential or Actual Piping

Special attention must be paid to careful observation of fissures, holes, and any signs indicating that the originally placed fill was disturbed. Such features should be identified, sketched, described and photographed. Particular care should be exercised in identifying such features immediately adjacent to the downstream rock face and the bottom of the key trench. If such features are discovered, it will be necessary to proceed with the greatest of caution in further excavation to protect vital evidence of erosion. At such junctures, the field staff will have to make ad hoc decisions how to proceed. Mr. Jansen should be notified immediately. When particularly meaningful discoveries are made, Mr. Jansen will confer by telephone with available geotechnical panel members.

d. Laboratory Tests

Preferably in a field laboratory, the following tests should be performed on representative samples:

- (1) Natural water content.
- (2) Grain size analyses.
- (3) Liquid and plastic limit tests. (Report actual test results; not the computed plasticity index in lieu of the measured plastic limit.)
- (4) Unconfined compression tests.

e. Miscellaneous Comments

The depth of the exploratory trenches should not exceed 6 feet to facilitate operations.

During removal of fill immediately adjacent to the rock slopes of the key trench, all loose rock should be removed to ensure safety of the men who will work later at lower levels.

2. Evaluation of Erosion Potential of Zone 1 Material

In view of the fact that the failure of Teton Dam has already been attributed to internal erosion of the Zone 1 material, it is important to establish the vulnerability to erosion of this particular material in comparison with that of other soils customarily used as core materials. This is particularly true since visual inspection and classification-test data of Zone 1 materials would appear to indicate that these soils would be highly susceptible to erosion.

To establish the erosion potential of this soil, it is recommended that selected samples be sent to two laboratories for independent evaluation as follows:

- a. A series of 10 samples should be sent to the Waterways Experiment Station at Vicksburg, Mississippi, for performance of the pinhole test as now standardized by that laboratory. Grain-sized distribution curves and liquid and plastic limit values should be

determined for each of the test samples and the results used to establish the relative erodibility of Teton Dam Zone 1 materials.

b. A series of 10 samples should be sent to a second laboratory specializing in measuring the erosion potential of soils (e.g., the Soil Mechanics Laboratory of the University of California at Davis) where the erodibility can be evaluated and compared with data for other soils by means of two or more appropriate types of tests. As before, grain-size distribution curves and liquid and plastic limit values should be determined for each test sample.

In all cases, the erosion tests should be performed on the undisturbed block samples cut from the right abutment key trench. The selected samples should be representative of the range of materials and densities found in the trench, with particular emphasis on materials that appear to be most erodible, as established in the field survey. To the extent practicable, the two independent laboratories should be sent similar suites of samples.

3. Determination of Stress-Strain Characteristics for Use in Finite-Element Analyses

To determine the possibility of hydraulic fracturing or of crack formation in the Zone 1 material, it is desirable to evaluate the stress distribution within Zone 1. This can best be achieved by finite-element analyses incorporating realistic representations of the stress-strain characteristics of the compacted loessial soil used to fill the key trenches and to form the main core of the embankment.

The stress-strain properties should be determined by several series of drained triaxial compression tests on representative samples cut from the Zone 1 section of the dam. At least 3 series of tests should be performed, each series including one test at each of four confining pressures, approximately 15, 40, 70 and 100 psi. Samples should be 1.4 inches in diameter and approximately 3-1/2 inches high and should not be saturated before testing. Stress-strain relationships should be recorded up to the point of failure.

At least one series of the drained tests should be conducted by stress-control techniques to investigate the creep characteristics under loads sustained for several days.

An additional two series of tests should be performed on samples tested as discussed above, but with specimens saturated prior to testing.

Representative grain-size distribution curves and liquid and plastic limit values should be determined for the samples in each series.

D. Embankment Stress Analysis

It is requested that additional finite element stress analyses be made of the embankment fill. This work would constitute an expansion of a pilot analysis submitted to the Panel on August 3, and would incorporate the following specific requirements:

1. Three cross sections of the original right abutment embankment between Stations 12+00 and 15+00, and one axial section of the right abutment embankment (Stations 12+00 to 20+00) should be analyzed. The three transverse stations utilized, and the details of analytical formulation, are to be selected after review of the shape of detailed as-built cross sections.

2. The displayed results should include vertical stress, minor principal stress and strain.
3. The stresses should be those developed by layered construction, as opposed to the "gravity-turn-on" option.
4. In addition, stresses should be calculated to reflect the effect on the embankment of a reservoir rise to Elevation 5300.
5. Two complete sets of stresses should be computed for each section:
 - a. One adopting a core stiffness factor K of 470, as measured by the USBR on a composite, reconstituted triaxial sample under rapid shearing; and
 - b. One utilizing a K of 200, a value judged to be a probable lower limit for the Zone 1 fill.

The foregoing finite element analyses should be undertaken at once, under the guidance of Mr. Leps and Dr. Seed, with a target delivery date of perhaps October 15. Concurrently, a suite of triaxial shear tests on representative samples should go forward, as covered in the previous section, to provide appropriate verification of the K-parameter range assumed in requirement 5, above.

E. Modifications in Program

Field conditions may require modification of some of the details of the recommended program. Moreover, as the findings accumulate, the results may suggest changes, additions, or deletions. The field staff is encouraged to make changes that appear appropriate and to inform the Panel promptly. If major changes seem desirable, the staff should communicate with the Panel.

The full Panel met again for technical working sessions and site inspection on October 4 through 6, 1976. At that time, drilling was requested into the foundation in the vicinity of fissures near Dam Sta. 4+00, described in USBR construction reports. This was in addition to drilling described in the schedule of August 5, 1976. One of the drill holes at that location was to extend into deep underlying sediments where samples could be taken for compression testing. At its October meeting, the Panel decided to have a model fabricated of the right abutment to help in visualizing features relating to the failure. It was also decided to perform hydraulic fracturing tests in bore holes at various sections of the intact portion of the embankment overlying the left abutment.

On October 4, members of the Panel entered the dewatered auxiliary outlet works. The tunnel was inspected for its full length and was found to be in sound condition with no sign of distress which could be related to the failure.

The Panel conducted technical working sessions in the period November 1-3, 1976 with eight of the nine Panel members in attendance. On November 1, inspection was made of the sluiced key trench; of the drilling sites; of the foundation areas uncovered by excavation and sluicing of debris on the right abutment between the spillway and the river. An examination of the remaining lower right canyon wall was made by boat.

Panel technical working sessions were held December 7-10, 1976 with all members participating. The recently completed model of the right abutment was examined. Detailed work was done in drafting the Panel's report due on December 31, 1976.

POST-FAILURE EXCAVATION

Contract DC-7232 was executed for three primary purposes: (1) exploration as necessary in the Panel's investigation of the cause of failure; (2) excavation of a 4,000-ft-long channel downstream from the spillway stilling basin and auxiliary outlet downstream portal for the purpose of permitting internal inspection of the auxiliary outlet and to restore it to service for river diversion; and further to unwater the right abutment for examination, especially in the region of the 50 cfs leak at the right toe of the dam at El. 5045; and (3) resloping the left portion of the dam embankment for public safety and to prevent uncontrolled damming of the river by slides.

All requirements under purpose (1) were determined by the Panel and controlled by the Panel's on-site representatives, acting through the Contracting Officer of the USBR. As suggested above, the Panel's primary interest under purpose (2) was examination of the unwatered auxiliary outlet tunnel, of the lower portion of the right abutment, and of the vicinity of the 50 cfs leak at El. 5045.

Exploration of Zone 1 in Right Abutment Key Trench.

Exploration, excavation, and sampling of Zone 1 materials and examination of the foundation structure in the right abutment foundation key trench proceeded generally as outlined in the Panel's July 2, 1976 letter to Mr. Arthur, with minor on-site modifications.

The near vertical face of the right wall of the breach was sloped for safety in successive vertical lifts to form horizontal working platforms using a 3/4-cu-yd dragline. Materials of all zones in each 5-ft platform to El. 5301 were excavated by a 2-cu-yd backhoe and a 5-cu-yd bucket loader.

A series of longitudinal and transverse backhoe trenches (Fig. 3-1) was excavated to El. 5296, and a series of drive samples was obtained.

Outside the key trench between the spillway and Sta. 12+50 the general foundation level over the full base width of the right abutment remnant was about El. 5295 to El. 5300. The excavation was entirely in Zone 1 at each level below El. 5296 and was made by the 2-cu-yd backhoe and 5-cu-yd bucket loader, also in increments of 5 ft, preceded by transverse trenches at both key trench walls. The transverse trenches were excavated by hand through the final 1 ft of Zone 1 to the rock surfaces. Close inspection, photographing, and mapping were done in these excavations.

Transverse trenches were excavated similarly to expose the key-trench invert whenever excavation neared that depth (Fig. 3-2).

At El. 5280, the 2-cu-yd backhoe was walked from the excavation, while egress was still possible, and replaced with a small combination backhoe and bucket loader. The excavation of Zone 1 materials from the key trench, preceded by exploration trenches at the side walls and invert by backhoe and hand shovel, was made in the same 5-ft vertical increments to El. 5215. Excavated material was hoisted from the key trench by the dragline until it reached its operational limit at El. 5260. Thereafter, material was removed from the key trench in skips hoisted by a truck-mounted crane equipped with a 160-ft boom until it in turn reached its operational limit at El. 5210 (Fig. 3-3). The backhoe was hoisted from the key trench and a small dozer was lowered in turn. The remaining materials were then dozed to the El. 5140 rock bench or to the river's edge as final excavation to rock was accomplished by hand methods.

Below El. 5265, in addition to the transverse trenches, longitudinal exploration trenches were continuously excavated on key-trench centerline, 5 ft from both key-trench walls and at intermediate positions.



Fig. 3-1 Exploration trenches



Fig. 3-2 Transverse trenches exposing key-trench invert and grout cap



Fig. 3-3 **Removal of zone 1 material by crane and skip**

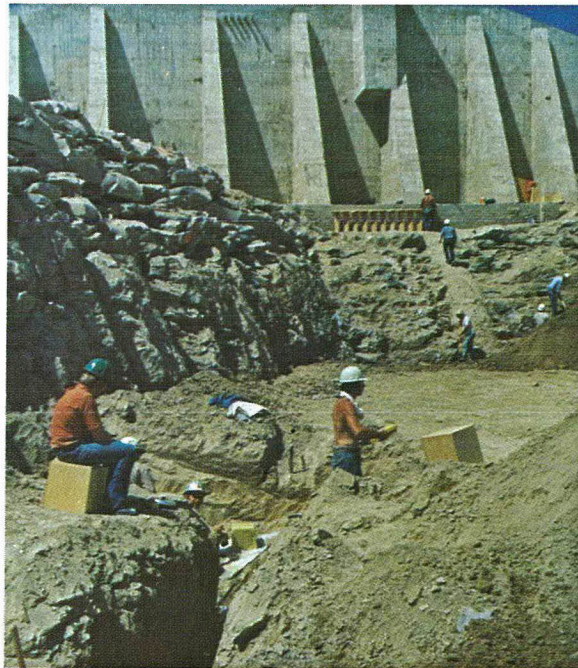


Fig. 3-4 **Obtaining block samples**

Ninety-two 9-in. cube samples and 47 3 in. x 36 in. Shelby tube drive samples were obtained at selected locations (Fig. 3-4).

Final exposure of all rock surfaces was carefully made by hand shovel throughout. Exposures in all trenches were carefully examined for paths of seepage, erosion channels, foundation bond, quality of foundation cleanup, rock nests, extreme variation of materials characteristics, extremely dry or overly wet layers, cracks and other indications of stress or displacement, and the integrity of the grout cap. The rock surfaces were examined and surveyed for joint and fracture patterns, intrusions of soil or extrusions of pre-failure filling, and evidence of pressure grout filling, displacement, or adjustment.

Related location surveys were made. Photographs were taken.

Upon completion of the removal of all soil by mechanical means to the water's edge, the rock surfaces of the key trench and of the right abutment were sluiced clean with fire hose nozzles supplied from water trucks positioned on the abutment near El. 5295.

Observations During Exploration.

The materials comprising Zone 1 appeared to be quite uniform and well compacted. Moisture contents were found to be slightly less than the USBR laboratory optimum. Penetration resistance readings using the Proctor needle varied from 1500 to 2600 psi, and decreased slightly with decreasing elevation of location. Practically all materials classified as nonplastic, inorganic silts (ML). Some visual distinction was possible, mainly in color, with brown, tan, gray, and black being present. The black color was due to a slight organic content in those soils, probably obtained from the near surface layers of the borrow pits. Variations in caliche contents were also present. Sizes larger than the No. 4 screen were practically nonexistent but, when present, were usually caliche clods or small caliche granules. Only one layer, near El. 5265, appeared to be clay, with a plasticity index of 7 and with 93 percent passing the No. 200 screen.

A possible erosion channel was noted adjacent to the upstream wall of the key trench at Sta. 13+00, El. 5261, but upon careful uncovering it proved to be localized and its cause undeterminable.

The first evidence of distress in the compacted fill was noted near El. 5270 and was judged to be localized horizontal slickensides attributable to overcompaction from extensive traffic during placement and abutment wheel rolling in the confined area of the key trench.

Only one vertical longitudinal crack was encountered. It was 1/16 in. to hairline in width, located about 2 ft from the upstream key-trench wall and traceable from El. 5267 to 5280 near Sta. 12+40. This crack may have been caused by differential settlement induced by the narrow horizontal bench on the upstream key-trench wall near El. 5265.

In all respects, the remnant of Zone 1 appeared to be a well-constructed impervious fill meeting all the requirements specified in the contract documents.

The embankment foundation contact in the key trench was excellent and well bonded where observed at many locations in the side wall, transverse invert trenches, and the longitudinal trenches extending to the top of the grout cap. Foundation cleanup was excellent. No rock nests, shattered foundation surfaces, or remaining grout spills were encountered. No dry, pervious, or low density layers or lenses were found.

A few localized, saturated pockets of Zone 1 material were encountered along the upstream wall of the key trench, as were several on the invert of the key trench at the upstream edge of the grout cap

where direct access of reservoir water was afforded by the interconnected joint and fracture structure.

The rock surfaces at the key-trench walls and invert are highly jointed and fractured, but the rhyolite rock is hard, dense, and strong. On the walls the joints and fractures are numerous and closely spaced. The openings are frequent and range up to 1 in., especially above El. 5280.

There was no evidence of the joints and fractures having been surface treated by slush grouting. The Zone 1 fill where placed against the open joints was found to bridge across them. Some local overhangs of limited extent were present under which the Zone 1 material was found in an uncompacted and saturated state.

As the Zone 1 fill was progressively and alternatively explored by trenches and excavated full width, it was found intact and undisturbed from El. 5332 to 5265. At El. 5265, the embankment was found to be cracked transversely at vertical and steeply dipping angles. Well-defined shear zones appeared. Hydraulically transported filling was found in some of the cracks. Wet clay coatings were also present. It was concluded that these cracks were associated with incipient sliding of the remnant of fill toward the face being eroded by the flood waters and that the filling was due to the flow of bank storage into the cracks as the failure progressed. Hence, the cracks were judged to be due to the consequences of the failure.

Finally, at the lower elevations, near El. 5225 and the rock bench at El. 5220, the well-defined, concentrated cracks disappeared, but the shear pattern became more intense and extensive until the embankment everywhere exhibited distress for horizontal distances in excess of 20 ft from the face of the breach. The shearing pattern was diamond-shaped, and the general configuration formed cupped or bowl-shaped surfaces concave toward the river, with the surfaces gradually becoming subtangent to the key-trench walls.

The longitudinal exploration trenches exposed the bench at 5220 and extended to the deeper key-trench invert beyond. Here the sheared zones were found concentrated at the key-trench profile break and appeared to be controlled by that break.

Near Sta. 13+15, at El. 5215, the embankment for the first time was found extremely wet continuously across the width of the key trench. Some free water was encountered. The fill was extremely muddy over the surface of the grout cap. Between the grout cap and the upstream key-trench wall, the backhoe sank up to the axle. Even under the lighter ground pressure of the small dozer, the fill was spongy and quick. The in-place embankment remaining at this elevation was very limited in axial extent, being about 15 ft. A transverse vertical face was cut by hand 3 to 4 ft to the key-trench invert rock. By probing over this vertical surface, a softer, wetter horizon was detected. Penetration resistance readings were in the 170-psi range while readings above were in the 400-psi range and those below averaged 330 psi. Because this horizon was everywhere within 15 in. of the rock, and in such close proximity to the face of the breach it was not possible to determine if this wetter horizon existed pre-failure or was created during the failure.

At Sta. 13+25 and El. 5206 on centerline of grout cap, the in-place embankment terminated, and all of the soil then remaining on the abutment foundation was identified as disturbed material which had sloughed down from the steep face of the breach.

Beyond that location, all the remaining soil on the abutment was gradually removed by the small dozer pushing the soil either to a stockpile on the bench at El. 5140 or completely down to the edge

of the river. By hand shovel, the grout cap was exposed ahead of the dozer operation to avoid any possible damage or displacement of the grout cap.

Care was also used in removing the soil immediately adjacent to the rock by hand, initially without water, so that any existing clues to the cause of failure might not be accidentally destroyed. The rock surfaces were then sluiced clean as previously described.

Channel Excavation.

Following the failure, the river flow stabilized with the reservoir at about El. 5056 and an intermediate pool in the breach at approximately El. 5053. The level of the intermediate pool was controlled by an extensive bar of large rocks. The auxiliary outlet portal was blocked by debris deposited in the stilling basin; consequently, a trapezoidal channel bypassing the bar was excavated, commencing 4,000 ft downstream from the stilling basin, and was completed sufficiently by September 27 to attempt a controlled lowering of the intermediate pool by gradual removal of the portion of the bar near the stilling basin which had been partially reinforced as a cofferdam at the head end of the bypass channel. Unfortunately, the cofferdam eroded very rapidly, lowering the intermediate pool to El. 5036 with consequent rapid erosion of Zone 1 of the left remnant in the river channel. To avert uncontrolled releases of the remaining reservoir storage, the cofferdam was quickly reestablished, again raising the intermediate pool to El. 5053 and arresting the erosion of Zone 1.

A temporary gated, double-barrelled culvert control structure of 1,000-cfs capacity was then constructed in the river bypass channel. After testing it by closing the gates and filling the lower pool thus formed at the spillway stilling basin, the cofferdam was removed and the river channel at the dam was slowly excavated to permit controlled draining of the reservoir through the bypass control structure. In this manner, the residual reservoir and intermediate pool were reduced to a negligible capacity by lowering the river channel invert, and the remaining abutment and the vicinity of the leak at El. 5045 were unwatered for inspection (Figs. 2-5 and 2-6).

SOIL SAMPLING AND TESTING

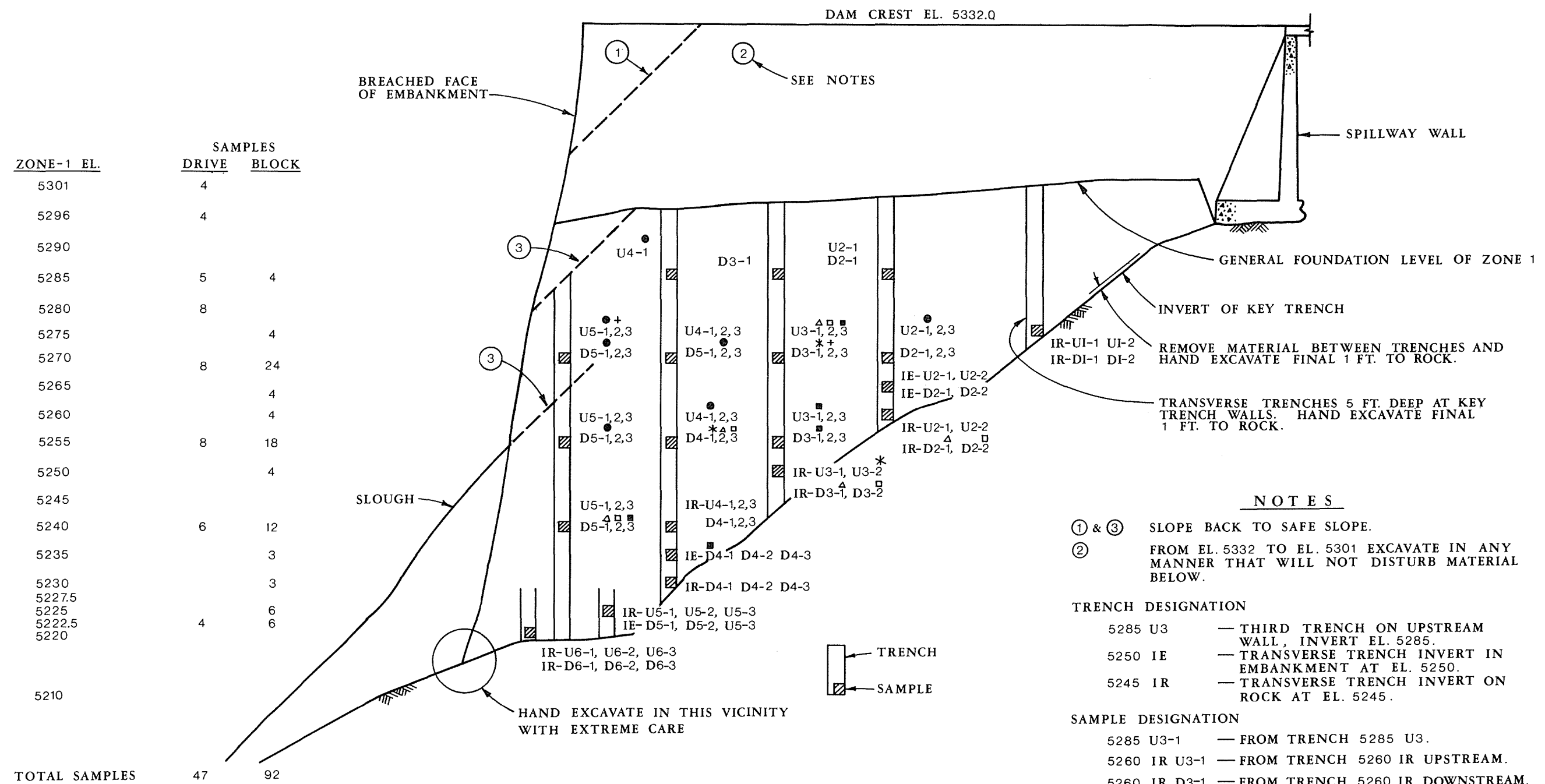
Undisturbed, hand-cut block samples, 9 in. x 9 in. x 9 in. in dimension, and 3 in. x 36 in. Shelby tube drive samples together with 10-lb bag samples taken nearby were obtained at the locations shown in Fig. 3-5.

Selected block samples, representative of the range of materials and densities found, and spanning the mass of the embankment remnant on the right abutment, were sent to various laboratories for identification tests and tests of designated engineering properties. To the extent practicable, two laboratories were sent similar samples for comparative purposes.

The dispersive characteristics of Zone 1 material were investigated by pinhole tests at the Waterways Experiment Station and the erodibility by flume tests and rotating cylinder tests by the University of California at Davis.

The stress-strain properties were investigated by drained triaxial compression tests at both placement moisture and saturated moisture contents by Northern Testing Laboratories, Billings, Montana, and by the Earth Sciences Branch, USBR, Denver, Colorado. Unconfined compression tests at varying moisture contents were also made by the latter.

Special horizontal permeability tests were made by the University of California at Berkeley.



NOTES

① & ③ SLOPE BACK TO SAFE SLOPE.

② FROM EL. 5332 TO EL. 5301 EXCAVATE IN ANY MANNER THAT WILL NOT DISTURB MATERIAL BELOW.

TRENCH DESIGNATION

5285 U3 — THIRD TRENCH ON UPSTREAM WALL, INVERT EL. 5285.

5250 IE — TRANSVERSE TRENCH INVERT IN EMBANKMENT AT EL. 5250.

5245 IR — TRANSVERSE TRENCH INVERT ON ROCK AT EL. 5245.

SAMPLE DESIGNATION

5285 U3-1 — FROM TRENCH 5285 U3.

5260 IR U3-1 — FROM TRENCH 5260 IR UPSTREAM.

5260 IR D3-1 — FROM TRENCH 5260 IR DOWNSTREAM.

TRENCH AND SAMPLE LOCATIONS ARE SCHEMATIC ONLY.

TESTING PROGRAM

SAMPLE RECEIVING LABORATORY TEST	+	*	●	△	□	■
U.C. - BERKELEY	NORTHERN TESTING	U.S.B.R.	U.S.C.E. - W.E.S.	U.C. - DAVIS	U.C. - DAVIS	
HORIZONTAL PERMEABILITY	DRAINED TRIAXIAL FIELD MOISTURE SATURATED	DRAINED TRIAXIAL SATURATED FIELD MOISTURE	PINHOLE DISPERSION	ROTATING CYLINDER EROSION	FLUME EROSION	
		STRESS CONTROLLED UNCONFINED COMPRESSION				

EXPLORATION OF ZONE 1 AND FOUNDATION KEY TRENCH

FIG. 3-5. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Gradation analysis and Atterberg limit determinations were made on all samples by the Teton Project Laboratory, including those samples shipped to the other laboratories for testing.

The results of all tests are discussed in Chapter 7, and the complete reports have been placed in the Panel's records. Samples not tested are stored at the USBR laboratories in Denver.

EMBANKMENT STRESS ANALYSIS

Interest developed early within the Panel as to the possibility of tension cracking of Zone 1 transversely within the key trench due to arching between the steep side walls of the narrow trench or due to differential settlement at any abrupt changes in the longitudinal slope of the key-trench invert, or due to the tendency of the embankment mass to pull away from the abutments as the dam settled. The Panel recognized that the state of stress within the embankment due to these factors would be intimately associated with and influenced by the intergranular forces imposed as the reservoir filled and saturation gradually spread through the embankment volume. A two-dimensional pilot study of the state of stress within Zone 1 at Sta. 14+00 was made at the Panel's request by Dynamic Analysis Corporation, Saratoga, California. The finite element analytical methods for soils developed in recent years primarily by the University of California at Berkeley were employed.

The results of these pilot studies, available to the Panel at its August meeting, were considered sufficiently revealing to warrant expanding the studies to three transverse sections at Stas. 12+70, 13+20, and 13+70 and to a longitudinal section along the key trench from Sta. 12+00 to Sta. 20+00. The University of California at Berkeley also undertook a two-dimensional finite element stress analysis of the embankment at Sta. 15+00. The results of these analyses are included in Appendix D and reviewed in Chapter 12.

HYDRAULIC FRACTURING TESTS IN BOREHOLES

Hydraulic fracturing tests were made in boreholes in the left portion of the remaining embankment at stations where the geometry of the key trench and the height of the overlying embankment were similar to those at the stations where the initial breach of the right key-trench fill occurred. The principal purpose was to determine, by comparing the results of the field tests with those of calculations, appropriate in-situ values of soil properties needed for finite-element analyses of stress conditions in the right key trench.

Three tests were performed. Sta. 26+00 was selected for the first test upon determining that the key trench and embankment-foundation geometry were analogous to that of Sta. 15+00.

The test procedure involved drilling a vertical hole directly over the key-trench centerline to a predetermined depth and subjecting an exposure of Zone 1 over a selected length of the hole near the bottom to a gradually increasing head of water. The length of hole so pressured was restricted by sealing an internal standpipe in the drill hole with a cement plug at the top of the length selected for testing and introducing water into the standpipe.

By observing the recession rate of the imposed head, a normal rate of seepage for the conditions established was determined. The head was then increased by increments and the recession rates observed. If an increment was reached for which the recession rate suddenly increased by a larger magnitude, the fill in the region of the hole was deemed to have been fractured by the hydrostatic pressures created by the head then imposed.

At Sta. 26+00 it was believed that the hole could be safely wash-bored to 150 ft and the plug set at that depth (Fig. 3-6). However, at 101.3 ft (El. 5211.7) a sudden loss of drill water occurred. Fracturing is believed to have occurred at that elevation and head. Through a misunderstanding, drilling of the hole continued to a depth of 150 ft with continued loss of drilling water and the injection of several thousand gallons of water into the adjacent soil. A 3-in. plastic pipe was sealed in the hole with the cement plug and the hole was extended for 39 ft beyond by drilling with air. Soil wetted by the previous drill water loss became lodged behind the drill bit and in forcefully freeing the drill string the plastic pipe was pulled from the plug. The hole was temporarily abandoned.

A second attempt was made at Sta. 26+25 by drilling a 4-in. hole with air to a depth of 150 ft, sealing a 3-in. plastic pipe in the hole with a cement plug at El. 5163, and extending the length of hole to be pressured 20 ft to El. 5143 by using air to facilitate drilling. Again wet drill cuttings lodged behind the drill bit, this time causing a momentary increase in air pressure, apparently sufficient to fracture the hole as evidenced by the sudden entry of water into the hole, most likely from the adjacent hole at Sta. 26+00.

Because Sta. 13+70 had also been analyzed and because Sta. 27+00 was analogous, a third test hole was located at that station and augered 109 ft to El. 5210. Nx casing was sealed in the hole at that elevation, and the hole was extended 20 ft with a split spoon drive sampler. The hole was then incrementally pressured as previously described. The test results are shown on Fig. 3-7.

As revealed by the water level recession rates, no fracturing occurred even with the water level raised to the top of the hole at El. 5317.

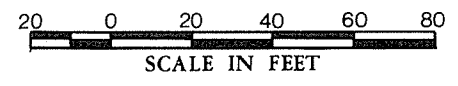
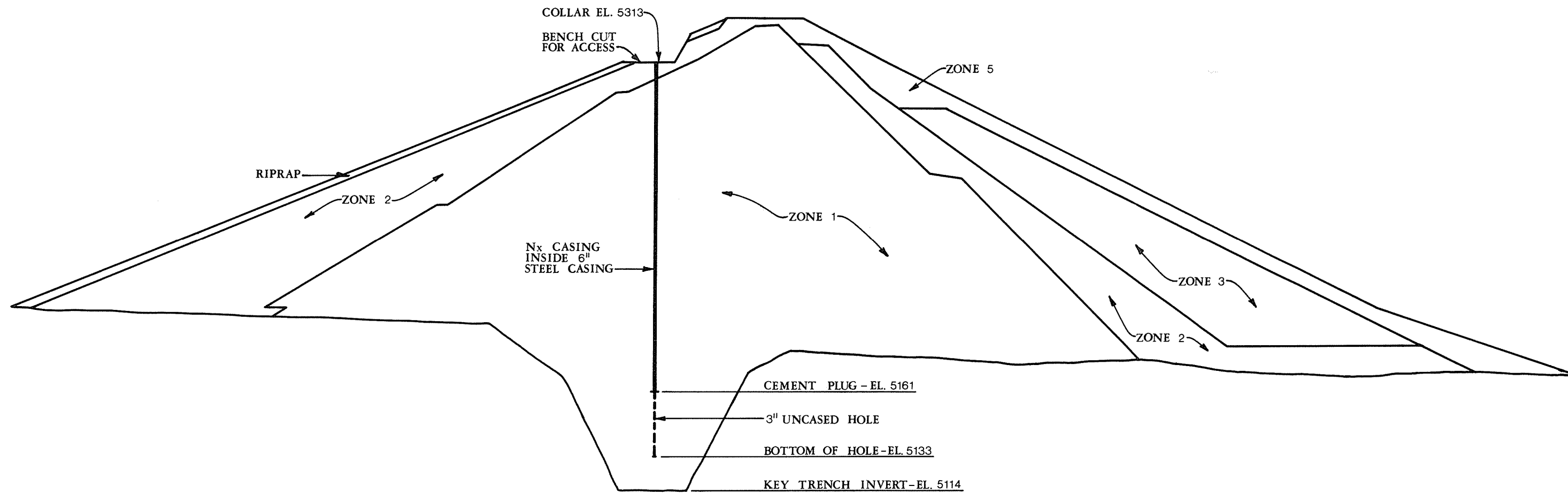
The hole at Sta. 26+00 which had been originally cased to 150 ft with 6-in. casing was restored by sealing Nx casing with a new plug at 152 ft and by cleaning out the original 39-ft-long hole extension with the split spoon drive sampler for 28 ft. The hole was then tested. Although some sloughing of the hole may have taken place during the test, the average head was assumed measured to the midheight of the restored hole, or El. 5147. The results are shown on Fig. 3-8 and indicate that the Zone 1 fill was fractured when the water surface was 20 ft below the top of the hole, or El. 5293.

POST-FAILURE FOUNDATION INVESTIGATION

Early in its investigation the Panel recognized the desirability of identifying the most probable path or paths of the leakage that led to failure of the dam. Efforts were directed to determine whether critical leakage had passed through, around, or under the dam, or had followed a combination of routes; also to establish the precise path or paths insofar as possible from the evidence remaining at the site.

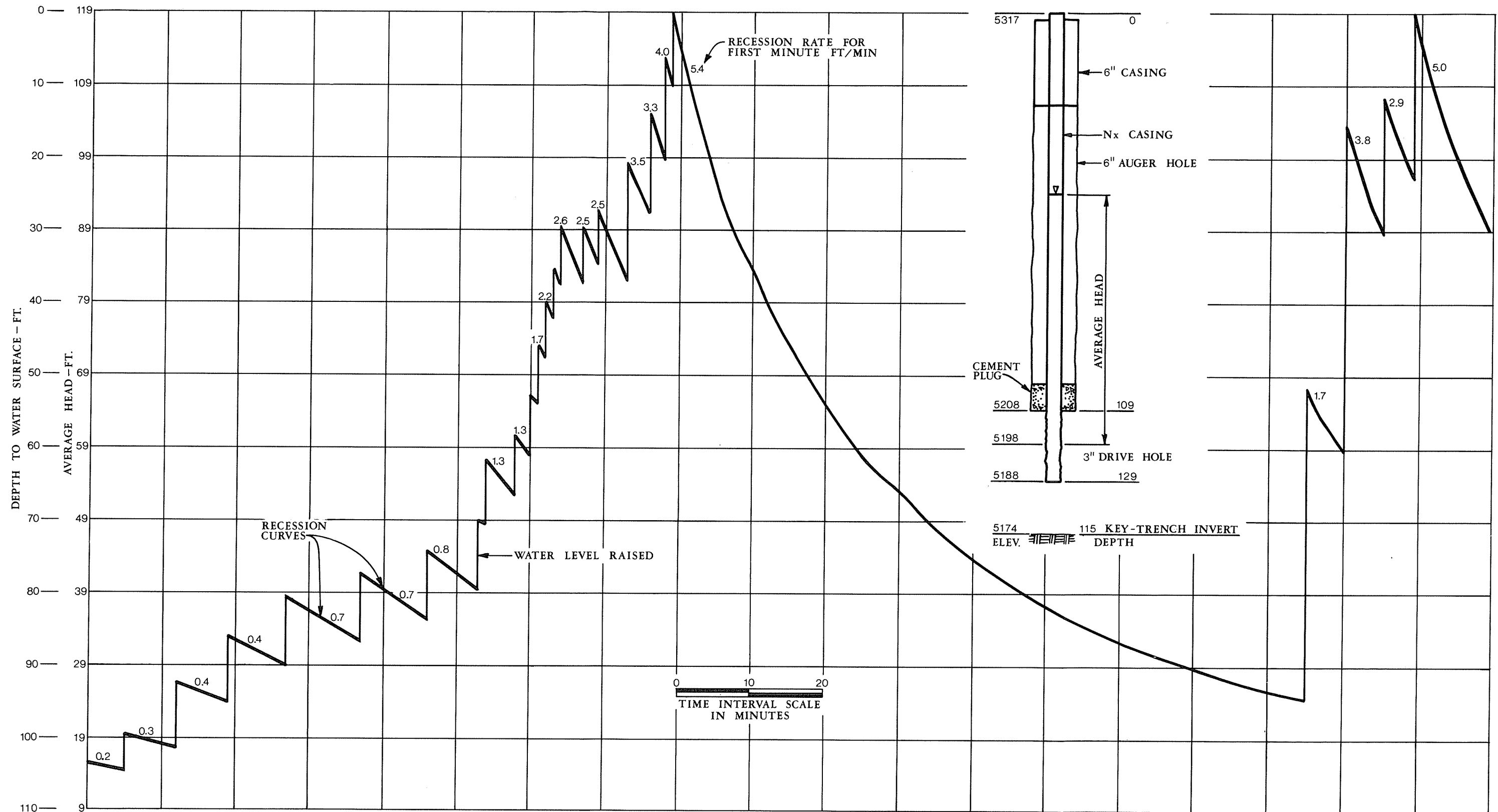
A geologic program was developed to investigate the following possible avenues of leakage through the foundation:

1. Around the right end of the dam.
2. Through the grout curtain.
3. Through large cavities discovered near the right end of the dam during its construction.
4. Through sedimentary deposits underlying the volcanic rock foundation.



BORE HOLE STA. 26+00

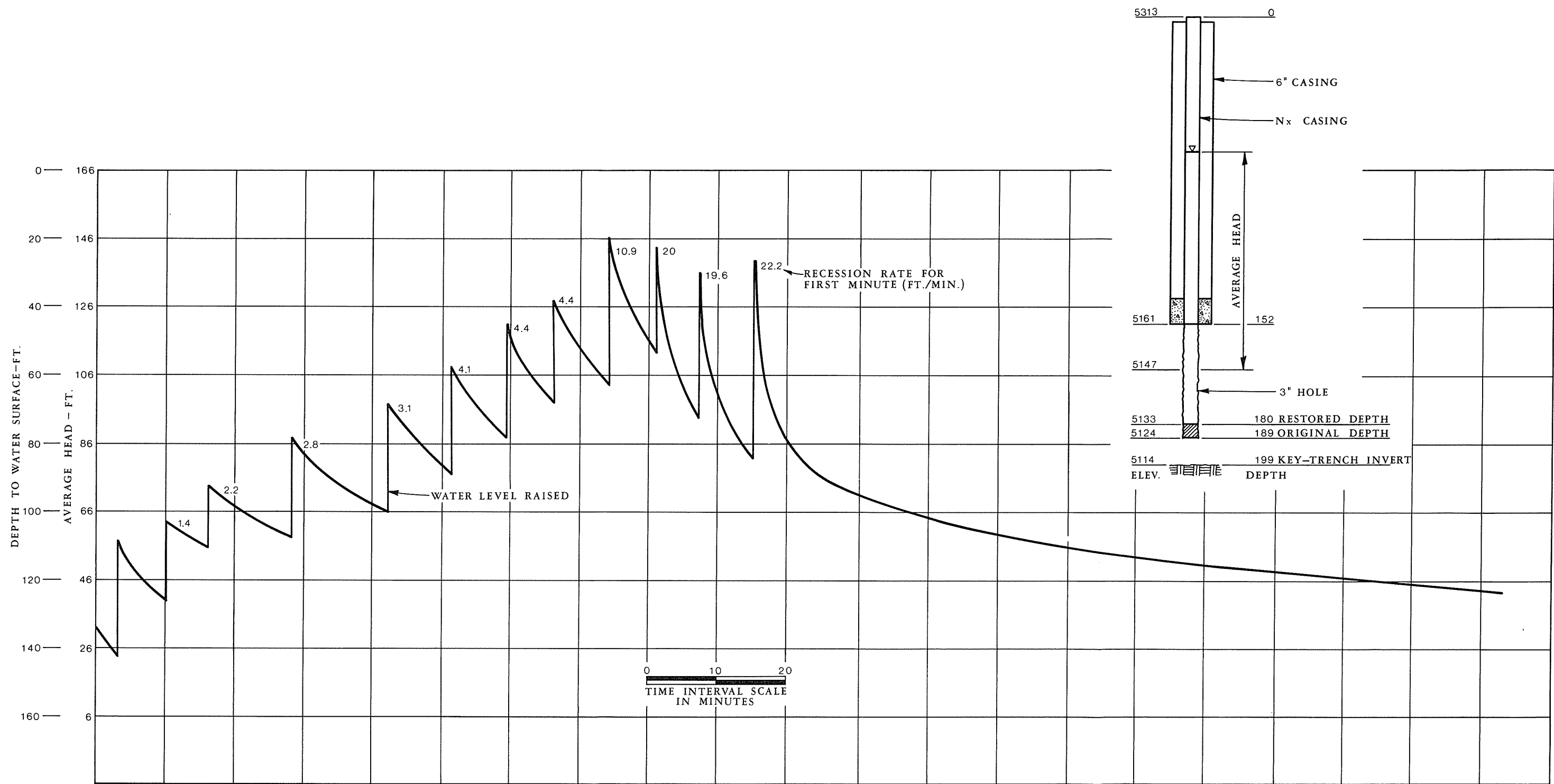
FIG. 3-6. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



BORE HOLE HYDRAULIC FRACTURING TEST,
STA. 27+00 WATER LEVEL RECESSION RATES

FIG. 3-7.

U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



BORE HOLE HYDRAULIC FRACTURING TEST
 STA. 26+00 WATER LEVEL RESSION RATES

FIG. 3-8. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
 INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Investigation of Possibility of Flow Around End of Dam.

As an initial step, a review was undertaken of the records of subsurface exploration performed by the Bureau of Reclamation prior to construction of the project. This effort produced convincing evidence that flow around the dam was possible through interconnecting joints, but the Panel concluded that at the time of the failure there was insufficient hydraulic gradient between the end of the dam and the leak at El. 5200 to account for the large flow that was estimated to have broken out at this point. The analysis leading to this finding is developed in Chapter 5.

Investigation of the Grout Curtain.

At its August 5th meeting, the Panel developed a program for the investigation of the key-trench invert and of the grout curtain. The purpose of the program was to determine if the joints and fractures intersecting the key-trench invert could pass water and if the grout curtain beneath the grout cap and under the spillway weir was watertight. The section tested in the right abutment key trench extended from grout cap centerline Sta. 12+60 to 14+26.*

The removal of all the loose soil covering the right abutment revealed the post-failure condition of the grout cap (Fig. 3-9). The cap was found intact and continuously in position from the spillway to Sta. 13+96. Between Stas. 13+96 and 14+26, the cap was entirely missing (Fig. 3-10). However, the original rock invert of the trench in which the cap concrete had been placed was undisturbed, as shown by the preservation of the original line drill holes. Three prominent, nearly vertical joints, striking approximately N20°W, cross this gap, the ends of which appeared to be determined by the presence of the two joints at the ends of the gap (Fig. 3-11). A 2-in. open vertical fracture striking S68W (Fig. 3-12) crosses the alignment beneath the cap at Sta. 13+90. This fracture was ponded for a joint transmissibility test and is identified as U13 in Table 3-1.

Between Stas. 13+30 and 13+96 (Figs. 3-13, 3-14, and 3-15) where the original side walls of the key trench were washed away during the failure, the cap exhibited a remarkable degree of resistance to the failure forces as evidenced by the large amount of rock plucking immediately adjacent both upstream and downstream and the surface erosion of the concrete.

The cap concrete was eroded flush with the adjacent rock surfaces between Stas. 14+26 and 14+46. Extensive concrete erosion was also present between Stas. 14+65 and 14+75. This erosion is attributed to flows occurring during failure. Beyond Sta. 14+95 the cap is again missing at least as far as the river level at El. 5052.

Wherever the original contact between cap concrete and rock foundation was exposed, it was found tight and intimate, indicating the absence of pre-failure displacement or separation. This observation was particularly striking at Stas. 14+26 and 14+65.

Transverse cracks, only one of which was more than hairline width, are spaced approximately 20 ft apart along the grout cap (Appendix E). These cracks are attributed to shrinkage during loss of heat of hydration with the possible exception of those cracks near 13+90 which could have been caused by a slight rotational force on the cap at the time the cap was severed at 13+96.

*Because they are on differing lines, the stations along the grout cap and along the axis are not the same.



Fig. 3-9 Post-failure exposure of the grout cap



Fig. 3-10 Grout cap severed at Sta. 13+96 and missing to Sta. 14+26. Open fracture shown in Fig. 3-12 is behind the ladder



Fig. 3-11 Three vertical joints crossing alignment where grout cap is missing between Stas. 13+96 and 14+26

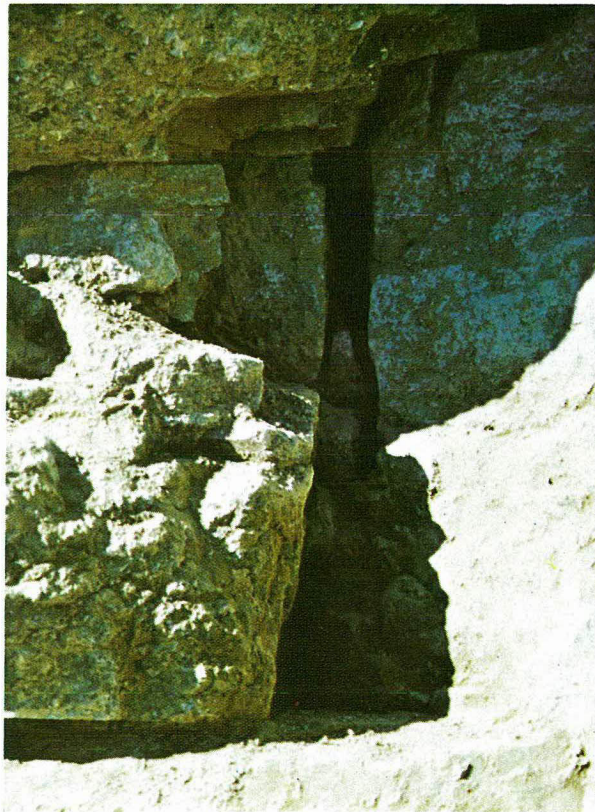


Fig. 3-12 2 in. open fracture crossing grout cap alignment near Sta. 13+90 (see Fig. 3-10 for location)

TABLE 3-1
JOINT TRANSMISSIBILITY TESTING

Joint No.	Dip	Strike	Location, Intersection of Strike w/Grout Cap \bar{C} Sta.	Submerged Length of Joint ft	Max Depth of Water ft	Volume of Pond gal	Loss gpm
U1	35W	N25W	13+50	5.4	0.7	37.7	0.01
U2	31W	N25W	13+45	4.4	0.7	12.0	None
U3	49W	N25W	13+41	3.4	0.6	3.3	None
U4	44W	N30W	13+32				
Pond 1				2.6	0.5	2.4	None
Pond 2				1.8	0.4	2.3	None
Pond 3				2.1	0.6	4.6	None
U5	45W	N20W	13+30				
Pond 1				3.0	0.6	3.1	0.09
Pond 2				1.2	0.4	0.4	None
Pond 3				2.0	0.8	4.6	0.38
U6	45W	N30W	13+27	2.2	0.5	18.0	1.09
U7	21W	N25W	13+23				
Pond 1				4.1	0.5	4.4	0.01
Pond 2						2.1	None
U8	38W	N20W	13+15	2.4	0.4	2.9	None
U9	30W	N15W	13+03	2.7	0.4		None
U10	25W	N 8W	13+00	8.0	0.6	26.4	0.10
U11	25W	N 5W	12+93				
Pond 1				2.7	0.5	16.3	0.08
Pond 2				3.4	0.7	10.7	None
U12	90	N24W	13+96	27.5	1.0	120.1	None
U13	90	S68W	13+90	3.7	3.7		>28.5
D1	25W	N10W	13+50	5.8	0.8	14.2	0.58
D4	42W	N13W	13+32	4.9	0.9	—	—
D5	36W	N25W	13+30	6.0	0.6	6.8	0.21
D7	78E	N33W	13+23	3.9	1.0	32.1	0.09

Notes:

- Ponds numbered from upstream.
- Stations on grout cap centerline.
- Joint U1 – Return flow from intersecting joints upstream of grout cap.
- Joint D1 – Joint truncated at outcrop affording free egress.
- Joint D4 – Could not effectively pond joint.
- Joint D5 – Return flow appeared downstream on canyon wall, Sta. 13+50, 20 ft downstream, El. 5185.
- Joint U6 – Return flow appeared from joint 6 ft downstream of grout cap, N30W, 77E. Pond also submerged a fracture, N50W, 72E.
- Joint U5 – Pond 3 – Return flow at same location as from Joint U6.
- Joint U10 – Return flow from intersecting joints upstream of grout cap.
- Joint U13 – Capacity of water supply was 28.5 gpm. Could not raise water surface higher. Return flow appeared along horizontal joint, Sta. 13+97, El. 5155, 2.5 and 5.5 ft downstream of grout cap.



Fig. 3-13 Rock structure along grout cap alignment Stas. 13+30 to 13+96



Fig. 3-14 Rock structure along grout cap alignment Stas. 13+30 to 13+96, upstream profile



Fig. 3-15 Rock structure along grout cap alignment Stas. 13+30 to 13+96, downstream profile



Fig. 3-16 Test ponds for joint transmissibility tests, looking downstream

Joint Transmissibility Testing.

The transmissibility characteristics of the rock foundation at the grout cap were first tested under a low gravity head by constructing small impoundments with mortar and stone over selected joints and fractures (Fig. 3-16). The ponds and joint traces were cleaned with air and water. The volume of the ponds and any loss rates were determined by metering water into the ponds. The dip, strike, and submerged length of each joint were measured. Twelve ponds were prepared upstream and four downstream of the grout cap between grout cap Stas. 12+93 and 13+96 (Fig. 3-17). These ponds were not in contact with the cracks in the grout cap. Also, as previously reported, one vertical joint (U13) where exposed by rock plucking over a considerable height below the top of the grout cap near Sta. 13+90 was tested by constructing a vertical riser against the rock face and about the joint (Fig. 3-18). The loss rate results and associated data for these tests are shown in Table 3-1.

The tests revealed that water could pass freely through the shallow joints and fractures beneath the grout cap at several locations in the vicinity of grout cap Stas. 13+27, 13+30, and 13+90 under the low gravity heads imposed by the tests.

Grout Curtain Testing in Foundation Key Trench.

The watertightness of the grout curtain at depths below the base of the grout cap and at the concrete-rock interface was tested by drilling Nx-sized holes at selected locations and inclinations along the grout cap centerline after the joint transmissibility ponding tests had been completed.

The locations initially designated by the Panel at Stas. 12+65, 13+05 and 13+40* were augmented on the basis of the results obtained in testing the original holes and in the joint transmissibility tests. The additional holes were located at Stas. 13+30, 13+77, 14+10, and 14+26. Inclinations were chosen for optimum intersection angles with the predominant joint planes.

The holes were staged downward, usually in two stages of approximately ten-foot lengths. The upper stage was tested with the packer set above the concrete-rock interface to include the effect of the watertightness of the contact. Applied pressure at the collar of the hole was limited to 10 psi in all tests except in one instance. In that case, upper stage, DH-621, the pressure was raised to 18 psi for comparison of loss rates at 10 psi pressure. The rate increased from 7.9 gpm to 13.0 gpm under those conditions.

After pressure water-testing by stages, each hole was subjected to a gravity water test by equalizing inflow with outflow as observed by stabilizing the water level at the top of the hole.

At each hole cluster, usually three holes per cluster, the hole previously tested was filled with thick grout after testing to avoid undetected water escape routes by possible hole interconnections when testing the next hole. Upon completion of testing of all holes in the cluster, the last hole tested was left open for possible future examination.

Twenty-three holes were drilled and water tested. The results and associated data for these tests are shown in Table 3-2.

The tests revealed that water could pass through the rock structure beneath the grouted portion of the key-trench foundation at the depth tested. The larger losses (up to a maximum of 14.1 gpm) occurred in the upper stages (10 ft below base of grout cap). Maximum loss observed at 10 psi pressure in the second stage (10-20 ft below base of grout cap) was 5 gpm. Return flows were observed from joints and fractures downstream from the grout cap. The greater losses occurred from the angle holes.

* Holes were actually drilled at Stas. 12+75, 13+15, and 13+50 respectively

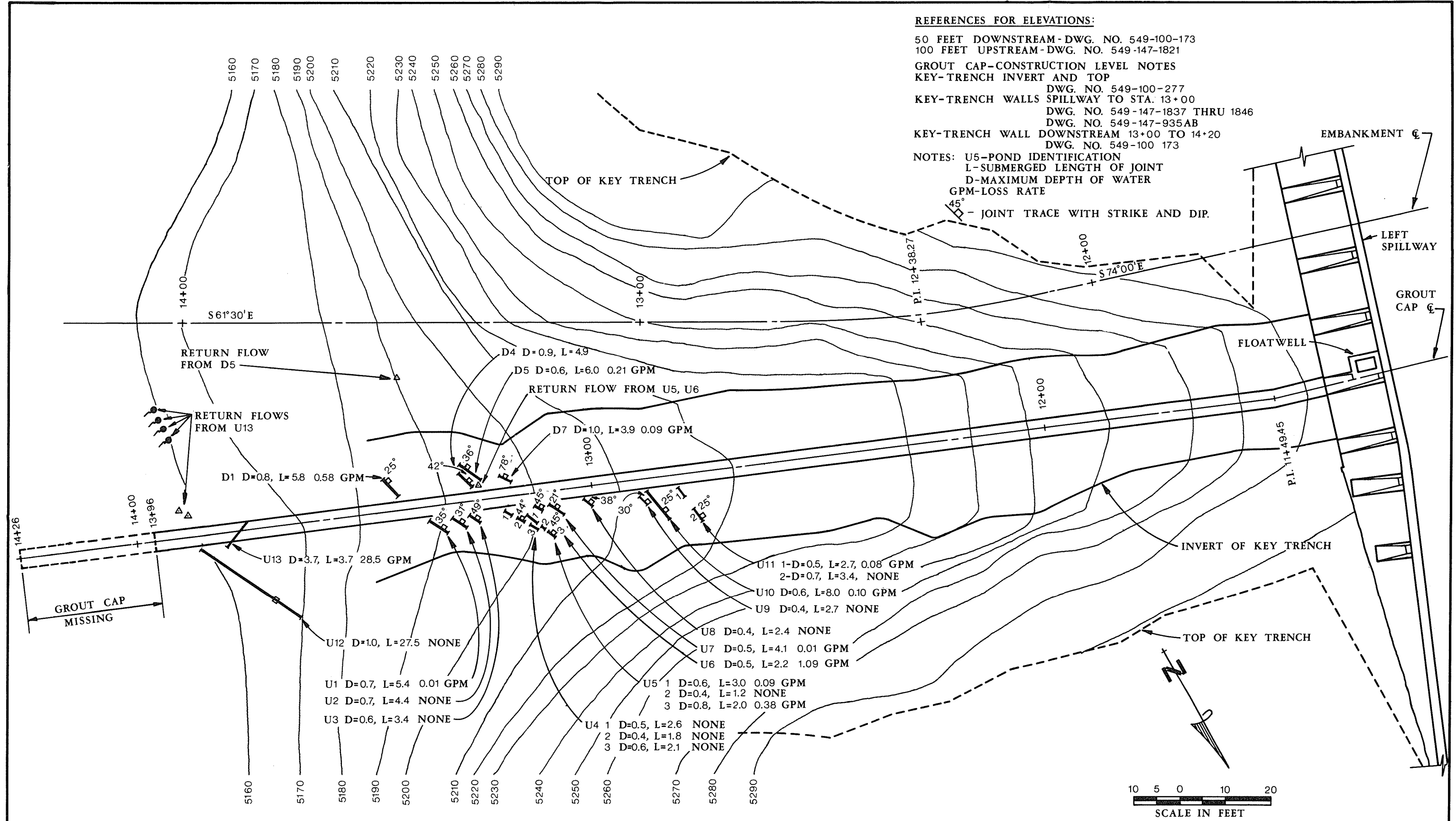
REFERENCES FOR ELEVATIONS:

50 FEET DOWNSTREAM - DWG. NO. 549-100-173
 100 FEET UPSTREAM - DWG. NO. 549-147-1821

GROUT CAP-CONSTRUCTION LEVEL NOTES

KEY-TRENCH INVERT AND TOP DWG. NO. 549-100-277
 KEY-TRENCH WALLS SPILLWAY TO STA. 13+00 DWG. NO. 549-147-1837 THRU 1846
 KEY-TRENCH WALL DOWNSTREAM 13+00 TO 14+20 DWG. NO. 549-147-935AB
 DWG. NO. 549-100 173

NOTES: U5-POND IDENTIFICATION
 L-SUBMERGED LENGTH OF JOINT
 D-MAXIMUM DEPTH OF WATER
 GPM-LOSS RATE
 45° - JOINT TRACE WITH STRIKE AND DIP.



JOINT TRANSMISSIBILITY TESTING

FIG. 3-17

U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
 INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

REFERENCE DATA:
 U. S. BUREAU OF RECLAMATION



Fig. 3-18 Vertical brick riser for ponding test at Sta. 13+90

TABLE 3-2
 GROUT CURTAIN WATER LOSS TESTING
 IN RIGHT ABUTMENT KEY TRENCH

Location			Pressure Tests				Gravity Tests	
Sta	Hole DH-	Angle	Interval ft	Water Loss gpm	Interval ft	Water Loss gpm	Interval ft	Water Loss gpm
12+75	610	42°R	3.8-14.8	0.38	14.8-24.8	0.05		
	611	20°R	2.5-13.9	0.48	13.9-23.9	0.1		
	612	0°	2.1-13.5	0.5	13.5-23.5	0		
13+15	613	45°R	3.4-14.8	1.1	14.8-24.8	0.2		
	614	22°R	4.4-14.4	1.0	13.4-24.4	0.9	0-24.4	0.5
	615	0°	3.5-14.9	0	14.9-24.9	0		
13+30	616*	47°R	2.9-14.3	10.	7.9-14.3	0.5	0-14.3	3.5
			14.3-24.3	5.2	18.9-24.3	0	0-24.3	4.5
	617	21°R	2.5-13.9	0	13.9-23.9	0	0-23.9	0.4
	618	0°	2.8-14.2	0	14.2-24.2	0.8		
	619*	31°L	5.3-16.7	1.6	16.7-26.7	4.8	0-26.7	4.8
13+50	620	44°R	4.4-14.4	0.1	14.4-24.4	0.1		
			24.4-34.2	0.1				
	621	22°R	3.5-13.5	5.1	13.5-23.5	0.1		
			3.5-23.5	7.9	3.5-23.5	13.0	(20 psi)	
	622	0°	2.1-13.5	1.9	13.5-23.5	0		
623*	22°L	3.4-13.4	12.7**			0-13.4	14.1**	
		11.4-21.4	1.2	6.4-21.4	1.6			
13+77	624*	45°R	2.9-14.3	3.5	14.3-24.3	0	0-24.3	1.8
	625	0°	3.9-15.3	0.4	15.3-26.3	0	0-26.3	0.2
	626*	22°L	5.3-16.7	12.5	16.7-26.7	7.5	(2 psi)	
						0-26.7	4.5	
14+10	627	46°R	1.6-11.0	0.2	11.0-21.0	0.1	0-21.7	0.03
			4.6-11.0	0	11.0-21.0	0.14	0-21	0.1
	629*	34°L	1.0-11.0	7.1	2.6-11.0	0.7	0-21	0.3
			11.0-21.0	0.7				

TABLE 3-2 (cont.)

Location			Pressure Tests				Gravity Tests	
Sta	Hole DH-	Angle	Interval ft	Water Loss gpm	Interval ft	Water Loss gpm	Interval ft	Water Loss gpm
14+26	630*	47°R	1.3-11.7	4.3	11.7-21.7	0.4	0-21.7	0.9
	631	0°	1.2-11.6	0.3	11.6-21.6	0.2	0-21.6	0.03
	632	34°L	1.7-13.1	2.6	13.1-23.1	1.6	0-23.1	0.7

Interval is measured along axis of hole from the collar.

All pressure tests conducted at 10 psi at collar elevation except where noted.

R and L designate holes angled to the right and to the left, respectively.

* Return flows observed from joints and fractures downstream of grout cap.

**During drilling a 50 percent water loss occurred at a depth of 2.4 ft or 1 ft below the base of the grout cap.

Geologic logs are contained in Appendix F.

Grout Curtain Testing at Spillway Weir.

The spillway weir cutoff serves as the grout cap beneath the spillway control structure and affords continuity of the grout cap beneath the embankments flanking each side of the spillway. During construction the rock formation beneath the entire area of the spillway control structure was treated extensively by blanket grouting to a depth of 80 ft on a primary hole spacing of 15 ft and closed to 7-1/2 ft. The grout curtain beneath the spillway weir location was then grouted after the blanket grouting was completed. That curtain was slightly modified to avoid interference with the access shaft of the auxiliary outlet by placing the center and upstream row of holes on the same alignment as the spillway weir cutoff. (See Chapter 9.)

As part of the Panel's investigation, nine holes were drilled and water tested, three each at the center of each spillway bay at dam crest centerline Stas. 10+82, 11+06, and 11+30. One hole in each bay was angled to the left, one to the right, and one vertical. The grout curtain in this vicinity was tested for watertightness with some modifications suited to the above conditions. The water test holes were located just upstream of the center row of curtain grout holes. The water test holes were drilled and water tested in three stages of 30 ft each for the vertical holes and three stages of 35 ft each for the inclined holes in order that the final stage would extend beyond the depth of the consolidation grouting.

Water test procedures and pressures were the same as those used for the foundation key trench. Every hole was filled with grout after being tested to avoid creating water escape routes by interconnections between holes. The results and associated data for these tests are shown in Table 3-3 and Appendix F.

DH-609 was extended an additional stage to a length of 145 ft to examine the region where the consolidation grouting pattern terminated and the curtain grouting pattern continued.

The tests indicated that the rock formation beneath the spillway control structure as grouted is reasonably impermeable within generally accepted standards.

Grout Curtain Testing Near Right End of Dam.

The following two sections of this chapter discuss the Panel's investigation of cavities discovered near the right end of the dam and of sedimentary deposits that underlie the volcanic rock foundation. Three holes, designated DH-650, DH-651AB, and DH-652, drilled primarily for these two studies, also provided an opportunity to test a section of grout curtain lying between Stas. 3+00 and 4+50 (Fig. 5-5). The results of water pressure tests conducted at these holes within this interval are given in Table 3-4, and the drilling logs are contained in Appendix F.

In DH-650 several large water losses were recorded during the pressure testing. However, a survey of the alignment of DH-650 by the USBR under observation of Panel staff indicated that the hole had been deflected from S19°E to S26°E. The water pressure tests therefore were performed in segments of the hole lying upstream of the grout curtain.

At DH-651 no significant pressure losses were measured. For DH-652 a loss of 12.8 gpm occurred in the 301.3-307.7 ft interval, indicating the probable existence of an ungrouted joint. All other losses measured at DH-652 were minor.

The water pressure tests near the right end of the dam did not disclose excessive losses. The reader is referred to Chapter 5 for comments concerning rock permeability beyond the right end of the dam.

**TABLE 3-3
GROUT CURTAIN WATER LOSS TESTING
AT SPILLWAY WEIR**

Location			Pressure Tests						Gravity Tests
Sta	Hole DH-	Angle	Interval ft	Water Loss gpm	Interval ft	Water Loss gpm	Interval ft	Water Loss gpm	Water Loss gpm
10+82	601	30°R	4.0-40.0	6.9	39.6-74.6	0.2	74.2-109.2	0.3	2.6
	602	0°	4.7-34.7	2.0	34.7-64.7	2.1	64.7-94.7	0.4	4.5
	603	30°L	4.5-39.5	1.0	40.1-75.1	0.1	73.7-108.7	0.4	2.3
11+06	604	30°R	5.5-40.5	0.8	40.1-75.1	0.5	74.7-109.7	0.2	0.5
	605	0°	6.0-36.0	0.2	36.0-66.0	0.2	66.0-96.0	0.03	0
	606	30°L	5.6-40.6	1.1	—	—	—	—	—
11+30	606A	30°L	6.8-41.8	0.4	41.4-76.4	0.6	76.0-111.0	0.2	0.4
	607A	30°R	5.3-40.3	1.3	39.9-74.9	0.2	74.5-109.5	0.04	—
	608	0°	5.8-35.8	1.3	35.8-65.8	0.3	65.8-95.8	0.07	0.6
			95.8-125.8	0.3					
609	30°L	5.7-40.7	0.8	40.3-75.3	1.9	74.9-109.0	2.3	3.1	
			110.0-145.0	0.3					

Interval is measured along axis of hole from the collar.

All pressure tests conducted at 10 psi at collar elevation.

All gravity tests conducted full length of hole.

Geologic logs are contained in Appendix F.

**TABLE 3-4
TETON DAM
OCTOBER 1976
DRILL HOLE WATER TESTS
NEAR RIGHT END OF DAM**

DH-650

Location: On dam at Sta. 3+00 approximately 4.7 ft upstream of centerline
 Bearing: S19E (A survey conducted after completion indicated this hole had been deflected to S26°E)
 Dip: 60° below horizontal
 Elevation: 5332 Total Depth: 351.5 ft

Depth Interval*	GPM	
90.0- 97.7	32.1	Hole cased to 90 ft. Lost drilling water return at 91.6 ft – never recovered.
99.8-104.8	32.9	
103.1-127.4	28.3	
127.6-162.6	13.4	
160.6-197.2	8.4	
197.6-232.6	0.0	
232.6-267.5	20.3	
267.5-302.5	6.1	
301.7-331.7	2.1	
331.5-351.5	0.0	

*Footages measured along axes of holes from collar.

All pressure tests conducted at 10 psi measured at hole collar.

Geologic logs are contained in Appendix F.

TABLE 3-4 (cont.)

DH-651

Location: On dam axis at Sta. 4+34
 Dip: Vertical
 Elevation: 5332 Total Depth: 622.4 ft

Depth Interval*	GPM	
80.0-100.0	0.2	
100.0-120.0	0.2	
120.0-140.0	1.0	
140.0-160.0	0.4	
160.0-180.0	0.0	Note: Lost 75% drilling water at 47.2 ft in Zone 1 fill.
180.0-200.0	0.0	
200.0-220.0	0.5	
219.9-239.9	0.4	
239.9-259.9	0.5	Concrete/Zone 1 contact at 78.8 ft.
259.9-279.9	0.4	
279.9-299.9	0.6	
299.9-319.9	1.3	
319.9-359.9	0.6	
359.9-399.9	1.3	
399.9-439.9	0.2	
432.4-472.4	0.0	
479.3-519.3	0.4	Packer in grouted zone.
517.7-527.2	2.2	
518.2-535.9	1.2	
530.8-543.7	6.2	
546.1-552.3	9.3	Packer in casing at 499.7.
546.1-566.1	2.1	Packer in casing at 539.4.
559.3-581.2	2.7	
560.0-600.0	35.0	Hole drilled to 600 ft, but bottom "washed in" to 580 ft. Could not get 10 psi at 35 gpm.

*Footages measured along axes of holes from collar.

All pressure tests conducted at 10 psi measured at hole collar.

Geologic logs are contained in Appendix F.

TABLE 3-4 (Cont.)

DH-652

Location: On dam at Sta. 5+10, 5.5 ft upstream of centerline.

Bearing: N18W

Dip: 60° below horizontal

Elevation: 5332 Total Depth: 450 ft

Depth Interval*	GPM	
95.0-130.0	0.4	Hole collar is 0.4 ft above dam crest.
130.0-165.0	0.4	
165.0-200.0	1.4	
200.0-235.0	0.1	Note: Hole cased to 90 ft, then driven to 94 ft
235.0-270.0	0.8	
267.9-302.9	1.2	
301.3-307.7	12.8	Lost drilling water at 303 ft.
307.7-347.7	0.8	
347.7-387.7	0.3	
387.4-427.4	0.1	
425.0-450.0	0.5	

*Footages measured along axes of holes from collar.

All pressure tests conducted to 10 psi measured at hole collar.

Geologic logs are contained in Appendix F.

Investigation of Cavities Near Right End of Dam.

Extensive fissures were exposed in the foundation of the dam during excavation for the key trench near Stas. 3+55 and 4+34. These are described in Chapter 5. Treatment entailed drilling 8-in.-diam holes into the cavities from ground surface through which high-slump concrete was poured, Figs. 5-15 and 5-16, Sections D-D¹ and E-E¹.

Under the guidance of the Panel staff three holes, designated DH-650, DH-651 AB, and DH-652, were drilled to explore for possible additional cavities undetected by the original investigation and to check the effectiveness of the grouting that had been undertaken. These holes were located near the right end of the dam, respectively at Stas. 3+00, 4+34, and 5+11.2 (Fig. 5-5). Holes DH-650 and DH-652 were inclined 60 degrees below horizontal and oriented to parallel the axis of the dam and to intersect the cavities well beneath the key trench invert. DH-651 was located directly above the largest cavity and was drilled vertically through the concrete filling.

Core samples obtained from the concrete-rock interface in the cavities disclose a tight bond, indicating an effective watertight seal at the contact points penetrated by the drills. This exploration revealed that the large cavities at those points drilled and tested were effectively sealed.

Investigation of Sedimentary Deposits.

Sedimentary deposits of unknown thickness underlie the volcanic rocks on which the dam rests. The possibility of seepage from the reservoir passing beneath the dam through permeable lenses within these deposits is considered in Chapter 5. The deep sediments are generally much less permeable than the overlying jointed volcanic foundation and consequently are judged a less likely avenue of significant leakage.

The deep vertical hole, designated DH-651, at Sta. 4+34 has been described in connection with the exploration of deep cavities near the right end of the dam. An additional purpose of this hole was to explore and sample the underlying lake and stream deposits. The drilling of DH-651 was terminated in the lake and stream deposits at a depth of 622 ft due to frequent blocking of the Nx-diameter core barrel by small rounded pebbles. The drill rig was moved 10 ft to Sta. 4+24 and a new hole, designated DH-651A, was started. Again difficulties necessitated a restart and the rig was shifted to drill DH-651B at Sta. 4+19. To improve core recovery in the deep sediments, the hole was enlarged to permit use of a 5-in. core barrel. Notwithstanding the use of the larger barrel, only a limited length of core could be obtained. Because of persistent blocking, primary emphasis was shifted from obtaining core to determining whether the sediments were comparatively thin beneath the dam or if a thick and lenticular section existed. The remainder of the hole to its final depth of 885 ft was drilled largely with rock bit cores being taken only when finer grained sediments were encountered.

An Nx-size core sample of silt with a liquid limit of 33 and a plasticity index of 8 was obtained from DH-651 at a depth of 595.3 to 596.2, beneath the water table which was at a depth of 312 ft. Three specimens were prepared from this sample and were tested for one-dimensional consolidation by Geo-Testing, Inc., San Rafael, California. The results are presented in the form of pressure-void ratio curves. The curves, when interpreted according to the customary procedures, indicate preconsolidation loads well below the existing overburden pressure. This results either from a high degree of disturbance associated with sampling at such great depths, from an imperfect fit of the stiff samples within the consolidation ring on account of difficulty of trimming the specimens, or both. In any event, the disturbance has so increased the compressibility of the samples that the results are not considered representative of the compressibility of the in-situ material.

INSPECTION OF AUXILIARY OUTLET WORKS

The auxiliary outlet works stoplogs were set at the intake on October 2, 1976 and the tunnel was drained and inspected by the Panel's on-site representative, accompanied by project personnel.

The tunnel was again inspected by Panel members, staff, and project personnel during the October Panel meeting. No offsets, open cracks, or other evidence of displacement, or evidence of overstressing were found. Resurveys on pre-failure bench marks at the gate chamber were made without finding any evidence of settlement.

ROCK JOINT SURVEY

Under the direction of the Panel, detailed maps of the joints in the right abutment were prepared to help determine the probable paths by which water from the reservoir reached the leaks that appeared downstream immediately prior to and during the failure, and establishing channels for the transport of Zone 1 material. The maps covered the entire area on the right abutment that had been covered with the dam embankment, with particular emphasis placed on mapping of the key trench. The base for this mapping consisted of plats to a scale of 1 in. = 5 ft covering the area extending 10 ft both upstream and downstream of the grout cap and from the spillway to the river channel. Topography within the trench was defined by contours drawn at 5-ft vertical intervals. Joints 10 ft or longer were numbered and mapped on the plats and their attitudes shown by conventional dip and strike symbols. Significant observations were recorded, and all notes were cross-referenced to the maps by joint numbers. The key trench joint maps were supplemented with two geologic cross sections drawn parallel to the axis of the key trench respectively 10 ft upstream and 10 ft downstream of the grout cap centerline. The cross sections were needed to define the numerous comparatively flat-lying joints which could not be shown effectively on the areal maps. The joint map and related geologic sections covering the key trench appear in Appendix E.

Major joints in the right abutment lying outside of the key trench were mapped on aerial photo overlays. A total of twelve 24-in. by 24-in. photos was required to cover the abutment to a scale of 1 in. to 20 ft. Two additional geologic cross sections were prepared in connection with this phase of the mapping program. Both these sections were oriented parallel to the centerline of the dam, one 150 ft upstream and the other 100 ft downstream of the centerline. These and other data from the joint survey are included in Appendix E.

COMPARISON OF PRE-FAILURE AND POST-FAILURE SURVEYS

At the Panel's request, post-failure resurvey was made of networks and bench marks established at the damsite and in its environs prior to the failure. The results are shown in Tables 5-5 and 5-7 and Fig. 5-21. No significant horizontal or vertical movement was measured that is pertinent to the cause of failure. Results of surveys of monuments on the dam are discussed in Chapter 11.

MODEL OF THE RIGHT ABUTMENT

The Panel retained ExhibiGraphics Group, a firm in Salt Lake City, to construct a model of the right abutment of Teton Dam to a scale of 1:400. The model has facilitated visualization of principal features of the dam and its foundation that relate to the mechanics of failure. It shows drill holes, observation wells, structures, foundation zones, major rock jointing, points of leakage, and the whirlpool of June 5, 1976. It has removable elements which show pre-failure and post-failure conditions.

CHAPTER 4
SITE SELECTION AND PROJECT SITE INVESTIGATIONS
(Panel Charge No. 3)

EARLY STUDIES

Consideration was given to possible water resources development on the Teton River in eastern Idaho by the Bureau of Reclamation and others as early as 1904. At various times since that time, reconnaissance investigations have been made on damsites on the Teton River and its tributaries. More detailed investigations have been made at several sites beginning in 1932. Nearly all of the sites then studied, however, were on the Upper Teton River or its tributaries. This area is far upstream from the present Teton Dam in a considerably different geologic environment.

None of the early investigations included the Teton (Fremont) site. However, much of the information obtained from these studies is helpful in understanding the geologic conditions at the Teton site.

U.S. Geological Survey.

The U.S. Geological Survey conducted some of the first investigations of the hydrologic and geologic features of the Teton River watershed, and during the 1960's participated with the USBR in inspections of the canyon near the present damsite.

U.S. Corps of Engineers.

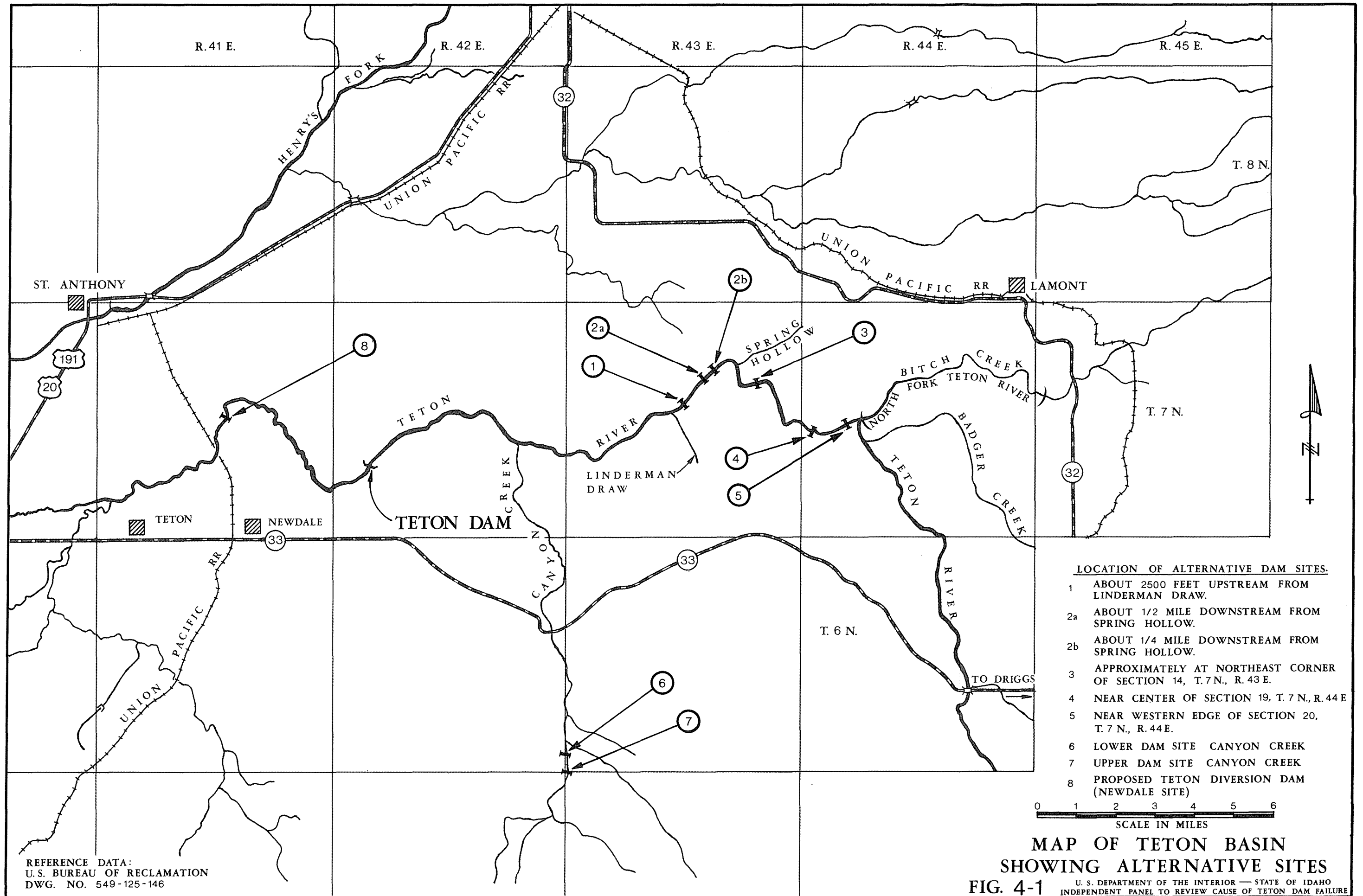
The Teton damsite as such was investigated by the Corps of Engineers in July 1957 by the boring of two diamond drill holes in the vicinity. One boring 146 ft deep was located in the river channel and the other was on the left abutment. The one in the channel showed that the alluvium was about 100 ft deep, while the abutment hole was in rhyolite for its entire depth of 285 ft.

U.S. Bureau of Reclamation.

In 1946 two damsites were investigated by the USBR on Canyon Creek, a tributary of the Lower Teton River, and a report titled "Reconnaissance Geologic Report on Canyon Creek Damsites near Newdale, Idaho," dated March 1947 was prepared by M.H. Logan and C.J. Okeson. The report concluded that storage would be expensive at either site and that seepage losses could be expected from the reservoirs. Locations of alternative damsites are shown in Fig. 4-1.

During August and September 1956, field examinations were made downstream from the Teton site at the Newdale site at the mouth of Teton canyon, three miles north of the town of Newdale. An earthfill diversion dam about 46 ft high was considered at this point where topography of the area was suitable for a diversion canal northward and westward to the North Fork of the Snake River, and ultimately onto the Snake Plain. It was believed that floodflows diverted onto the Snake Plain would sink and add to the groundwater supply to the southwest. The damsite was considered worthy of further consideration and four diamond drill holes were completed approximately along the considered axis. Hole No. 1 was on the left abutment, Nos. 2 and 3 on the valley floor, and No. 4 on the right abutment. All but Hole No. 3 penetrated bedrock. Percolation tests were performed in the overburden and in the bedrock.

The subsequent report by M.J. Athearn, titled "Reconnaissance Geologic Report, Teton River Diversion, Newdale Damsite," dated March 1957, concluded that the Newdale site was infeasible.



- LOCATION OF ALTERNATIVE DAM SITES.**
- 1 ABOUT 2500 FEET UPSTREAM FROM LINDERMAN DRAW.
 - 2a ABOUT 1/2 MILE DOWNSTREAM FROM SPRING HOLLOW.
 - 2b ABOUT 1/4 MILE DOWNSTREAM FROM SPRING HOLLOW.
 - 3 APPROXIMATELY AT NORTHEAST CORNER OF SECTION 14, T. 7 N., R. 43 E.
 - 4 NEAR CENTER OF SECTION 19, T. 7 N., R. 44 E.
 - 5 NEAR WESTERN EDGE OF SECTION 20, T. 7 N., R. 44 E.
 - 6 LOWER DAM SITE CANYON CREEK
 - 7 UPPER DAM SITE CANYON CREEK
 - 8 PROPOSED TETON DIVERSION DAM (NEWDALE SITE)



**MAP OF TETON BASIN
SHOWING ALTERNATIVE SITES**
 FIG. 4-1 U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
 INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

REFERENCE DATA:
 U.S. BUREAU OF RECLAMATION
 DWG. NO. 549-125-146

The Bureau of Reclamation prepared a reconnaissance geologic report on Teton Dam and Pumping Plant site in January 1961 and a special report in March 1962. At that time the dam was planned as an earthfill about 310 ft high above stream level, with a chute spillway on the right abutment. Ample quantities of impervious material were estimated to be available within one mile of the damsite in the tableland on either side of the canyon. Sand, gravel, and volcanic rock for construction purposes were also obtainable nearby.

In October 1961, representatives of the Teton County Wheatgrowers Association proposed that the Bureau of Reclamation consider storage in the vicinity of the mouth of the North Fork Teton River as an alternative to the Teton site. They felt that a diversion from a reservoir in this location could serve a much greater area of new land. (Fig. 4-1.)

At that time the USBR cited storage capacity for flood control, ease of diversion to the Enterprise and East Teton canals, and the more climatically suited lands of the Rexburg Bench as important reasons for placing the storage as far downstream in the Teton Canyon as possible. Upstream from the mouth of the North Fork, the gradient of the river steepens considerably and the USBR pointed out that the suggested reservoir would therefore have less capacity.

The Teton River canyon upstream from the Teton site was believed by Bureau geologists to have been subjected to some faulting as evidenced by displacement in the relatively young basalt flows that cap the canyon rim. Any such faults were difficult or impossible to discern in the rhyolite or welded tuff in the canyon walls. Preferably, any damsite would have to be located some distance from these faults and in an area where there was competent rock on both sides of the canyon.

In November 1961 a Bureau geologist looked at five possible sites in the stretch of the river from Linderman Draw upstream to the mouth of North Fork. From these observations, it was concluded that the best site for a dam in this reach would be about one-half mile upstream from Linderman Draw, with the second choice about one-half mile downstream from Spring Hollow. Damsites farther upstream would have less capacity because of the steep gradient of the river. Linderman Draw is on the south side of the Teton River canyon 9.6 miles upstream from the Teton damsite, and Spring Hollow is on the north side about 12 miles upstream.

Following are excerpts from a report entitled "Teton Basin Project, Lower Teton Division," made by the U.S. Bureau of Reclamation, March, 1962:

. . . Fremont storage is closer to points of use than are the present sources of water distributed by the Enterprise and East Teton Canals. By supplying these uses from the proposed new storage, an appreciable water economy could be effected by savings in canal losses now experienced in the diversions

. . . Fremont Reservoir could be operated on a forecast basis to reduce floodflows to the 2,000-cubic-foot-per-second bankfull capacity for most floods on lower Teton River. This regulation would also effect a large reduction of floodflows in lower Henrys Fork and a significant reduction of flows in Snake River below Henrys Fork. . . .

. . . The channel capacity in the lower reach of the Teton River is about 2,000 cubic feet per second, and general inundation occurs with a discharge of 4,000 cubic feet per second. . . .

SELECTION OF TETON DAMSITE

The reservoir was finally located as far downstream in the Teton River Canyon as elevation, topography, and geology would permit in order to minimize the cost of the conveyance system for water between the reservoir and the project lands. Alternative upstream sites were rejected because of smaller storage capacity and more difficult canal construction. Downstream, the topography was judged to be unfavorable. Engineers and geologists from USBR offices in both Denver and Boise participated in the final site selection.

CORE DRILLING AT TETON DAMSITE AND RESERVOIR AREA

About 100 core drill holes were bored at the damsite in the period 1961-1970. Locations of holes are shown in Figs. 4-2, 4-3, and 4-4.

1961-62.

The USBR started its diamond drilling program at the damsite and reservoir area in July 1961. Four drill holes were completed and two others started in that year. In 1962, drilling started in July and ended in November with the completion of the two holes started the previous year and the boring of six additional holes. Thus, twelve holes were drilled in the 1961-1962 period — ten near the damsite and two in the reservoir area about 9.6 miles upstream from the damsite. The total footage of drill holes was 5,107 lin ft.

1967.

A total of 36 holes was drilled at the damsite in 1967, as follows:

Canyon Bottom	6
River Outlet Works	10
Power and Pumping Plants	4
Right Abutment	4
Left Abutment	7
Spillway	5

1968.

Ten more holes were bored this year, consisting of six for the river outlet works, two on the left abutment, and two for the pump canal discharge line. In addition, three holes were drilled in the basalt riprap source area two miles downstream from the damsite.

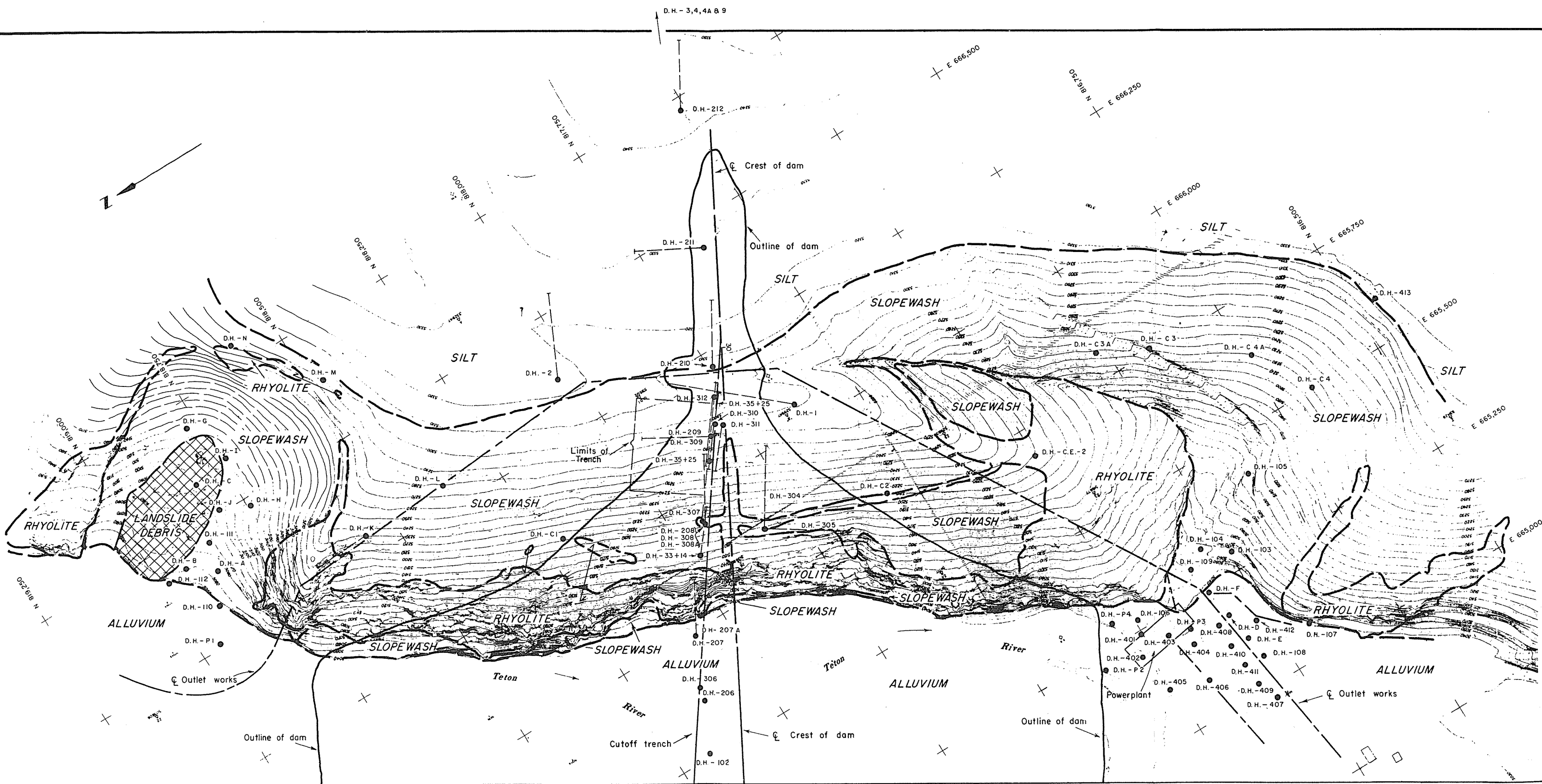
1969.

Fourteen holes were completed at the damsite in 1969, comprising four for the river outlet works, two on the left abutment, one for the auxiliary outlet works, and seven for the spillway. In addition, seven shallow auger holes were bored in the spillway area.

1970.

Thirty holes were drilled at the damsite during 1970, including ten to check the pilot grouting results (nine on the left abutment and one in the canyon bottom). The other 20 were located as follows: ten at the river outlet works, six at the powerplant, three on the right abutment, and one on the pump canal discharge line. The holes drilled in 1970 to verify the results of the pilot grouting program are discussed later in this chapter.

Percolation tests, using single mechanical packers, were made in the drill holes in intervals of from 10 to 60 ft. Most of the sections were tested for 5 minutes each at pressures of 25, 50, or 100 psi.



- Match Line Sheet 2 of 2
- EXPLANATION**
- LANDSLIDE DEBRIS - Unsorted mixture of gravel to block-size pieces of rhyolite in a silty soil.
 - SILT - Loosely to moderately compacted windblown silt on the uplands; locally covers the slopewash on the valley sides.
 - SLOPEWASH - lightly to moderately compacted, gravel- to boulder-size, angular and platy fragments of rhyolite in a silty soil matrix.
 - TALUS - Accumulation of loose, platy and angular fragments of rhyolite up to 24" across.
 - ALLUVIUM - Stream-deposited silt, sand, gravel, and some cobbles.
 - DH-101 DIAMOND DRILL HOLE - vertical
 - DH-201 DIAMOND DRILL HOLE - angle
 - AP-S1 POWER AUGER HOLE
 - APPROXIMATE BOUNDARY BETWEEN GEOLOGIC UNITS

Match Line Sheet 2 of 2

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

TETON BASIN PROJECT-IDAHO, WYOMING
LOWER TETON DIVISION-IDAHO

TETON DAMSITE
AREAL GEOLOGY

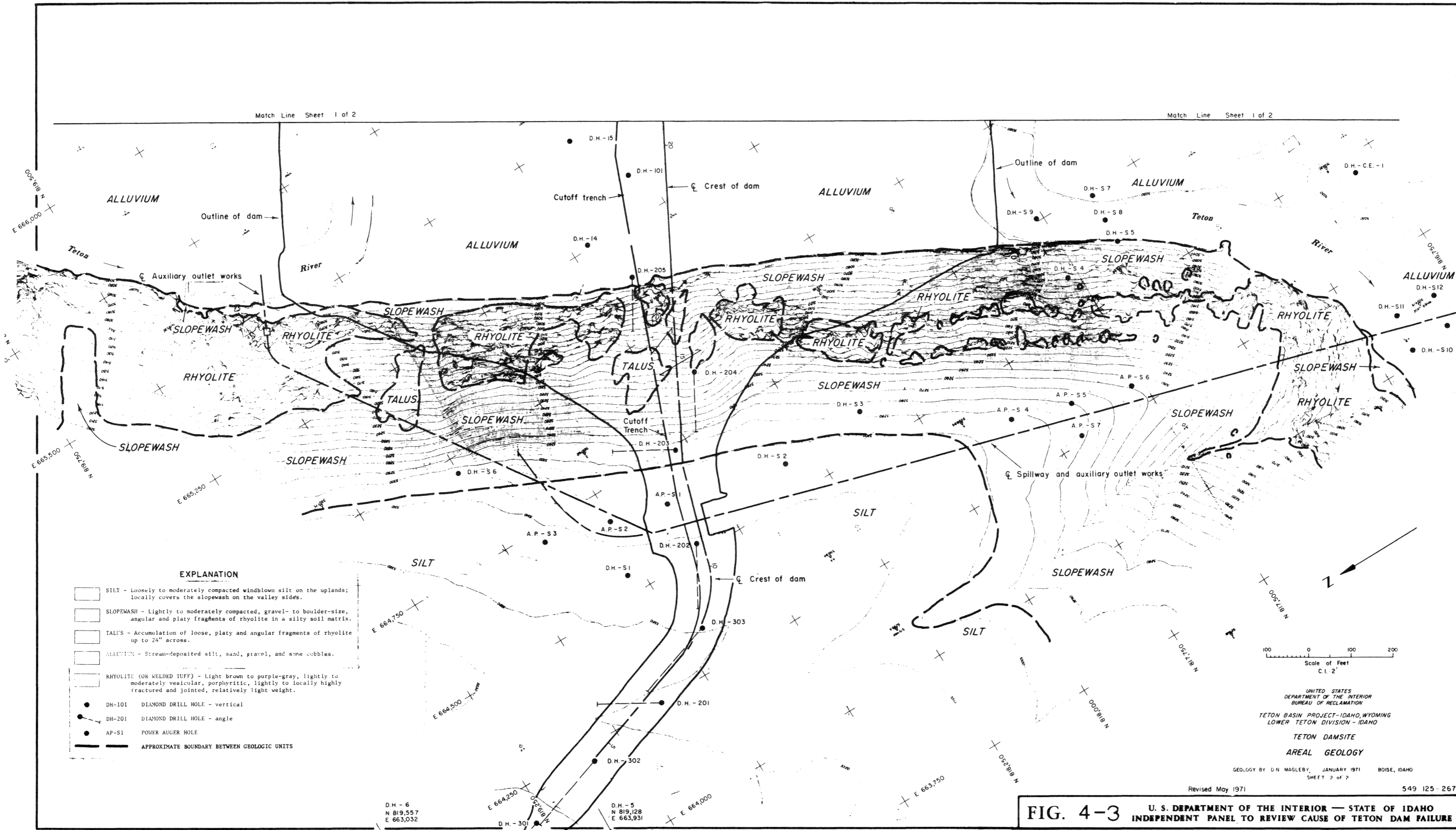
Scale of Feet
C.I. 2'

GEOLOGY BY D.N. MAGLEBY JANUARY 1971 BOISE, IDAHO
SHEET 1 of 2

Revised May 1971

549-125-266

FIG. 4-2. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



EXPLANATION

- SILT - Loosely to moderately compacted windblown silt on the uplands; locally covers the slopewash on the valley sides.
- SLOPEWASH - Lightly to moderately compacted, gravel- to boulder-size, angular and platy fragments of rhyolite in a silty soil matrix.
- TALUS - Accumulation of loose, platy and angular fragments of rhyolite up to 24" across.
- ALLUVIUM - Stream-deposited silt, sand, gravel, and some cobbles.
- RHYOLITE (OR WELDED TUFF) - Light brown to purple-gray, lightly to moderately vesicular, porphyritic, lightly to locally highly fractured and jointed, relatively light weight.
- DH-101 DIAMOND DRILL HOLE - vertical
- DH-201 DIAMOND DRILL HOLE - angle
- AP-S1 POWER AUGER HOLE
- APPROXIMATE BOUNDARY BETWEEN GEOLOGIC UNITS

Scale of Feet
1" = 200'

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TETON BASIN PROJECT-IDAHO, WYOMING
LOWER TETON DIVISION-IDAHO

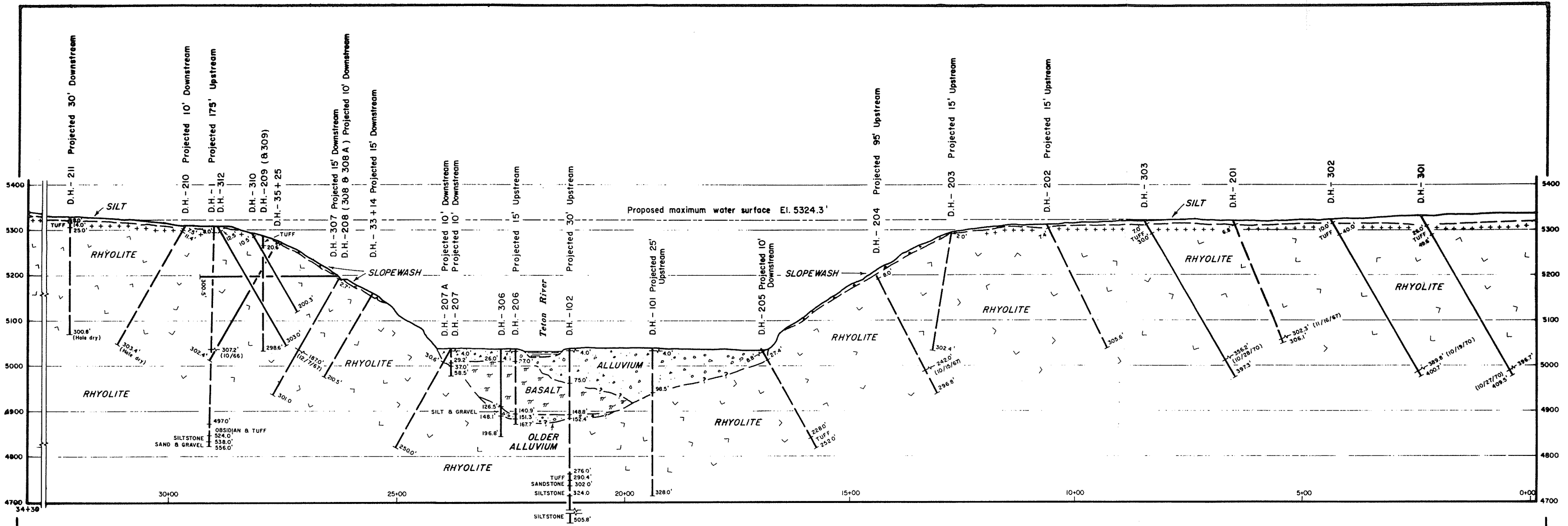
TETON DAMSITE
AREAL GEOLOGY

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SHEET 2 of 2

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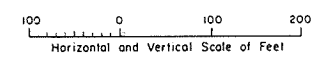
549 125-267

**FIG. 4-3 U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE**



EXPLANATION

- SILT - Windblown silt on the uplands; locally covers the slopewash on the valley sides.
- SLOPEWASH - Lightly to moderately compacted, gravel- to boulder-size, angular and platy fragments of rhyolite in a silty soil matrix.
- ALLUVIUM - Stream-deposited silt, sand, gravel, and some cobbles.
- BASALT - Intracanyon basalt flow, dark purple to black, dense to slightly vesicular, fairly fresh, hard, highly fractured.
- OLDER ALLUVIUM - Stream-deposited silt, sand, and gravel that is present in Teton River canyon directly beneath the intracanyon basalt.
- RHYOLITE (OR WELDED TUFF) - Light brown to purple-gray, dense to moderately vesicular, porphyritic, lightly to locally highly fractured and jointed, relatively light weight. The upper few feet of the flow is tuffaceous (++++).
- OLDER SEDIMENTS - Siltstone, claystone, sand and gravel and other sediments that underlie the rhyolite flow.
- D.H. - 207
DIAMOND DRILL HOLE - (dashed where projected), water level. ---
58.8'
- BOUNDARY BETWEEN GEOLOGIC UNITS - Dashed where approximate; queried where in.



UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TETON BASIN PROJECT - IDAHO, WYOMING
LOWER TETON DIVISION - IDAHO
TETON DAMSITE
GEOLOGIC CROSS SECTION ALONG CUTOFF TRENCH
GEOLOGY BY D. N. MAGLEBY JANUARY 1971 BOISE, IDAHO

Revised May 1971

549-125-274

FIG. 4-4. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Percolation tests showed many sections of rhyolite to be relatively impermeable, but individual cracks or joints were capable of transmitting large quantities of water — over 100 gpm.

Extended pump-in tests were made in five of the drill holes on the right abutment. These were drilled on a dip of 60° (30° from vertical). (Fig. 4-3). (The following depth figures refer to vertical measurements.) In DH-301, water pumped into the hole at a rate of 290 gpm raised the water level in the hole 159 ft, and water pumped in at a rate of 460 gpm raised the water level 171 ft. In DH-302, a pump-in test for 19 hours at a rate of 450 gpm raised the water level 57 ft. In DH-303, water pumped into the hole for two weeks at a rate of 450 gpm raised the water level 185 ft. In DH-202, pumping at a rate of 165 gpm maintained the water level at the collar of the hole. In DH-203, water pumped in at a rate of 400 gpm raised the water level 200 ft (from the bottom of the hole, which was dry). During these extended pump-in tests, no water was found to leak from the abutment, although there were rises in the water levels in some of the surrounding holes on the right abutment. Chapter 5 contains more information on water testing of drill holes.

The Bureau's bore-hole television camera was used to examine some of the drill holes on the right abutment and some of the grout-check holes on the left abutment. The camera observations showed many cracks and joints of apparent random orientation. The widest crack measured was 1.7 in.; most cracks were 0.1 to 0.5 in. wide.

OTHER EXPLORATION

In the period 1961-70, in addition to the core drilling programs, geological mapping of the joints appearing in outcrops in the canyon walls was carried out. In 1967 a magnetometer survey was made along the line of the proposed cutoff trench, to determine if any large cavities were present in the left abutment. The results of this survey were reported to be inconclusive.

RESERVOIR LEAKAGE STUDIES

The estimation of reservoir leakage was a major consideration in all the investigations leading to the final design. The bulk of the core drilling and permeability testing for this purpose was done in direct coordination with the dam foundation investigations.

The Teton River normally loses water to the surrounding ground in the reach of the canyon where the dam is located. Although the water table has a regional gradient toward the southwest, locally in the vicinity of the damsite it slopes 5-1/2 ft per mile to the northwest. In addition to the regional water table, there is a well defined perched water table which, prior to reservoir filling, was 100 ft or more above the regional water table.

In a report titled "Ground-water Aspects of the Lower Henrys Fork Region, Idaho," 1967, by E.G. Crosthwaite, M.J. Mundorff, and E.H. Walker of the U.S. Geological Survey, estimated seepage losses from the proposed Teton Reservoir were 49 cfs (rounded to 50 cfs, or 36,000 acre-ft per year). This figure was offered as only "an order of magnitude," and was compared with an estimate of 8 to 42 cfs made by Okeson and Magleby of the Bureau of Reclamation in 1963.

The Independent Panel regards reservoir loss rates as primarily of economic importance and not related directly to the safety of the dam.

ROCK CORE TESTING

In 1970 the USBR laboratories in Denver made various tests of Nx and Bx core specimens of foundation rock selected from the Teton damsite. These tests were conducted to determine the physical and mechanical properties. The cores were taken from holes DH-1, DH-D, DH-L, DH-S6, DH-S10, and DH-108. The test results are summarized in Table 4-1.

The properties of each of the rock types tested – basalt and rhyolite – were found to be fairly uniform in their relationships, with the basalt having high elasticity and strength (averaging about 9.4 million psi and 13,600 psi, respectively) and the rhyolite having lower corresponding values (averaging 1.6 million psi and 5,960 psi, respectively).

Tests of Nx rock cores from holes DH-402 and DH-403 at the site of the Teton Powerplant were made in 1970. They showed comparable average properties of basalt and rhyolite, namely 7.1 million psi and 13,000 psi versus 2.3 million psi and 6,860 psi, respectively.

The core specimens from these drill holes were subjected to petrographic examination at the Denver laboratory.

PILOT GROUTING PROGRAM

To assist in appraising the feasibility of the Teton damsite, the Bureau of Reclamation conducted a pilot grouting program on the left abutment of the Teton damsite in 1969. This program consisted of grouting and pressure testing 23 holes, including previously drilled exploratory holes as well as new curtain and blanket grouting holes. Curtain holes generally are part of an in-line series of relatively deep borings that are grouted with the objective of influencing seepage patterns. Blanket holes are usually shallower borings arranged in an areal pattern and grouted with the intent of strengthening the foundation.

There were significant grout takes in several holes. The grout injection in two exploratory drill holes alone exceeded the originally estimated take for pressure grouting in the entire program. Final quantities injected into these two holes were 15,720 sacks of cement and 17,787 cu ft of sand. In the blanket grouting effort the largest grout take in any hole was 1,626 sacks of cement.

In addition to other results, the basalt interflow was shown to be very hard but intensely jointed. The gravel layer between the basalt and the rhyolite accepted grout.

The curtain holes showed exceptionally high takes at depths less than 70 ft and considerable grout travel, up to 300 ft downstream. Subsequently, after thickening the grout using cement-sand mixes and calcium chloride, the leaks tended to seal. Due to a persistent surface leak located 300 ft downstream of Sta. 33+00, stages from 30 to 70 ft could not be completed to refusal. Grouting of curtain holes at depths less than 30 ft was abandoned, since the key trench was expected to be at least that deep.

In 1970 ten holes were drilled in the area of the pilot grouting to check the effectiveness of that grouting. Most of the water tests took very little water. Geologic logs were prepared showing the detailed water loss information, description of joints, and occurrences of grout found in the core. The drill cores contained numerous seams of grout ranging in width from about 1/50 of an in. to 4 in. The grout was generally well bonded to the rock. The USBR bore hole television camera was used in some of these drill holes to observe and measure the thickness and attitude of grouted cracks. Much of

TABLE 4-1
SUMMARY OF FOUNDATION ROCK PROPERTIES

Specimen No. (DH-Depth)	Rock type	Teton Damsite – Teton Basin Project		Compressive strength, psi	Absorption % by wt	Specific gravity
		Elasi- ticity* 10 ⁶ psi	Poisson's ratio			
S10-51.7	Basalt	—	—	14,500	0.28	2.83
S10-55.3	Basalt	9.4	—	15,800	0.20	2.83
S10-55.5	Basalt	—	—	10,400	0.27	2.86
108-50.7	Basalt	—	—	—	0.30	2.80
Avg.		9.4		13,600	0.26	2.83
1-318.1	Rhyolite	—	—	6,990	4.48	2.28
1-319.0	Rhyolite	1.7	0.10	7,260	3.87	2.34
1-320.9	Rhyolite	2.1	0.13	6,400	3.92	2.30
1-321.6	Rhyolite	1.4	0.12	6,050	4.07	2.29
D-29.7	Rhyolite	1.5	0.12	5,040	3.82	2.38
D-31.9	Rhyolite	1.5	0.13	3,640	4.77	2.36
D-32.3	Rhyolite	1.7	0.17	3,780	4.42	2.37
D-34.1	Rhyolite	1.4	0.11	4,160	4.33	2.37
L-249.0	Rhyolite	1.1	0.13	6,000	5.04	2.24
L-249.9	Rhyolite	1.6	0.15	7,400	4.84	2.25
L-251.7	Rhyolite	1.6	0.17	6,820	4.34	2.27
L-252.6	Rhyolite	1.7	0.16	7,230	4.32	2.27
S6-262.8	Rhyolite	**0.5	**0.10	6,880	3.42	2.40
S6-265.3	Rhyolite	1.9	0.25	6,460	3.48	2.39
S6-265.7	Rhyolite	**0.7	**0.20	5,600	3.83	2.38
S6-266.6	Rhyolite	1.2	0.13	6,060	3.61	2.39
Avg.		1.6	0.15	5,960	4.16	2.33

*Secant modulus of elasticity (E_s) at 1000 psi stress, first cycle.
 ** E_s and μ at 500 and 700 psi stress for S6-262.8 and S6-265.7, respectively; these omitted from averages.

Note: Specimens with no E_s values were unsuitable for the test;
 DH-108 specimens broke in preparation.

the grout in the upper 50 to 70 ft of the holes was found in cracks and openings about parallel to the nearly horizontal flow planes in the rhyolite.

From review of the drill logs and the pilot grouting data, it was concluded by the designers that it would be more economical to remove the upper 70 ft of the foundation than to conduct the grouting necessary to seal this horizon. Accordingly, a foundation key trench about 70 ft deep was provided above El. 5100 in both abutments seeking to intercept the more open-jointed rock and to reach a groutable horizon in the more sound rock.

COMMENTS

The final location of the Teton Dam was largely based upon factors not directly related to the foundation conditions at the site nor the type of materials available for the construction of the dam. The location was selected primarily because of the increased reservoir volume, as compared with upstream sites, and the lower costs of constructing the conveyance system from the reservoir to the project lands.

The investigations of the geology in the region of the damsite and the foundation conditions at the site were sufficiently detailed to indicate to the designers that the selected site was as favorable for the construction of a dam as any of the other sites studied.

The foundation exploratory drilling, geologic mapping, pumping tests, groundwater observations, and pilot grouting tests which had been completed prior to the adoption of the final design for Teton Dam were sufficiently detailed to provide the designers with adequate knowledge of the site conditions. The jointed character of the foundation rock, with the large water-carrying capacity of the joint system, was well documented from the results of the core borings, water testing of drill holes, groundwater table studies and the pilot grouting tests. The presence of the basalt flow in the canyon at the base of the left abutment was also well defined. Therefore, it can be concluded that the preliminary investigations had disclosed the major characteristics of the foundation and abutments needed to develop a satisfactory design.

CHAPTER 5
GEOLOGY
(Panel Charge No. 1)

REGIONAL GEOLOGY

Teton Dam is located in a steep-walled canyon incised by Teton River into the Rexburg Bench, a volcanic plateau draining into the Snake River Plain. The exposed rocks are almost entirely of volcanic origin (Fig. 5-1), but these are covered on the high lands flanking the canyon by a layer of aeolian sediments up to 50 ft thick.

The volcanic rocks consist of quaternary basaltic cones and flows underlain and interfingering by rhyolite. Rhyolite accumulations include welded ash-flow sheets, lava flows, airfall and waterlaid tuffs, and tuffaceous sediments.

Deep water wells have encountered lenses of sediments of late-Tertiary age enclosed within the volcanic units (Haskett, Gordon I., 1972). They are known locally as "lakebed" or "lake and stream" sediments. Lenses range from a few thousand square feet to several square miles in areal extent and from a few feet to over 900 feet in thickness. These deposits are believed to have accumulated within intermittent lakes created where volcanic flows dammed ancient stream courses. They were buried to their present depths by subsequent volcanic outpourings.

The relationships of the geologic units are shown on Geologic Sections, Figs. 5-2 and 5-3.

Regional Tectonic Activity.

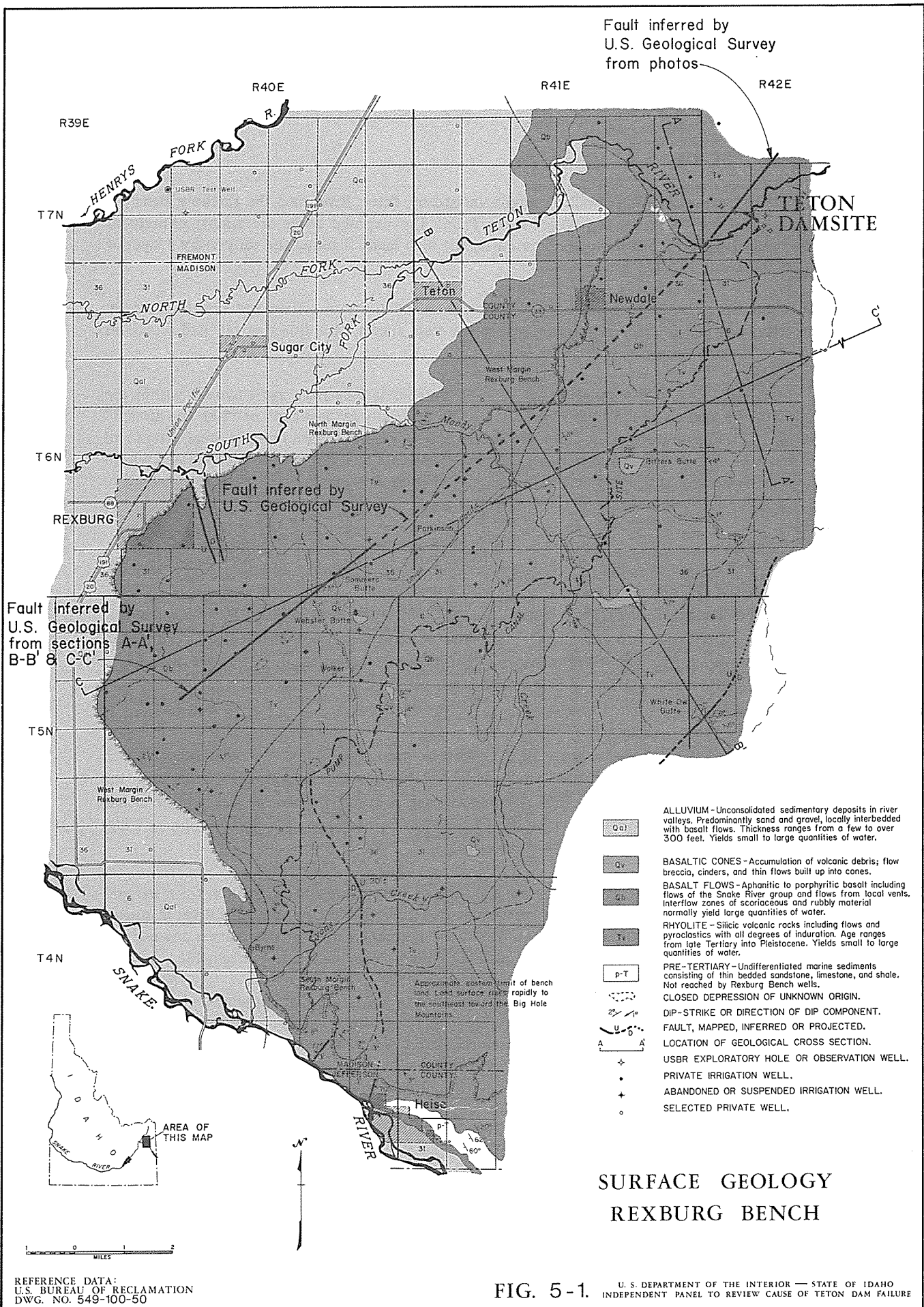
The region surrounding Teton Dam is one of volcanic and tectonic instability. Steep escarpments along major fault zones and records of the occurrence of earthquakes within historic time attest to continuing seismic activity. However, as discussed in Chapter 6, the Panel's investigation disclosed no evidence that earthquakes contributed in any way to the failure of the dam.

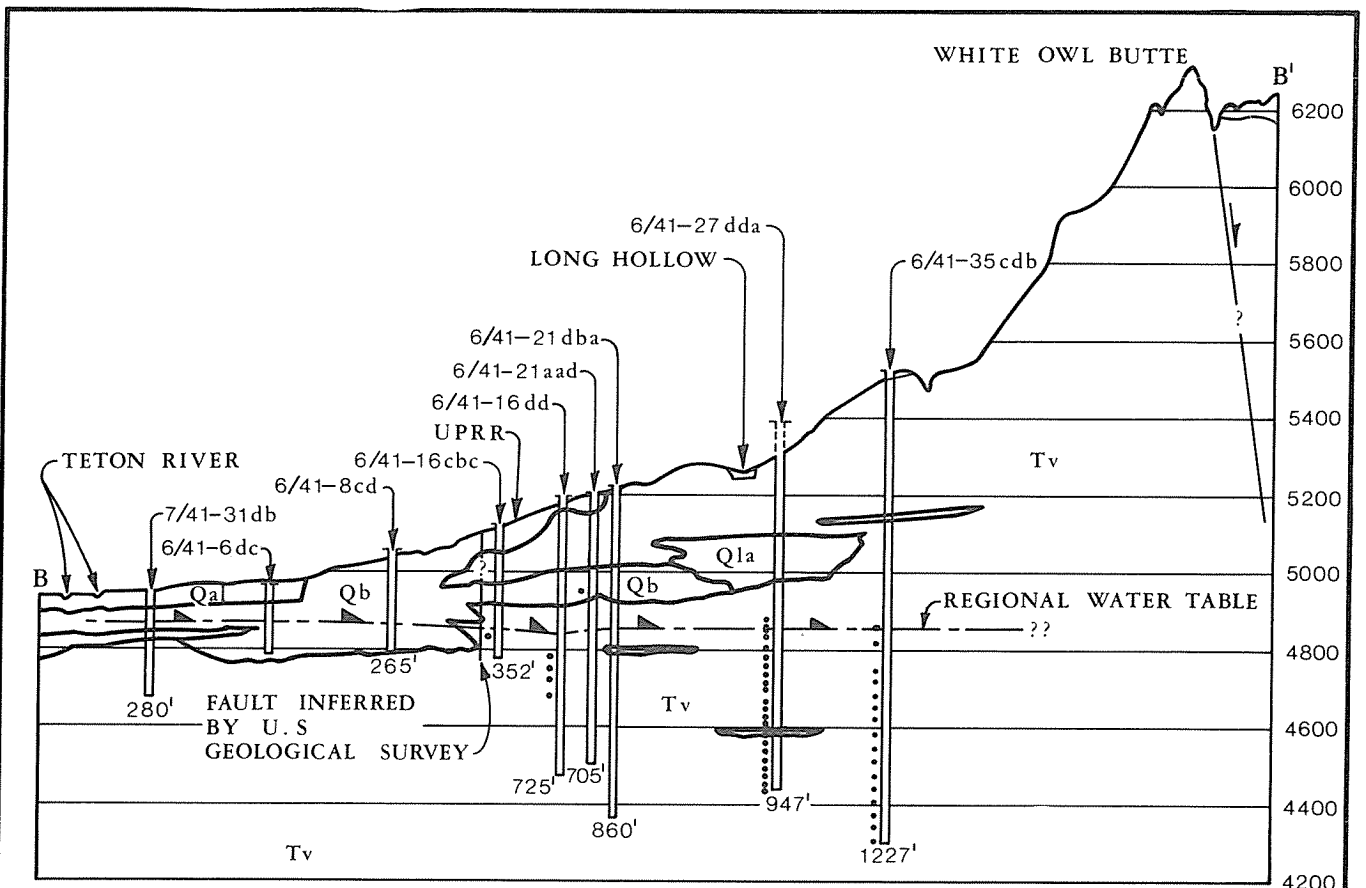
Regional Groundwater Geology.

An extensive network of joints has rendered the otherwise dense volcanic rock of the Rexburg Bench into a highly permeable aquifer. An indication of the magnitude of its permeability is found in the performance of wells which tap it. For example, a well located about three quarters of a mile downstream from the mouth of Teton Canyon, 1½ mile from Teton Dam, reportedly produces 1,800 gpm with a water level drawdown of only two feet.

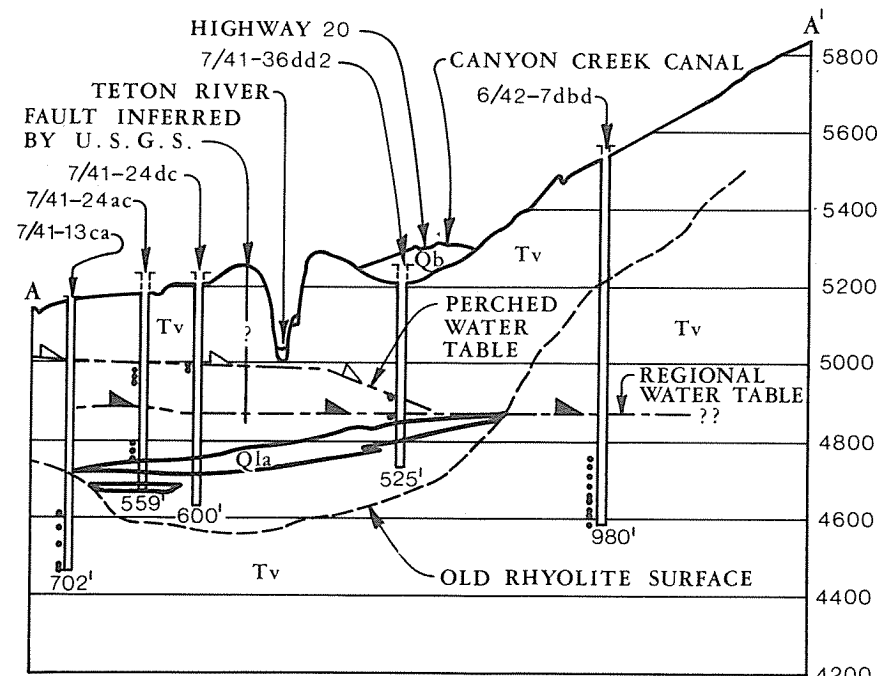
Groundwater replenishment is achieved by precipitation on the surface and percolation from streams. Streamflow measurements by the Bureau of Reclamation along the course of Teton River in August and October 1961 indicated that the stream lost from 25 to 50 cfs through percolation downstream from the site of the dam, but that streamflow losses in the upstream reaches were negligible.

Groundwater also occurs within the buried lake and stream deposits which contain sand and gravel lenses. However, these deposits are generally regarded as poor aquifers in comparison to the jointed volcanic rocks. In some areas they form the base upon which perched water bodies accumulate. Local well drillers are known to terminate drilling when these sediments are encountered because of the lower probability of developing a satisfactory well within them.





SECTION B-B'
CITY OF TETON TO WHITE OWL BUTTE



SECTION A-A'
VICINITY OF TETON DAMSITE

LEGEND

Qa1	ALLUVIUM
Qb	BASALT FLOWS
Tv	RHYOLITE
Q1a	LAKE AND STREAM DEPOSITS
•	WATER BEARING, REPORTED
◄	PERCHED WATER TABLE, REPORTED DURING DRILLING
▲	WATER LEVEL, MEASURED OR REPORTED

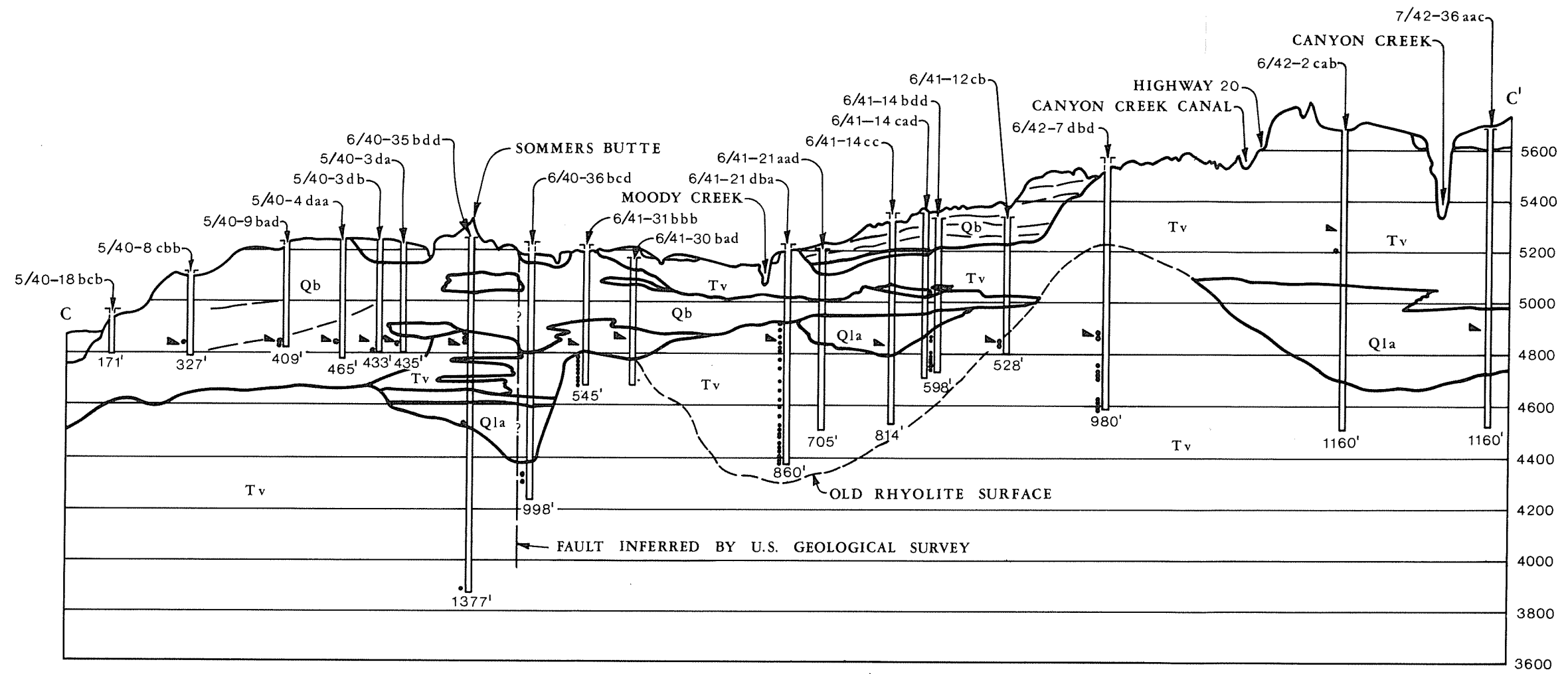
6/41-36cdd-DESIGNATES SE 1/4 SE 1/4 SW 1/4 Sec. 36 T6N R41E

b	a
b a	d
c	d

LOCATIONS SHOWN IN FIG. 5-1.
REFERENCE DATA:
U. S. BUREAU OF RECLAMATION
DWG. NO. 549-100-54

GEOLOGIC CROSS SECTIONS REXBURG BENCH

FIG. 5-2. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



SECTION C-C'

LEGEND

- Qa1 ALLUVIUM
- Qb BASALT FLOWS
- Tv RHYOLITE
- Q1a LAKE AND STREAM DEPOSITS
- WATER BEARING, REPORTED
- ▲ WATER LEVEL, REPORTED OR MEASURED

GEOLOGIC CROSS SECTION
REXBURG BENCH

LOCATIONS SHOWN IN FIG. 5-1.
REFERENCE DATA:
U. S. BUREAU OF RECLAMATION
DWG. NO. 549-100-55

FIG. 5-3 U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

GEOLOGY OF DAM AND RESERVOIR SITE

The walls of Teton Canyon at the damsite consist of late-Tertiary rhyolite welded tuff which has undergone various degrees of welding. It is probably part of the Huckleberry Ridge Member of the Yellowstone Group that was emplaced approximately 2 million years ago as determined from radiometric measurements (Christiansen and Blank, 1972). Alluvium has been deposited in the channel of the canyon to a depth of about 100 ft, and the high lands near the ends of the dam are mantled up to 30 ft with aeolian sediments. The relationship between these formations and those that underlie the ash-flow deposits are shown diagrammatically on Fig. 5-4.

Rhyolite (Welded Ash-Flow Tuff).

Rhyolite is a term used for all silicic lavas in the older literature and reports. More recently the air fall nature of these volcanic rocks has been recognized and thus they may be classified as welded ash-flow tuffs. Welded ash-flow tuff comprises most of the foundation for Teton Dam. Exposures at the site and cores from drill holes display varying degrees of welding. The rock is light weight, has a porphyritic texture with coarse-grained feldspar phenocrysts within a fine-to-medium-grained tuff matrix, and has variable jointing. Upstream from the dam axis, the tuff is divisible into three units. This division is based largely on variations in intensity and character in rock jointing and is described more fully under the subsequent paragraph on that subject.

Physical and mechanical properties of the welded tuff from Teton damsite as determined by laboratory tests on selected drill core specimens are given in Table 4-1.

The contact between the ash-flow tuff and underlying sedimentary deposits as partially established from the logs of foundation drilling appears to be an erosion surface of moderate relief (Fig. 5-5). Possibly, an ancient deep valley existed in this area with its channel about 400 ft below the elevation of the present streambed.

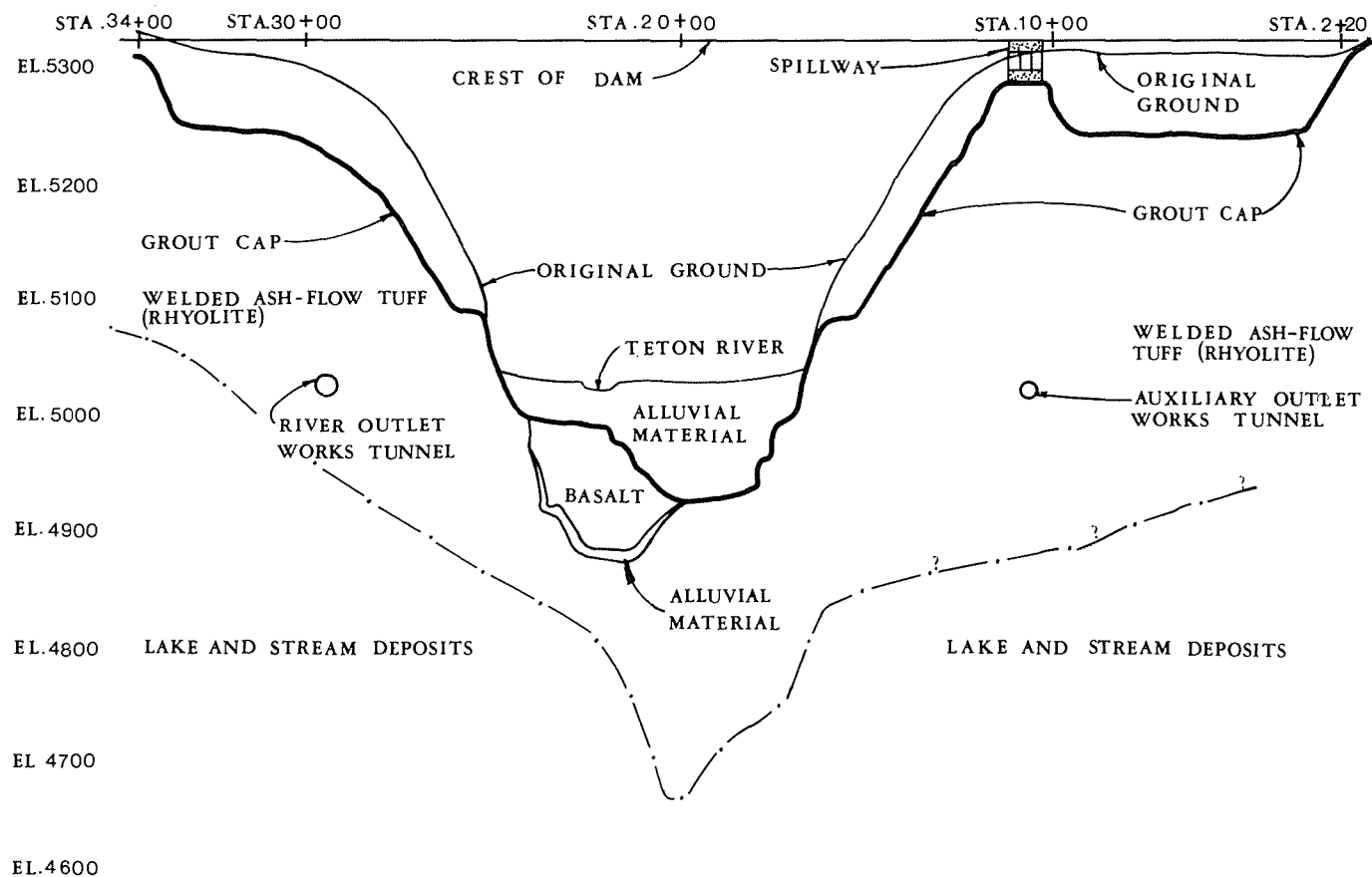
Lake and Stream Sediments.

Lake and stream sediments that interfinger the volcanic rocks consist of a variety of sedimentary types described in the logs of exploratory drill holes as tuffaceous conglomerate, agglomerate, sandstone, tuff, lapilli tuff, ash, tuffaceous sediment, volcanic ash, sand and gravel, boulders and cobbles, and interlayered silt and gravel (Table 5-1).

Water pressure tests were conducted within the lake and stream deposits in a few deep drill holes. Table 5-2 summarizes this information and sets forth all significant water losses that were recorded. The results suggest that some zones within the lake and stream deposits are significantly permeable.

Because of the lenticular structure of the lake and stream sediments, it has not been possible to correlate sand, gravel, or other apparently permeable members from one drill hole to another. However, behavior of groundwater levels in hole DH-506 suggests that Teton Reservoir was connected hydraulically with the lake and stream deposits.

DH-506 is located on the right abutment on the projection of the axis of the dam about 500 ft beyond the end of the embankment (Figs. 5-5 and 5-6). It was drilled to a depth of 644.5 ft, the bottom 53.5 ft penetrating lake and stream deposits. A steel pipe piezometer was installed to a depth of 643.7 ft. A cement seal was emplaced between 576.4 and 566.5 ft to isolate the piezometer from shallower groundwater bodies.

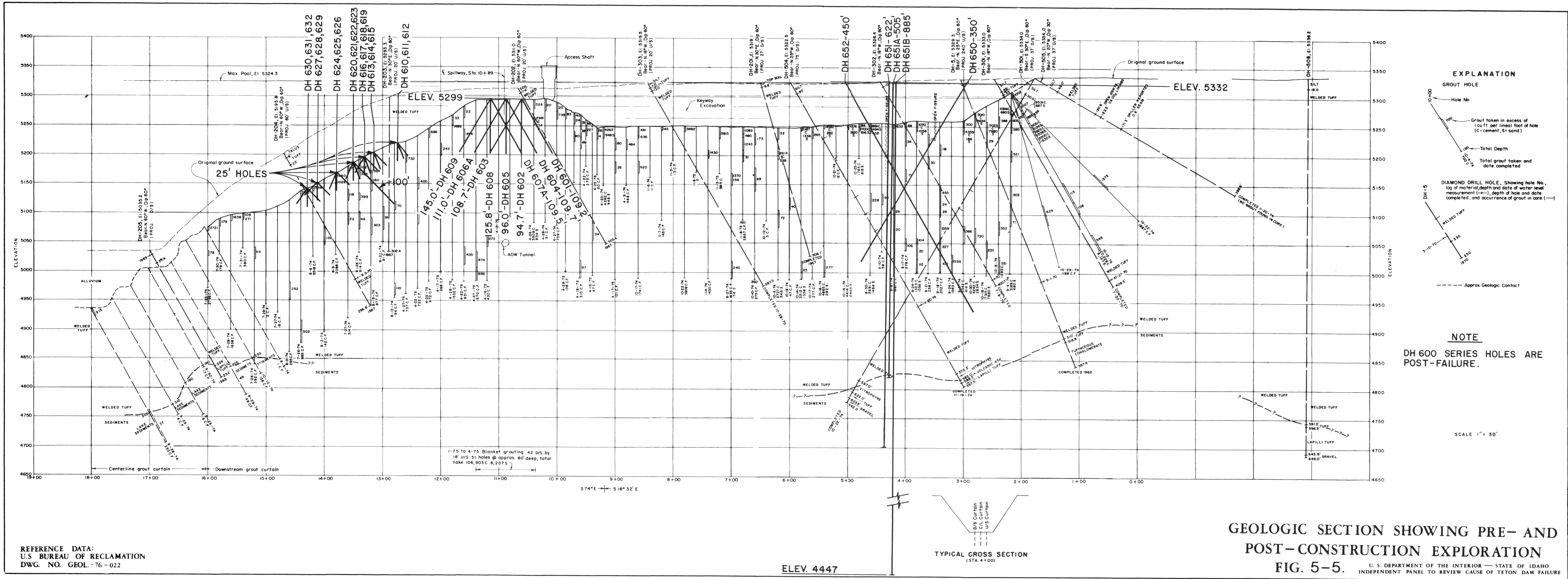


PROFILE OF TETON DAM ALONG GROUT CAP

REFERENCE DATA:
 ABERLE, PETER P., AUGUST 1976

FIG. 5-4.

U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
 INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



EXPLANATION

- GROUT HOLE
- Hole No.
- Grout taken in excess of 1 cu ft. per lineal foot of hole (C = cement, S = sand)
- Total Depth
- Total grout taken and date completed
- DIAMOND DRILL HOLE, Showing hole No., log of material, depth and date of water level measurement (—), depth of hole and date completed, and occurrence of grout in core (—)
- Approx. Geologic Contact

NOTE

DH 600 SERIES HOLES ARE POST-FAILURE.

SCALE 1" = 50'

REFERENCE DATA:
U.S. BUREAU OF RECLAMATION
DWG. NO. GEOL.-76-022

GEOLOGIC SECTION SHOWING PRE- AND POST-CONSTRUCTION EXPLORATION

FIG. 5-5. U.S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

TABLE 5-1
TYPICAL DESCRIPTIONS OF LAKE AND STREAM SEDIMENTS
UNDERLYING THE VICINITY OF TETON DAM
As Abstracted From U.S. Bureau of Reclamation Drill Hole Logs

Drill Hole No.	Vertical Depth Interval (feet)	Core Recovery (%)	Description
1 (Fig. 4-4)	440-443	100	Tuff (soft, friable)
	443-454	100	Tuff (quite hard and dense)
	454-456	22-100	Siltstone (quite well consolidated)
	456-467	75-100	Siltstone or silt (quite soft)
	467-477	9	Boulders and cobbles
	477-482	33	Sand and gravel
5 (Fig. 5-5)	442-448	15	Gray/white tuff (very soft)
	448-491	10-78	Tuffaceous conglomerate
9 (Fig. 5-6)	276-288	3	Tuff (dense, fine grained)
	288-291	?	Tuff (well consolidated)
	291-299	58	Silt (nonplastic, compacted, quite dense)
	299-309	0	Gravel (rounded)
	309-314	0	Not described
	314-324	0	Gravel
	324-334	22-38	Silt (compacted)
	334-335	38	Silt (soft)
	335-347	95	Silt (lightly compacted)
	347-351	0	Gravel
	351-359	22	Silt (lightly compacted)
	359-377	0	Gravel
	377-378	40	Silty sand (75% fine sand, 25% fines)
	378-379	0	Gravel
	379-384	45	Silt and silty clay
102 (Fig. 4-4)	276-290	100	Tuff (or tuffaceous sandstone). Firm, cannot be broken with hands.
	290-302	100	Sandstone (tuffaceous, firm)
	302-326	100	Siltstone (or tuff, fairly hard)
	326-335	50	Lapilli tuff (fairly hard)
	335-413	0	Sand and gravel
	413-421	0	Sandy clay (80% medium plastic fines, 20% fine sand)
	421-506	36	Claystone (fairly hard, quite brittle)
501 (Fig. 5-5)	539-549	0	Lapilli in a medium to well-consolidated fine tuff matrix
	549-556	0	Gravel (particles ½" to 2" across)

TABLE 5-1 (cont.)

Drill Hole No.	Vertical Depth Interval (feet)	Core Recovery (%)	Description
503 (Fig. 5-6)	216-226	30	Ash (crumbles in fingers)
	226-296	96	Tuffaceous sediment
	296-306	20	Sand and gravel
	306-344	0	Interpreted to be gravel with layers of sand and/or silt
504A (Fig. 5-5)	507-510	100	Volcanic ash (fairly well consolidated)
	510-517	99	Lapilli tuff (medium to well consolidated)
506 (Fig. 5-5)	591-645	72-100	Tuff and lapilli tuff (medium to well consolidated, jointed)
507 (Fig. 5-6)	343-351	84	Ash (crumbles with finger pressure)
	351-365	90	Tuffaceous sediment (scratches with hard fingernail pressure)
	365-372	18	Sand and gravel

TABLE 5-2
DESCRIPTIONS AND RESULTS OF WATER PRESSURE TESTS OF PERVIOUS
SEDIMENTARY FORMATIONS WHICH UNDERLIE VOLCANIC ROCK
AT TETON DAM

Drill Hole Number	Vertical Depth to Top of Sediments (Feet)	Elevation to Top of Sediments	Type of Permeable Sediments Logged	Vertical Thick- ness of Permeable Sediments Penetrated (Feet)	Results Water Pressure Tests		
					Zone Tested	Loss GPM	Pressure PSI
DH-1 (Fig. 4-4)	454	4846	Siltstone	13	4848 to 4839	1.6 2	25 50
	467	4834	Boulders & cobbles	11		Not Tested	
	477	4823	Sand & gravel	3.6		Not Tested	
DH-5 (Fig. 5-5)	448	4882	Tuffaceous conglomerate	44	4838 to 4882	31 34.5 37.4	25 50 100
					4772 to 4729	11.3 13.7	25 100
DH-9 (Fig. 5-6)	291	5130	Interlayered silt & gravel	93	5074 to 5117	25 37	25 100
DH-15 (Fig. 5-6)	145	4891	*Tuff breccia & gravel	5		Not Tested	
DH-102 (Fig. 4-4)	149	4886	Gravel	4	*4884 to 4894	8 13.6 19	25 50 100
					4879 to 4893	9 12 20.6	25 50 100

*Sedimentary deposits underlying basalt in left abutment. Not related to the deeper lake and stream deposits.

TABLE 5-2 (Cont.)

Drill Hole Number	Vertical Depth to Top of Sediments (Feet)	Elevation of Top of Sediments	Type of Permeable Sediments Logged	Vertical Thick- ness of Permeable Sediments Penetrated (Feet)	Results Water Pressure Tests		
					Zone Tested	Loss GPM	Pressure PSI
DH-102 (cont.)	290	4745	Sandstone & siltstone	36	4725 to 4745	7	25
						10.6	50
						13.4	100
	335	4700	Sand & gravel	78		Not Tested	
DH-501 (Fig. 5-5)	549	4785	Gravel	7		Not Tested	
DH-503 (Fig. 5-6)	206	5068	Sand & gravel	10		Not Tested	
DH-506 (Fig. 5-5)	591	4745	Tuff & lapilli tuff	53.5		*Not Tested	
DH-507	365	4675	Sand & gravel	7.2		Not Tested	
DH-651 (Fig. 5-5)	560	4772	Tuff, gravel & clay	35	4732 to 4772	35	10

*Hydrograph of piezometer sealed in bottom 20 feet of hole correlates with reservoir stage and water levels in adjacent drill holes.

The hydrograph from the water stage recorder at drill hole 506 shows a fairly close correlation between groundwater levels in the lake and stream deposits, with other observation wells located in the right abutment, and with reservoir stage (Fig. 5-7). Thus, these observations would suggest a fairly extensive permeable zone to exist within the lake and stream deposit in the right abutment area. However, the effectiveness of DH-506 as a piezometer within the lake and stream deposits is subject to question. The log of the hole shows that the cement seal was placed in rock containing joints stained by moving water and that water pressure tests conducted within this interval showed significant losses as follows:

<u>Depth Interval Tested</u> (ft)	<u>Pressure</u> (psi)	<u>Length of Test</u> (min)	<u>Water Loss</u> (gpm)
565-570	50	5	8.9
	100	5	13.6
570-575	50	5	4.9
	100	5	6.4

Thus it is possible that the cement seal is bypassed by open joints in the surrounding rock and therefore that the water levels and hydrographs recorded for this piezometer are not the true levels within the lake and stream deposits.

Basalt.

Basalt was encountered beneath the alluvium near the left wall of the canyon during site exploration and was subsequently exposed when the channel section for the dam was excavated (Fig. 5-4). Basalt was also exposed in the foundation for the power and pumping plant and in outcrops in the spillway stilling basin. Its source is believed to be an unidentified vent near the mouth of Teton Canyon, whence it flowed upstream over the thin veneer of alluvium covering the floor of the gorge to an elevation of about 5005 ft. The river eroded the basalt from the right side of canyon but left it on the left side, where it was subsequently buried with alluvial debris.

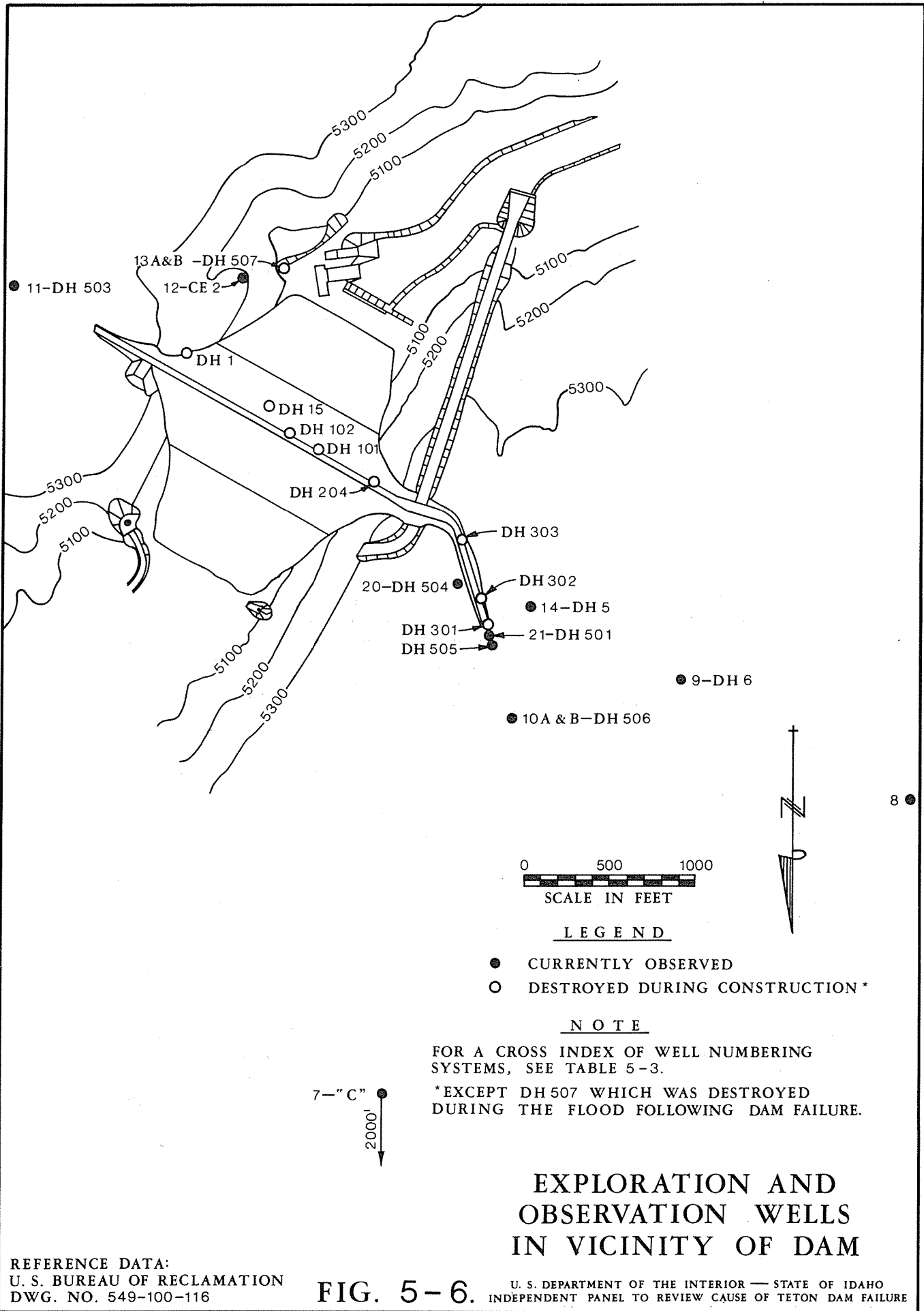
Joints.

Joints are prevalent in the volcanic rocks. They are exposed prominently in the walls of the canyon and are evident in the drill cores obtained during site exploration. In the right abutment, they are largely either steeply dipping or near horizontal in attitude. Flat-lying joints prevail upstream of the dam axis and vertical joints dominate downstream (Fig. 5-8).

Joints in the reservoir walls are part of an extensive interconnecting system that transmits and stores groundwater beneath the Rexburg Bench. Regionally, they render the volcanic rock highly permeable, providing multidirectional flow paths.

The three separate units of welded ash-flow tuff identified in the right wall of the canyon upstream of the dam are apparent in Fig. 5-9. They are described as follows:

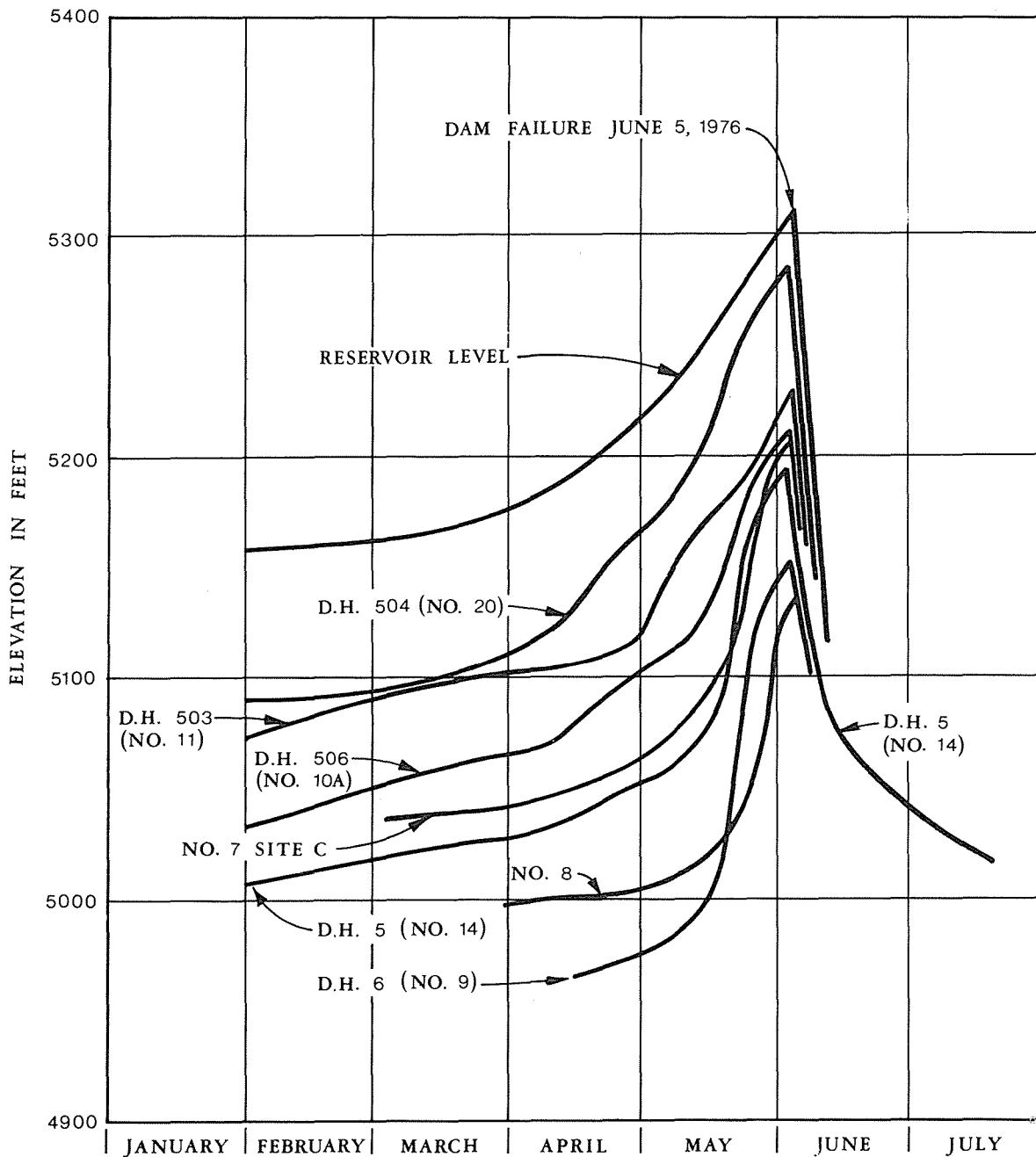
Unit 1, the uppermost layer, consists of lenticular and tabular plates mainly 2 to 6 in. thick, but some are up to 18 in. thick (Fig. 5-10). The plates are nearly horizontal and parallel the joint foliation in the rock. Open partings between plates are ¼ in. to 2 in. wide. Some are coated with calcite layers up to ¼ in. thick. Caliche and silt fill some of these openings in the upper 5 to 6 ft of the unit. High angle joints are scarce.



REFERENCE DATA:
 U. S. BUREAU OF RECLAMATION
 DWG. NO. 549-100-116

FIG. 5-6.

U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
 INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



1976

SEE FIG. 5-6 FOR LOCATIONS D.H. 506 EQUIPPED WITH CONTINUOUS WATER STAGE RECORDER. ELEVATIONS SHOWN FOR THE OTHER WELLS ON JUNE 5, 1976 WERE EXTRAPOLATED FROM JUNE 1 WATER LEVEL MEASUREMENTS.

WATER LEVEL ELEVATIONS IN OBSERVATION WELLS

FIG. 5-7. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



Fig. 5-8 Showing prevalent horizontal joints upstream and dominating vertical joint system downstream of dam in right wall of canyon (post-failure photo)



Fig. 5-9 The ash-flow tuff upstream of dam axis is divided into three units (post-failure photo)



Fig. 5-10 Slab-like structure in Unit 1 of ash-flow tuff exposed in rim of downstream wall of keyway in right abutment. (post-failure photo)



UNIT 2

Fig. 5-11 Joint pattern in Unit 2 of ash-flow tuff, right wall of canyon upstream from key trench. (post-failure photo)

Unit 2, the intermediate layer, is gradational with the overlying Unit 1, the contact occurring at about El. 5250 ft. Jointing is moderate to intense. Horizontal partings prevail, but east-west trending high angle joints spaced at 10 to 20 ft intervals also are prominent (Fig. 5-11). Minor random joints are generally spaced 1 to 2 ft apart. Most joints are open from 1/8 in. to 1 in. and are coated or filled with calcium carbonate. Flow lineations due to flattened lapilli fragments and flattened vugs are near vertical. Contact with the underlying unit is marked by a breccia zone 6 in. to 2 ft thick consisting of rock fragments cemented with calcium carbonate. The contact zone is located at about El. 5185 ft. It is nearly flat lying, but in some places it is a wavy, irregular surface with openings ranging from 1/4 to 3 in.

Unit 3 forms bold outcrops in the lower abutment from El. 5060 to 5185 ft (Fig. 5-9). Near vertical joints are prominent and can be traced for over 100 ft. The dominant joint trend is northwesterly with lesser northeasterly trends. Spacing is commonly 5 to 10 ft, with openings ranging from 1/4 in. to as much as 3 in. Most joints are stained with iron and manganese oxides. Separation along the dominating low angle joint planes has led to the development of prominent benches along the canyon wall.

As previously mentioned, Unit 2 is not recognizable in the right wall of the canyon downstream of the dam. Here extensive, steeply dipping joints prevail (Fig. 5-12). The transition from predominantly near-horizontal to near-vertical jointing occurs near the axis of the dam (Fig. 5-13).

DH-505, located at the extreme right end of the dam (Fig. 5-5), is slanted into the abutment at an angle of 30 degrees below horizontal and is oriented N20W along the projected bearing of the axis of the dam. It encountered several open joints. No grout was detected in the drill core; notwithstanding, the hole had been completed on November 20, 1974 subsequent to emplacement of the grout curtain. During drilling, all water return was lost at a depth of 70 ft and was not regained throughout the remainder of the operation. No water pressure tests were made because of caving and ravelling conditions. However, percolation tests made at two depth intervals are summarized as follows:

<u>Date</u>	<u>Drill Hole 505</u> (ft)	<u>El.</u> (ft)	<u>Vertical Depth</u> (ft)	<u>Quantity Injected</u> (gals)	<u>Duration</u> (min)	<u>Discharge</u> (gpm)
11/12/74 ^(A)	199.8	5234	100	2300	40	57.5
11/19/74 12:40 p.m. ^(B)	399.8	5134	200	5705	90	
11/19/74 2:15 p.m. ^(B)	399.8	5134	200	7970	135	

(A) Unable to raise water level to surface.

(B) Unable to raise water level above El. 5136.

The drillers of DH-505 reported open joints between Els. 5272 and 5274 and a 0.6-ft seam at 5248 ft (Fig. 5-5). Thus, the existence of permeable joints beyond the end of the dam is established.



Fig. 5-12 Prominent vertical joints in right wall of canyon downstream of the axis of the dam. (post-failure photo)

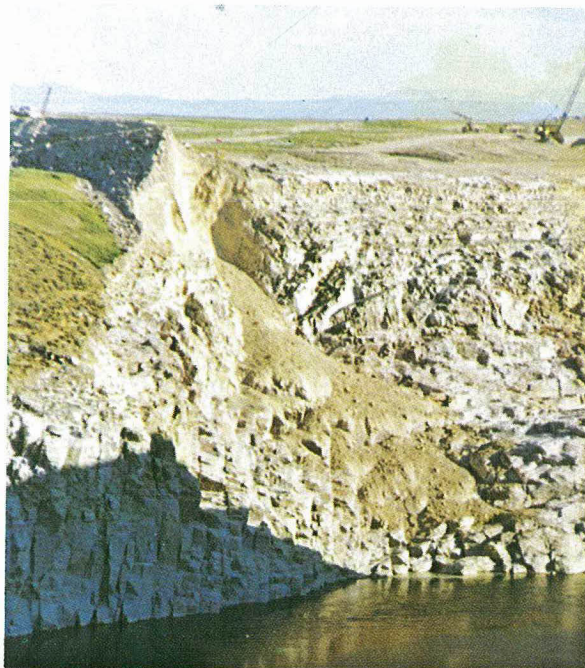


Fig. 5-13 Transition from predominantly flat-lying to near-vertical jointing occurs near axis of dam. (post-failure photo)

During filling of the reservoir in Spring 1976, water levels were monitored in a number of observation wells located in vicinity of the dam. Water levels in these holes responded to the rise in the reservoir stage, indicating hydraulic interconnection through the joint systems. The locations of these wells and hydrographs reflecting the correlations are shown on Figs. 5-6 and 5-7.

Some confusion arises as a result of the several different systems that have been used for numbering water level-observation wells. The Bureau of Reclamation has assigned two separate numbers to wells which were drilled for foundation exploration and later included in the groundwater monitoring program. In addition, the U.S. Geological Survey has assigned numbers following its customary well numbering system. Furthermore, local irrigation and domestic wells are often referred to by the owner's name. Table 5-3 provides a cross-index of these well designation practices.

The rapid rise of the water table in response to reservoir filling indicates that joints in the canyon walls extend into and beyond the right abutment, and that the rock is permeable. It is apparent from Fig. 5-7 that at holes DH-5, DH-6, DH-503, DH-506, and Observation Wells Nos 7 and 8, the rise in water table was more rapid than that of the reservoir level during May and the first week of June 1976. This condition is attributed to flow through dominant horizontal joints that exist in rock Units 1 and 2 exposed in the right wall of the reservoir. As the reservoir reached the levels of these joints, water appears to have flowed along them and to have caused a more rapid rise in water elevation in the drill holes. A particularly rapid rise in groundwater occurred in DH-5 when the reservoir stage reached El. 5250, commencing about 18 days before failure of the dam. Since DH-5 is located downstream of the key trench, this rise indicates that some water from the reservoir was bypassing the dam, possibly flowing around the right end through interconnecting joint avenues. However, on the day of failure the water level in DH-5 was 104 feet lower than the reservoir.

During geologic exploration of the site, prior to commencement of dam construction, water pressure testing of Nx-diameter drill holes in and near the dam abutments had in several instances shown high water losses. A listing of tests in which leakage exceeded 50 gpm is shown in Table 5-4. In addition to the customary pressure tests, experiments were conducted wherein water was pumped into drill holes in the right abutment. The injections were metered, and the effects on groundwater levels in other drill holes in the abutment area were observed. Fig. 5-14 depicts the results of a pump-in test at DH-303 wherein over 24 acre-ft was injected over a 15-day period. Running at maximum capacity, the pump for this test delivered a discharge of 440 gpm to DH-303 without filling it. Water levels rose in holes DH-5, 6, 204, 301, and 302 during the period of injection and dropped abruptly upon its termination. Results of the pump-in test at DH-303 confirm the openness and intercommunication of the joint system in the volcanic rock in the right abutment between DH-204 near the wall of the canyon and DH-6 which is located about 1100 ft west of the right end of the dam.

Maps of the rock joints and geologic sections along the right abutment key trench invert prepared after failure of the dam are contained in Appendix E where additional photographs and detailed descriptions of joints are also found. The mapping program is discussed further in Chapter 3 under the section entitled Rock Joint Survey.

Rock Cavities.

During excavation of the dam foundation, large openings were uncovered in left and right abutment key trenches. Near the right end of the dam, two large fissures were exposed near Stas. 3+55 and 4+34. These are shown in plan and cross-section in Figs. 5-15 and 5-16. Figs. 5-17 and 5-18 show the exterior and interior of the fissure near Sta. 4+34. Both fissures trend generally east-west and cross the axis.

TABLE 5-3
CROSS-INDEX OF NUMBERING SYSTEMS FOR WELLS AND DRILL HOLES
USED FOR MONITORING WATER LEVEL ELEVATIONS

USBR Observation Well No.	USBR Exploration Hole Designation	USGS No.**	Local Designation and Comments
1		6N/41E-11cdl	Clyde Packer Irrigation Well. Equipped with Stevens A-35 Recorder.
2		7N/41E-25cbl	Trupp Irrigation Well.
4		7N/42E-6ddl	Remington Irrigation Well.
5		7N/42E-8cal	Schwendiman Well.
6	Site A	7N/42E-17bcl	Equipped with Stevens A-35 Recorder.
7*	Site C	7N/42E-19abl	
8*		7N/42E-19ccl	Remington Irrigation Well.
9*	DH-6	7N/42E-19cdl	Angle Hole — Dip 30 ^o From Horizontal.
10A*	DH-506	7N/42E-19dcl	Deep Piezometer Monitors Underlying Lake and Streambed Deposits.
10B*	DH-506	7N/42E-19dcl	Shallow Piezometer.
11*	DH-503	7N/42E-29bdl	
12*	Corps Engrs No. 2	7N/42E-29bcl	
13A*	DH-507	7N/42E-30ad	Deep Piezometer. Destroyed June 5, 1976. Monitored "Lake and Streambed Sediments."
13B*	DH-507	7N/42E-30adl	Shallow Piezometer. Destroyed June 5, 1976.
14*	DH-5	7N/42E-30abl	
15	Site D	7N/42E-32bbl	Equipped with Stevens Type F Recorder.
17	DH-8	7N/43E-16cbl	Plugged at About 400 ft.

TABLE 5-3 (cont.)

USBR Observation Well No.	USBR Exploration Hole Designation	USGS No. *	Local Designation and Comments
18	DH-7	7N/43E-21aal	Piezometer Open to Basalt Below 467 ft.
19		7N/42E-15abl	Supply Well, Hog Hollow Recreation Area.
20*	DH-504	7N/42E-19ddcl	
21*	DH-501	7N/42E-19dcd2	Angle Hole – Dip 60° From Horizontal.

* Observation wells located in vicinity of dam, Fig. 5-6.

**Designates well location based on Township, Range, Section, Quarter-section, and Quarter-quarter section. Final number indicates whether the well was 1st, 2nd, 3rd, etc., to be drilled in the area.

TABLE 5-4
SUMMARY OF EXCEPTIONALLY HIGH WATER LOSSES*
EXPERIENCED IN DRILL HOLES IN RIGHT ABUTMENT
DURING WATER PRESSURE TESTING
 (Locations shown on Figs. 5-5 and 5-6)

Drill Hole No.	Depth Interval (ft)	Water Pressure (psi)	Water Loss (gpm)
5	51.8- 61.8	25	68.7
	61 - 71	25	56.6
	41 - 71	25	60
	73.3- 83.3	35	70
	117 -127	25	52.3
	102 -132	50	58
	141 -151	25	54.5
	100.6-160.6	50	55.2
	169.7-179.7	50	56.5
	190.3-200.3	25	64.6
	160.3-200.3	25	55.7
	210.2-220.2	25	42.1
	190.2-220.2	35	55.1
	160.2-220.2	50	52
	222.6-232.6	25	62.1
	242.4-252.4	25	59.7
	251.9-261.9	40	62.5
6	27 - 77	85	52.4
	71.9-100.4	25	59.8
	140.4-190.4	90	50.1
	179.3-229.2	95	54.6
	254.6-304.6	100	51.6
	294.0-344	100	54.6
102	15.5-306.1	6	112
301	29 - 39	50	120
	49 - 59	50	110
	84.7- 94.7	50	100
	94.7-104.7	25	150
	104.7-114.7	25	100
	134.5-144.5	25	162
	144.5-154.5	90	57
	164.2-174.2	20	150
	174.6-184.6	20	150
	215.7-235.7	90	74
235.5-255.5	100	32	

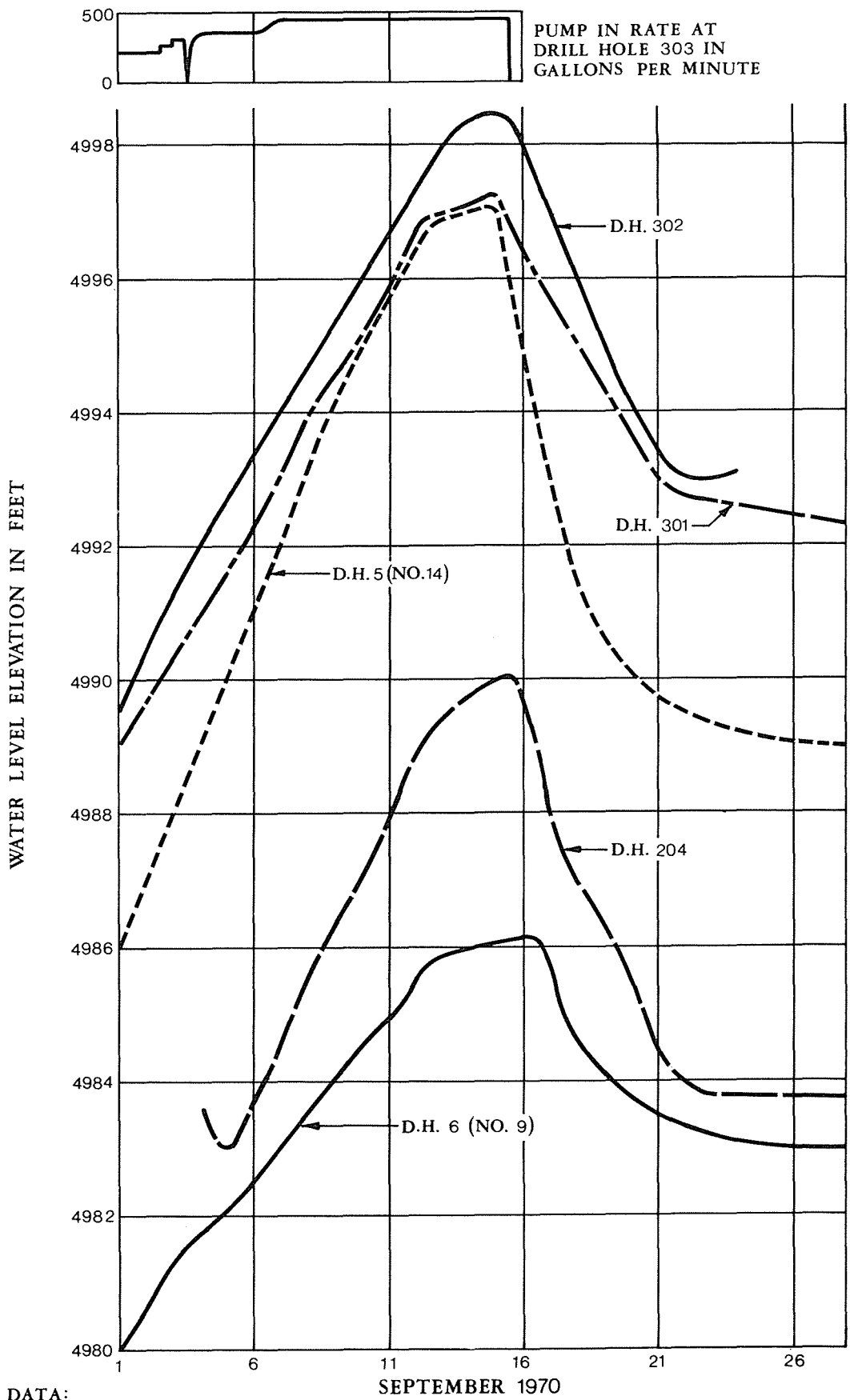
*Arbitrarily selected as losses exceeding 50 gpm.

TABLE 5-4 (cont.)

<u>Drill Hole No.</u>	<u>Depth Interval (ft)</u>	<u>Water Pressure (psi)</u>	<u>Water Loss (gpm)</u>
301 (cont.)	255.5-271.7	50	53.7
	270.2-290.2	50	70
	289.5-309.5	50	90
	349.5-389.5	30	100
302	24 - 39	50	112
	49.4- 59.4	90	62.1
	89.3- 99.3	30	157
	99.2-109.2	20	164
	126.4-136.4	50	79.6
	173.2-183.2	15	170
	181.8-191.8	15	150
	191 -211	20	150
	201 -211	25	120
	211.2-221.2	0	180
	269.5-289.5	0	165
	311.7-331.7	0	170
	370.9-390.9	0	180
	303	42.5- 52.2	10
52.3- 60.3		0	75
60.3- 70.3		0	75
70 - 80		0	75
80 - 90		0	93
90 -100		80	100
98.8-109.8		0	179
119.6-129.6		0	165
188.6-198.6		0	176
267.7-277.7		0	110
504	597	0	Pump-in test 11/20/74. Pumped in 6,589 gal from 8:00 a.m. to 4:00 p.m. Water level at 4:00 p.m., 185.0 ft.
505	<p>Excerpt from USBR drill hole log:</p> <p>No percolation tests taken because of caving and ravelling conditions.</p> <p>Seam at 171.3 where drill rods dropped 0.6 ft after loosening chuck.</p> <p>120.6-123.5 "Void determined to be at least 3" wide"</p> <p>124-131 "Void determined to be at least 3" wide"</p>		

TABLE 5-4 (cont.)

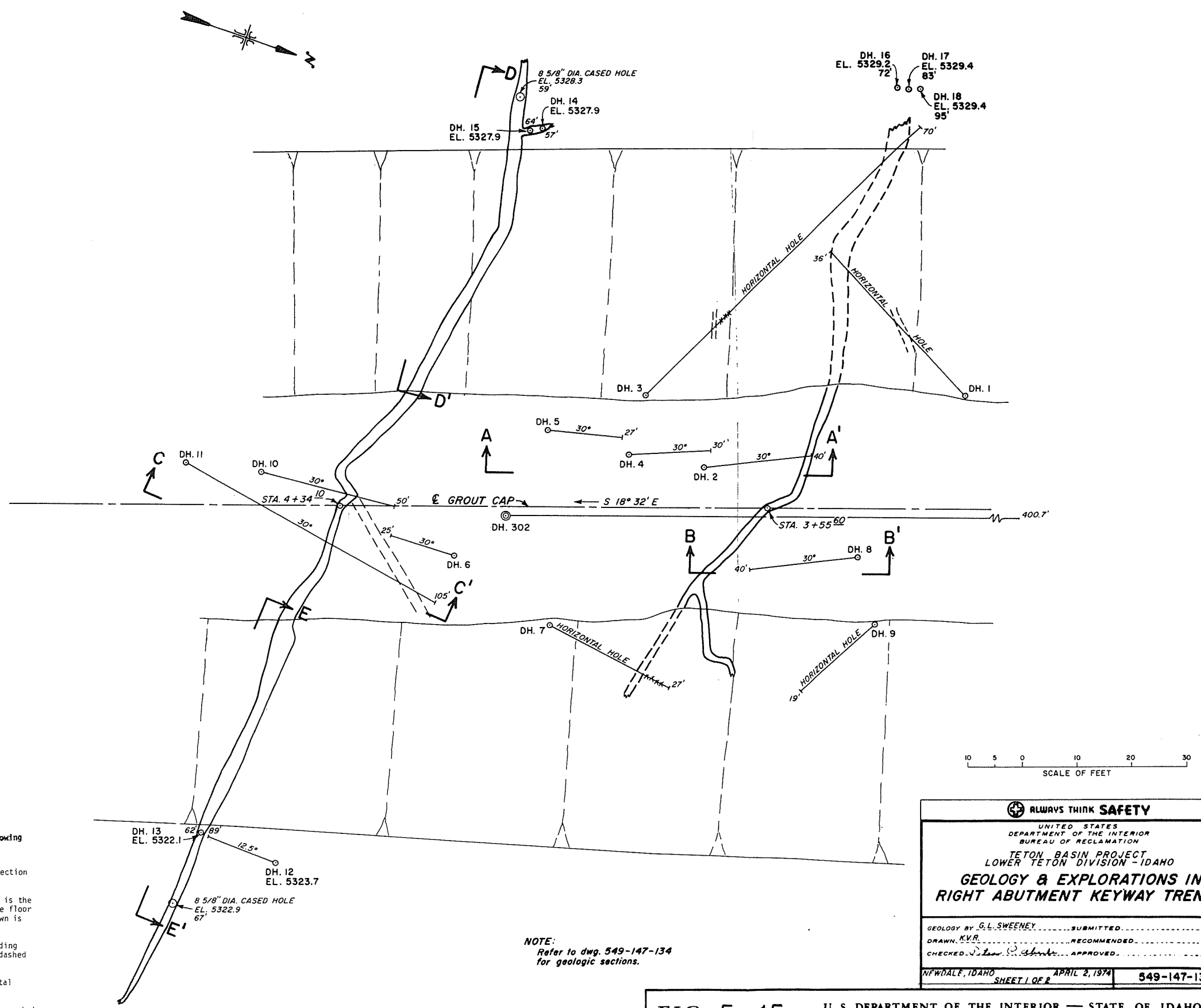
Drill Hole No.	Depth Interval (ft)	Water Pressure (psi)	Water Loss (gpm)
505 (cont.)	137.1-138.9 "Void space up to 1" locally along joint" 139.8-141 "Scattered void spaces mostly less than 1/2" open along joint" 143.3-144.6 "Scattered void spaces mostly less than 1/2" open along joint" "Hole at depth 199.8. Pumped through wire line rods with rods pulled back to 160.0 ft. Pumped in 2300 gallons in 40 minutes. (57.5 gpm) – unable to raise water level to surface." This test was performed 11/12/74 after installation of grout curtain. No grout was found in the core.		



REFERENCE DATA:
U. S. BUREAU OF RECLAMATION

EFFECT OF PUMP-IN TEST AT DRILL HOLE 303 ON WATER LEVELS IN NEIGHBORING DRILL HOLES

FIG. 5-14. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



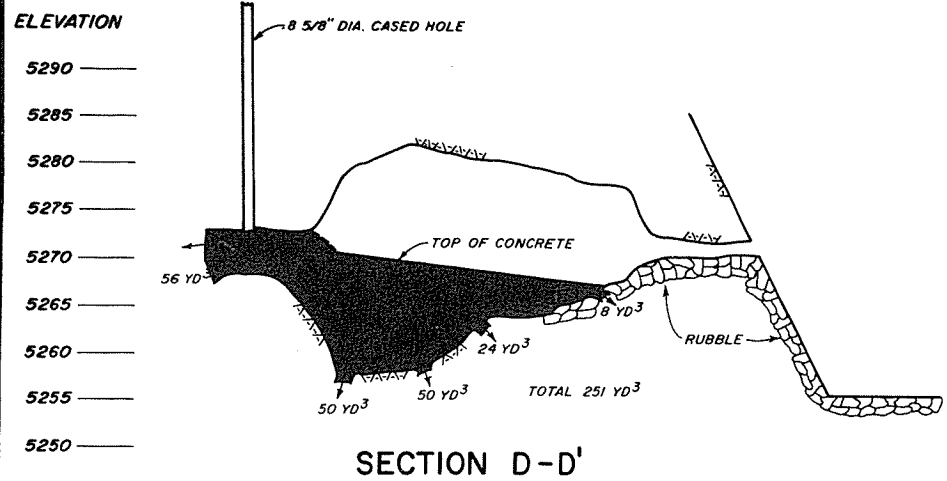
EXPLANATION

- DH. 13
62' Vertical air track drill hole showing depth to top of fissure.
- DH. 5 Air track drill hole showing direction and slope of hole from vertical.
- DH. 302 Core drill hole - Location shown is the intersection of the hole with the floor of the keyway trench. Depth shown is from original ground surface.
- Boundaries of fissure zones, including zones filled with broken rock - dashed where inferred.
- ||| Void zone intercepted by horizontal drill holes.
- *** Zone of broken or loose rock intercepted by horizontal drill holes.

NOTE:
Refer to dwg. 549-147-134 for geologic sections.

ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION	
TETON BASIN PROJECT LOWER TETON DIVISION - IDAHO	
GEOLOGY & EXPLORATIONS IN RIGHT ABUTMENT KEYWAY TRENCH	
GEOLOGY BY G.L. SWEENEY	SUBMITTED
DRAWN BY K.V.R.	RECOMMENDED
CHECKED BY <i>[Signature]</i>	APPROVED
NEWDALE, IDAHO	APRIL 2, 1974
SHEET 1 OF 2	549-147-133

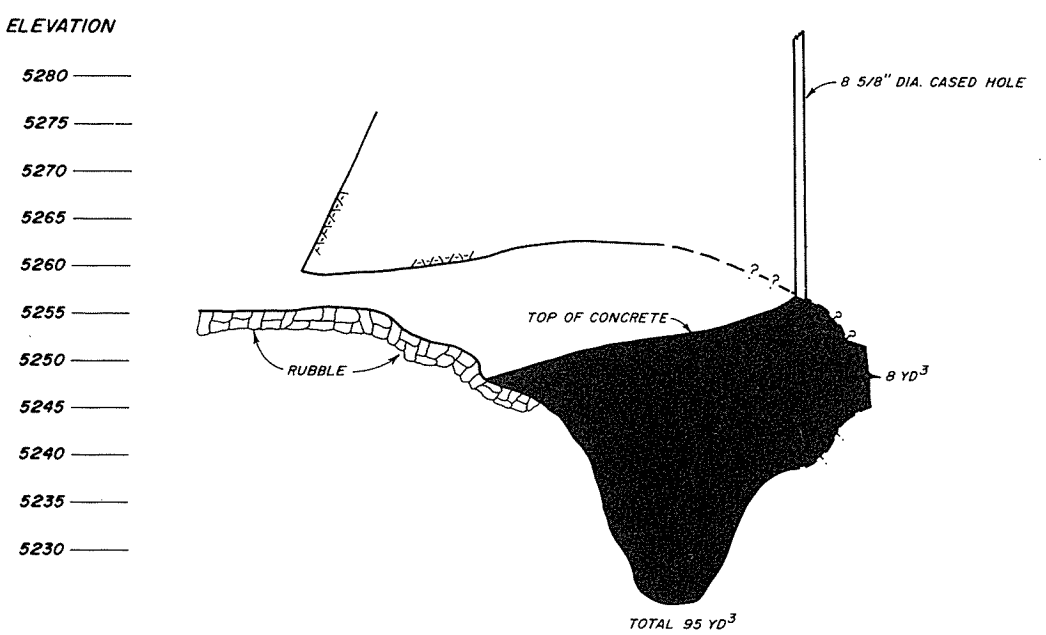
FIG. 5-15. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



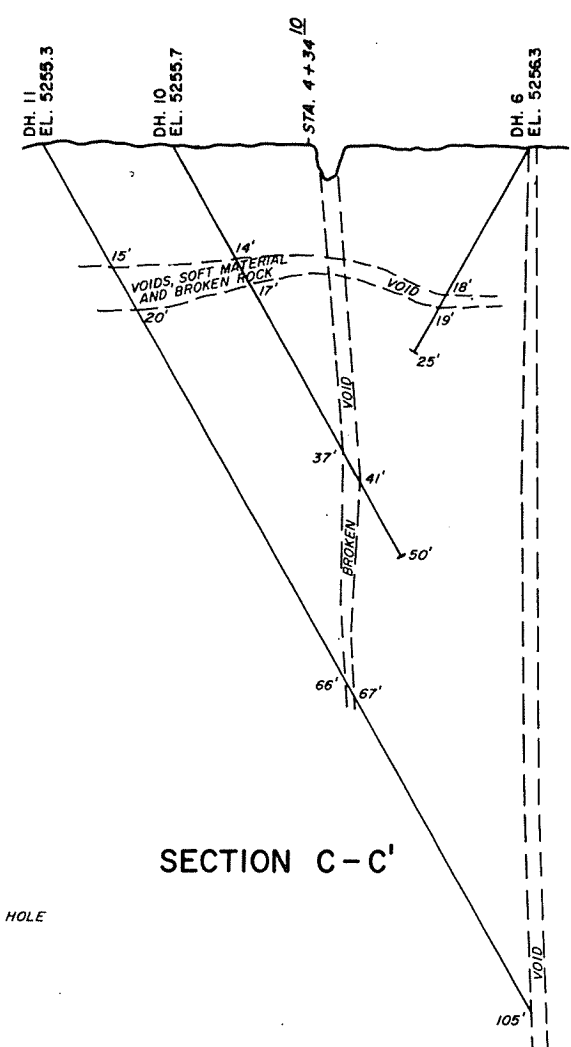
SECTION D-D'

Placed 4-27-74
 Placed 5-6-74

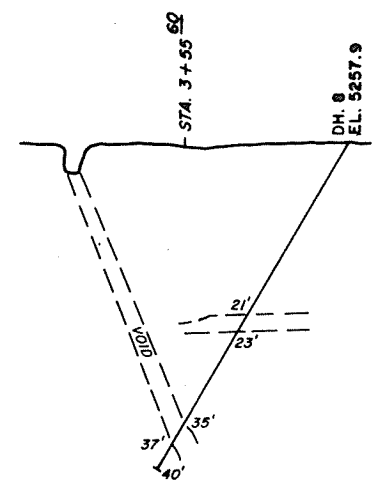
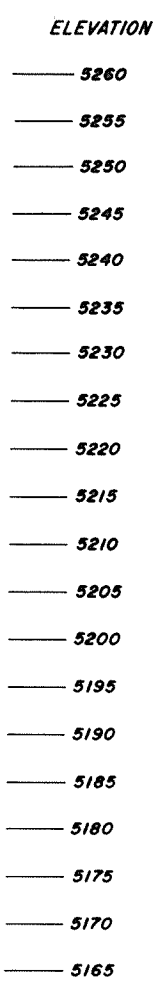
Rock outline in fissure openings estimated.



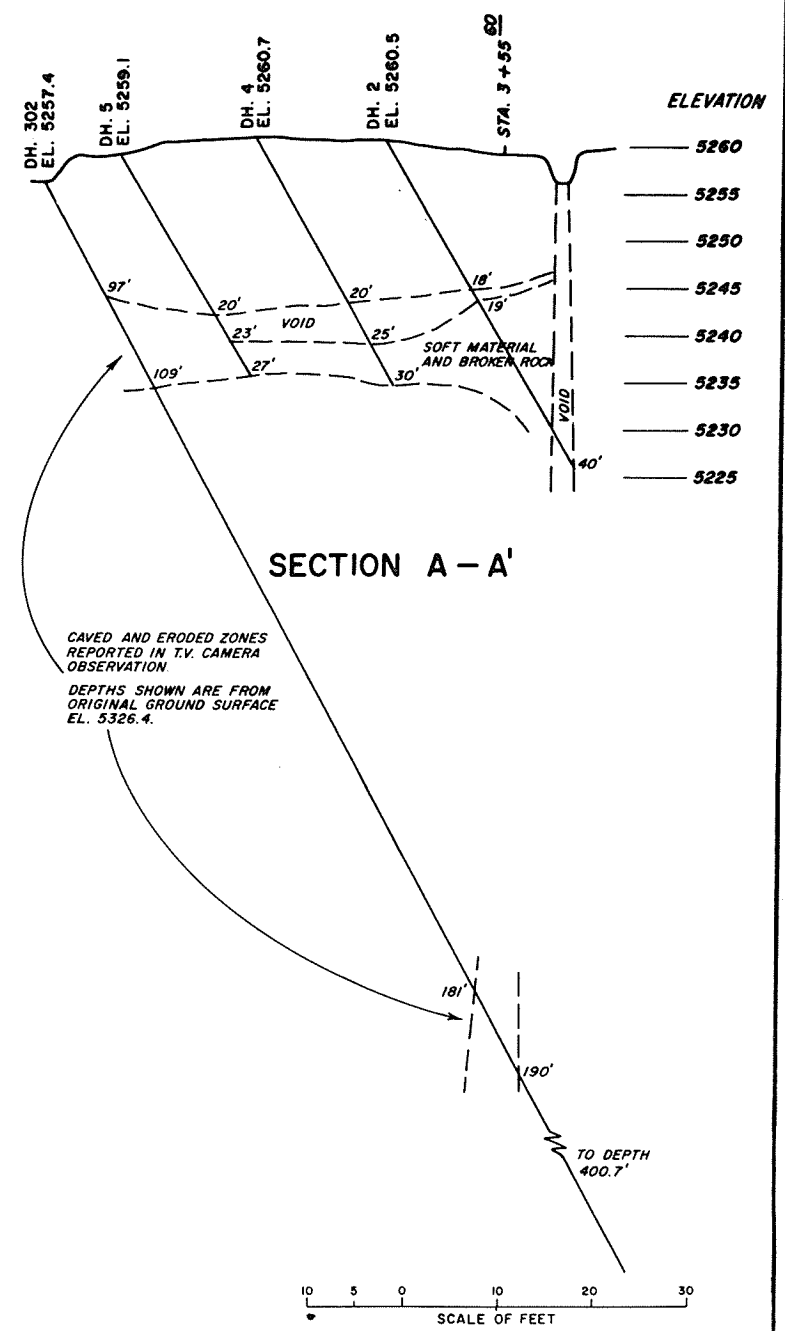
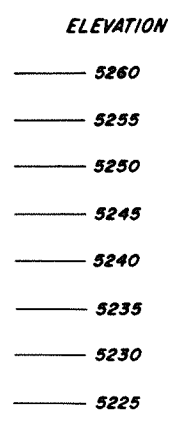
SECTION E-E'



SECTION C-C'

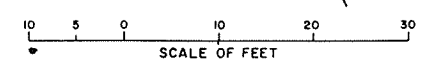


SECTION B-B'



SECTION A-A'

CAVED AND ERODED ZONES
 REPORTED IN T.V. CAMERA
 OBSERVATION
 DEPTHS SHOWN ARE FROM
 ORIGINAL GROUND SURFACE
 EL. 5326.4.



NOTE:
 Refer to dwg. 549-147-133 for plan.

ALWAYS THINK SAFETY

UNITED STATES
 DEPARTMENT OF THE INTERIOR
 BUREAU OF RECLAMATION
 TETON BASIN PROJECT
 LOWER TETON DIVISION-IDAHO

**GEOLOGIC SECTIONS ACROSS FISSURES IN
RIGHT ABUTMENT KEYWAY TRENCH**

GEOLOGY BY G. L. SWEENEY SUBMITTED
 DRAWN BY M. V. R. RECOMMENDED
 CHECKED BY Peter C. Adams APPROVED

NEWDALE, IDAHO SHEET 2 OF 2 APRIL 2, 1974 549-147-134

FIG. 5-16. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



Fig. 5-17 Rock fissure near Sta. 4+34 discovered during excavation of key axis trench of the dam. Crevice extends through axis of dam for nearly 100 ft both upstream and downstream. (1974)



Fig. 5-18 Interior of fissure shown in Fig. 5-17. Photo taken 10 ft from upstream wall of key trench. Spot in background is light from a bulb suspended through a 6-in. vertical drill hole located about 25 ft from camera. (1974)

The fissure near Sta. 4+34 was entered and explored by a Bureau of Reclamation employee for a distance of about 100 ft both downstream and upstream of the dam axis and an estimated 100 ft below the key trench invert. An interview with him is summarized in a memorandum dated November 16, 1976, contained in Appendix B. He described the cavity downstream as fairly consistently about 4 ft wide with a floor strewn with angular blocks of rock measuring up to 4 or 5 ft on a side. Upstream from the keyway, the roof and floor of the cavity were reported lined with stalactites and stalagmites up to 3/8 in. in diameter. About 100 ft upstream from the keyway wall the fissure pinched and turned so that the end could not be seen. It was reported that in winter vapor could be seen emerging from the downstream segment and that this segment was warm and could be entered in winter without a coat. Conversely, upstream of the keyway the air was reported to be cold. The end of the downstream segment was blocked by a large rock "the size of a pickup truck." A room or passage could be seen beyond, but the opening into it was too small to enter. In one place the cavity walls were described as covered with a red coating "which rubbed off on our clothes."

The interior of the fissure at Sta. 3+55 was not examined since it was too narrow to permit entry. High-slump concrete was poured into both fissures during project construction. (Fig. 5-16 and Appendix B document dated August 25, 1976.) The extent to which the uppermost parts of the cavities may have been sealed by this procedure is uncertain. However, the concrete-rock contact was drilled at three points during post-failure exploration, and in each instance the rock cores obtained displayed a tight bond between grout and rock (Chapter 3).

The Bureau of Reclamation has referred to these cavities as tensional cooling cracks modified by ascending hot gasses and water vapor (Gebhart, L.R., et al, March 1974). Disruption caused by steam ascending from the saturated lake and stream deposits upon which they were emplaced could possibly have been a contributing factor. Another theory is that they are tension cracks that developed as the result of tectonic deformation.

Additional description of these fissures and their treatment is contained in a memorandum to the Director of Design and Construction of the U.S. Bureau of Reclamation from the Project Engineer, Newdale, Idaho, dated March 14, 1974, Appendix E.

Several other large fissures are exposed in the right wall of the canyon, one-eighth to one-quarter-mile upstream from the dam (Figs. 5-19 and 5-20). The walls of a few of these openings are curved and parallel with configurations such that opposite walls "fit" like the mating pieces of a jigsaw puzzle.

Subterranean cavities in the region are commonly reported by local well drillers. A water well drilled in Section 18, northwest of the right abutment, is said to have encountered a void into which the drill bit dropped about 6 ft. At another well, the well bailer was lost in a cavity. The hole was subsequently abandoned and a new well started a few feet away. During this redrilling, the cavity was again encountered and the lost bailer snagged and fortuitously retrieved.

The genesis of the cavities that have been observed or reported is controversial and perhaps academic. However, there is no question that they increase the permeability of the abutment and may serve as significant feeders to other joint conduits. Other cavities may lie undiscovered deep in the dam foundation.

Faults.

Although Teton Dam is situated in a seismically active region, there are remarkably few identifiable faults in vicinity of the damsite. The closest two faults are located, respectively, about 10 miles upstream and 10 miles downstream.



Fig. 5-19 Opening in upper right wall of Teton Canyon 1/8 to 1/4 mile upstream from dam. (post-failure photo)



Fig. 5-20 Joint in right wall of Teton Canyon 1/8 to 1/4 mile upstream from dam. (post-failure photo)

The U.S. Geological Survey has suggested the possible existence of a northeast trending fault in the right abutment of the dam (Oriel et al, 1973). Its approximate location is shown on Figs. 5-1 and 5-3 as inferred by the USGS from aerial photographs and from geologic sections prepared as part of the Bureau of Reclamation's groundwater investigation of the Rexburg Bench (Haskett, 1972). The aerial photographs of the area west of the dam were interpreted as showing surface breaks in slope similar to lineaments sometimes manifested by eroded fault scarps. In some of the geologic sections, based upon information from water well logs, the ends of the lenses of volcanic and sedimentary deposits show a near-vertical alignment, suggesting possible truncation by a fault plane (Figs. 5-2 and 5-3).

Inspection of the area through which faulting was inferred has failed to disclose positive evidence of the existence of a fault. Furthermore, as discussed in Chapter 6, no significant earthquakes were detected on the day of the failure by the sensitive seismographs that monitor this region. Should this fault exist near the right abutment as postulated, it is improbable that it played a significant role in the failure of the dam.

Foundation Deformation.

Teton Dam rests directly upon a foundation of welded tuff and basalt which in turn overlies the lake and stream deposits (Fig. 5-4). Physical properties of the volcanic rocks indicate these rocks to be strong and rigid except where locally weakened by joints (Table 4-1).

Logs of exploration holes drilled into the lake and stream sediments describe some lenses of sediments as soft, friable, or lightly compacted (Table 5-1). These descriptions imply that the deposits are compressible. However, they could undergo further consolidation in place only if subjected to pressures that exceeded maximum historic loads. The heaviest loading at the site probably existed when the tuff was first emplaced, before Teton Canyon was incised by the river. Subsequent removal of rock during erosion of the canyon reduced this initial load and was undoubtedly attended by some elastic rebound. Construction of Teton Dam and filling of the reservoir restored only a fraction of the initial load inasmuch as the combined weight of water and embankment was considerably less than that of the rock that earlier occupied the reservoir site. While the weight of the dam and impounded water surely caused some elastic strain in the foundation, significant settlement is not expected to have occurred.

Clearly, the lake and stream deposits have borne the weight of the overlying ash-flow tuff since its deposition about 2 million years ago, and they have had the ensuing period during which to consolidate. At the dam the load is presently approximately 570 ft of rock and soil on the right abutment, over 200 ft on the left abutment, and about 290 ft of alluvium and rock in the river channel.

That no significant deformation has occurred at the dam since its construction is evidenced by the results of geodetic and leveling surveys conducted before and after its failure. Tables 5-5 and 5-6 compare pre- and post-failure elevations and positions of survey stations, the locations of which are shown on Fig. 5-21. Table 5-7 makes a comparison of benchmark elevations along part of the right abutment grout cap. No significant changes in elevation or position of the unmolested stations are indicated to have occurred.

Inspections of the auxiliary outlet tunnel that passes through the right abutment disclosed no observable cracks in the concrete lining, providing further evidence that no significant deformation has occurred in this abutment, at least along the tunnel alignment, since completion of construction.

**TABLE 5-5
COMPARISON OF PRE- AND POST-FAILURE ELEVATIONS
OF BENCHMARKS IN VICINITY OF TETON DAM**

<u>Benchmark Designation*</u>	<u>Elevations in Feet</u>			
	<u>Pre-failure</u>	<u>(Date of Survey)</u>	<u>Post-failure</u>	<u>Change, ft</u>
Tri Station BEV	5299.22	(4/24/72)	5299.18	-.04
Tri Station CORA	5289.62	(4/24/72)	5289.62	.0
Tri Station DOT	5280.96	(4/25/72)	5280.93	-.03
Tri Station ALICE	5298.20	(4/24/72)	5298.16	-.04
Photo Pt. 3-19	5323.69	—	5323.72	+03
Tri Station No. 7	5310.54	(7/24/69)	5310.52	-.02
Station 0+00	5335.79	(5/7/74)	5335.67	-.12
SS Bolt 17+79 Lt.	5331.95	(4/16/76)	5331.91	-.04
Pt. 16+25 P.I. R.O.W.	5322.63	(10/17/74)	5322.67	+04
Tri Station No. 3	5290.51	(4/19/72)	5290.44	-.07
SS Bolt 34+35.5 Rt.	5038.43	(8/18/75)	5038.33	-.10
	5038.41	(6/3/76)	5038.33	-.08
4' x 4' Gate	5045.88	Prior to 6/5/76	D/S Lt. 5045.80 D/S Rt. 5045.81 U/S Lt. 5045.82 U/S Rt. 5045.82	
Tri Station B-Pt.-9	5271.76	(4/19/72)	5271.71	-.05
Cor. No. 3	5342.37	(10/17/74)	5342.35	-.02

*Locations shown on Fig. 5-21.

TABLE 5-6
COMPARISON OF DISTANCES BETWEEN SURVEY STATIONS
MEASURED BEFORE AND AFTER FAILURE OF TETON DAM

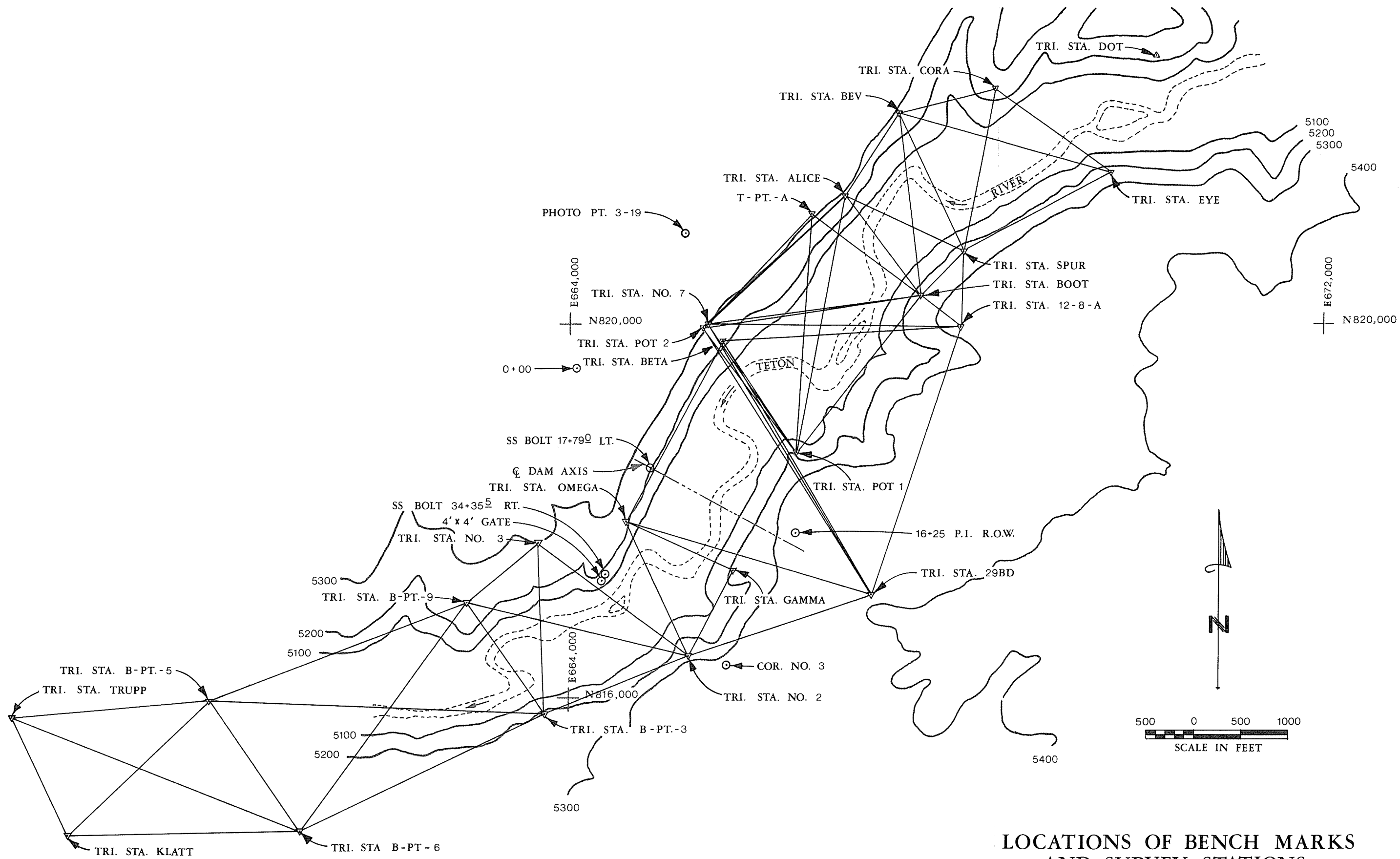
Tri. Sta ¹	To	Tri. Sta.	Distance Before	Distance After	Change in Distance (ft)
Trupp	—	B-Pt-6	3,323.586	3,323.615	.029+
Trupp	—	B-Pt-5	2,148.384	2,148.332	.052-
Trupp	—	Klatt	1,387.410	1,387.452	.042+
Klatt	—	B-Pt-5	2,078.905	2,078.873	.032-
Klatt	—	B-Pt-6	2,496.616	2,496.609	.007+
B-Pt-5	—	B-Pt-6	1,651.849	1,651.826	.023-
B-Pt-5	—	B-Pt-3	3,555.890	3,555.838	.052-
B-Pt-5	—	B-Pt-9	2,901.992	2,902.008	.016+
B-Pt-6	—	B-Pt-9	2,980.990	2,980.964	.026-
B-Pt-6	—	B-Pt-3	2,868.941	2,868.934	.007-
B-Pt-9	—	B-Pt-3	1,476.568	1,476.533	.035-
B-Pt-9	—	# 3	982.586	982.600	.014+
B-Pt-9	—	# 2	2,459.435	2,459.432	.003-
B-Pt-3	—	# 3	* 1,810.686	1,810.632	.054-
B-Pt-3	—	# 2	1,654.797	1,654.787	.01-
# 3	—	# 2	2,005.236	2,005.210	.026-
# 2	—	Omega	* 1,577.374	1,577.364	.01-
# 2	—	Gamma	* 1,028.444	1,028.677	.233+
# 2	—	29BD	* 2,048.537	2,048.538	.001+
Omega	—	Beta	2,182.560	2,182.517	.043-
Omega	—	29BD	* 2,723.499	2,723.470	.029-
Omega	—	Gamma	1,246.310	1,246.286	.024-
29BD	—	# 7	* 3,367.434	3,367.481	.047+
29BD	—	Beta	* 3,122.492	3,122.493	.001+
29BD	—	12-8-A	* 2,995.038	2,994.899	.039-
Pot #1	—	Boot	2,149.352	2,149.365	.013+
Pot #1	—	Alice	2,809.447	2,809.482	.035+
Pot #1	—	Pot #2	1,656.547	1,656.536	.011-
Pot #1	—	# 7	1,670.932	1,670.939	.007+
12-8-A	—	Beta	* 2,513.037	2,512.972	.065-
12-8-A	—	# 7	* 2,672.141	2,672.082	.059-
12-8-A	—	T-Pt-A	* 1,985.313	1,985.315	.002+
Pot #2	—	Alice	2,056.381	2,056.397	.016+
Pot #2	—	Boot	2,319.802	2,319.667	.135-
# 7	—	T-Pt-A	* 1,591.714	1,591.633	.081-
# 7	—	Alice	1,976.380	1,976.340	.04-
# 7	—	Boot	2,255.677	2,255.518	.159-

*Distances computed from coordinates.

¹See Fig. 5-21 for locations.

TABLE 5-6 (cont.)

<u>Tri. Sta¹</u>	<u>To</u>	<u>Tri. Sta.</u>	<u>Distance Before</u>	<u>Distance After</u>	<u>Change in Distance (ft)</u>
Boot	—	Alice	1,336.178	1,336.005	.173-
Boot	—	Bev	1,937.760	1,937.606	.154-
Boot	—	Spur	643.659	643.706	.047+
Spur	—	Alice	1,385.799	1,385.761	.062-
Spur	—	Bev	1,609.437	1,609.380	.057-
Spur	—	Cora	1,757.141	1,757.113	.028-
Alice	—	Bev	1,025.878	1,025.854	.024-
Bev	—	Eye	2,353.080	2,353.064	.016-
Bev	—	Cora	1,052.972	1,052.960	.012-
Cora	—	Eye	1,533.391	1,533.360	.031-
Eye	—	Spur	1,796.022	1,796.008	.014-



LOCATIONS OF BENCH MARKS AND SURVEY STATIONS

REFERENCE DATA:
 U. S. BUREAU OF RECLAMATION
 DWG. NOS. 549-147-2075; 549-147-2076

FIG. 5-21

U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
 INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

**TABLE 5-7
A COMPARISON OF
ELEVATIONS OF POINTS ON THE RIGHT ABUTMENT
GROUT CAP AS DETERMINED FROM SURVEYS
MADE BEFORE AND AFTER FAILURE
OF THE DAM**

<u>Grout Cap Station</u>	<u>Elevations in Ft</u>		<u>Change, ft</u>
	<u>Pre-failure</u>	<u>Post-failure</u>	
11+72	5275.617	5275.618	+0.001
12+09	5254.120	5254.119	-0.001
12+16	5250.483	5250.462	-0.021
12+50	5223.524	5223.514	-0.010
12+60	5222.320	5222.312	-0.008
13+27	5198.422	5198.415	-0.007

Survey points consisted of straight lengths of Number 6 reinforcing bar which were embedded in the concrete grout cap. During dam construction, the contractor bent over or cut off those bars that threatened damage to his rubber-tired equipment. The stations included in this table were inspected by a Panel representative and judged to be unmolested.

FINDINGS OF ENGINEERING SIGNIFICANCE BASED ON AVAILABLE GEOLOGIC INFORMATION

1. The volcanic rock surrounding Teton Dam is moderately to intensely jointed and, consequently, is permeable. At the damsite a blocky and slabby structure is displayed on the right abutment between El. 5185 and the canyon rim. A severely jointed zone is located above El. 5280. Both flat-lying and steeply dipping open joints are prevalent above 5185. Groundwater was therefore free to move with almost equal ease in most directions within the upper two zones, except locally where the joints had been effectively grouted. On reservoir filling, water moved rapidly to the foundation of the right end of the dam, as indicated by the observation well hydrographs of Fig. 5-7. Open joints also existed in the upstream and downstream faces of the right abutment keyway trench, providing potential conduits for access or egress of water. See maps, geologic sections, and photos, Appendix E.
2. The rock beyond the right end of the dam is jointed and permeable. The log and water testing at drill hole 505 (drilled after emplacement of the grout curtain) suggest exceptionally high permeability above El. 5250. (Table 5-4 and Fig. 5-5.)
3. Pump-in tests at drill hole 303 established the existence of interconnecting open joints between drill holes 5, 6, and 204; in other words, underground conduits exist on the downstream side of the dam through which water could travel from the right end of the dam to the canyon wall. These holes are all located either downstream or beyond the end of the grout curtain; therefore, it is doubtful that grouting significantly affected the carrying capacity of these joint paths. On the basis of these observations, it appears that no natural watertight barrier existed at the end of the dam and that it was possible for some water to follow the shortest path or paths around the end of the grout curtain and re-enter the canyon downstream. However, the maximum elevation of groundwater at DH-5, the only observation well near the downstream side of the dam, approached 5200 ft, which is approximately the level of the leak observed on the morning of the failure. Thus the available evidence argues that there was insufficient hydraulic gradient between DH-5 and the canyon wall to provide the high velocity underflow leading to the breakout that was observed at El. 5200 on the downstream side of the dam.
4. The lake and stream deposits beneath the right wall of the canyon contain permeable members which may connect hydraulically with the reservoir and with drill hole 506 beyond the right end of the dam. Whether such members extend beneath the dam and interconnect with vertical joints or fissures that extended upward to the base of the embankment is not known. However, the lake and stream deposits beneath the Rexburg Bench are generally much less permeable than the overlying volcanic rocks. Although some leakage under the dam may have occurred through these sediments, flow through joints in the welded tuff was more likely a significant factor in the failure process.
5. A comparison of geodetic surveys completed before and after failure of the dam indicates that no significant deformation of the dam foundation or vicinity occurred since construction of the project was undertaken.

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CHAPTER 6
SEISMICITY
(Panel Charge No. 2)

HISTORICAL

In general terms the region of Teton Dam is included in Zone 3 on the Seismic Risk Map of the United States, Fig. 6-1 (Algermissen, 1969). This is the zone of highest risk. Furthermore, the dam lies from 35 to 50 miles west of the great Intermountain seismic belt, a complex system of faults aligned in a north-south direction as shown on Fig. 6-2, numbers 1 to 13 inclusive. The general seismicity of the region is also indicated by the clusters of epicenters, as shown on Fig. 6-3 in the Yellowstone Park and the Jackson Hole area to the south of Yellowstone Park.

However, it should be emphasized that the highly seismic areas mentioned above almost surround the long aseismic protrusion to the northeast of the Snake River plain; this is best seen on Fig. 6-2. It is in this relatively seismically quiet platform of Cenozoic volcanic and sedimentary rocks that Teton Dam was built.

Prior to 1932 earthquake data are based almost exclusively on felt and damage reports. Furthermore from 1932 to 1961 no seismograph stations were operating in Idaho. Pre-instrumental epicentral locations may have errors of 10 to 20 miles. Since 1961 many seismometers have been installed and these have been especially valuable in identifying the location and magnitude of the smaller seismic events. In the period 1969-74, five earthquakes were located within 30 miles of the damsite and two of these had magnitudes greater than 3.

GEOLOGIC SETTING

The geologic history has been described in the previous chapter. It will suffice here to restate a few comments on the regional geologic setting.

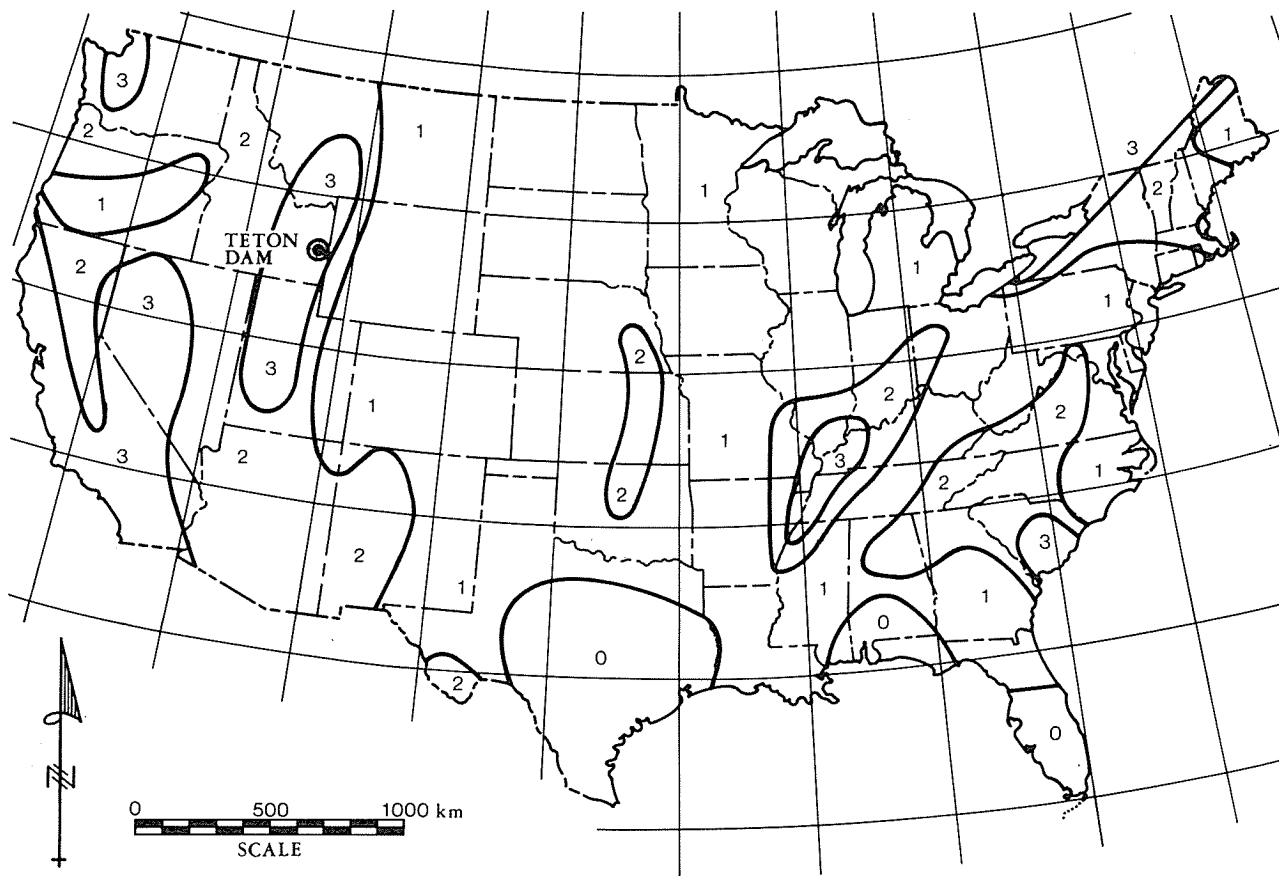
The eastern Snake River plain is a northeast trending depression formed in late Cenozoic time with concurrent volcanism. Rhyolitic ash-flow tuffs lie beneath the younger basalts which make up much of the surface of the Snake River plain. At Teton Dam the two-million-year-old ash-flow tuffs are several hundred feet thick.

Teton canyon is incised in this thick rock unit as a result of local uplift across the path of the river. Remnants of younger basalts are found in places along the canyon bottom. Downwarping, and possibly adjustments by faulting, continued until late Cenozoic time and undoubtedly the broad, regional picture is one of structural deformation continuing up until the present.

FAULTING

Fig. 6-2 shows the distribution of the major active faults, particularly the Madison (3), Hebgen (4), Yellowstone (5) and the Grand Valley fault zone (7). Teton Dam is located approximately 25 miles south of the Island Park caldera (6).

The largest historic earthquake in the Intermountain seismic belt was in 1959 at Hebgen. The ground rupture of 15 miles with a 20-ft normal displacement produced a 7.1 magnitude earthquake



SEISMIC RISK MAP OF THE UNITED STATES

ZONE 0—NO DAMAGE

ZONE 1—MINOR DAMAGE: DISTANT EARTHQUAKES MAY CAUSE DAMAGE TO STRUCTURES WITH FUNDAMENTAL PERIODS GREATER THAN 1.0 SECONDS; CORRESPONDS TO INTENSITIES V AND VI OF THE M.M.* SCALE.

ZONE 2—MODERATE DAMAGE; CORRESPONDS TO INTENSITY VII OF THE M.M.* SCALE.

ZONE 3—MAJOR DAMAGE; CORRESPONDS TO INTENSITY VIII AND HIGHER OF THE M.M.* SCALE.

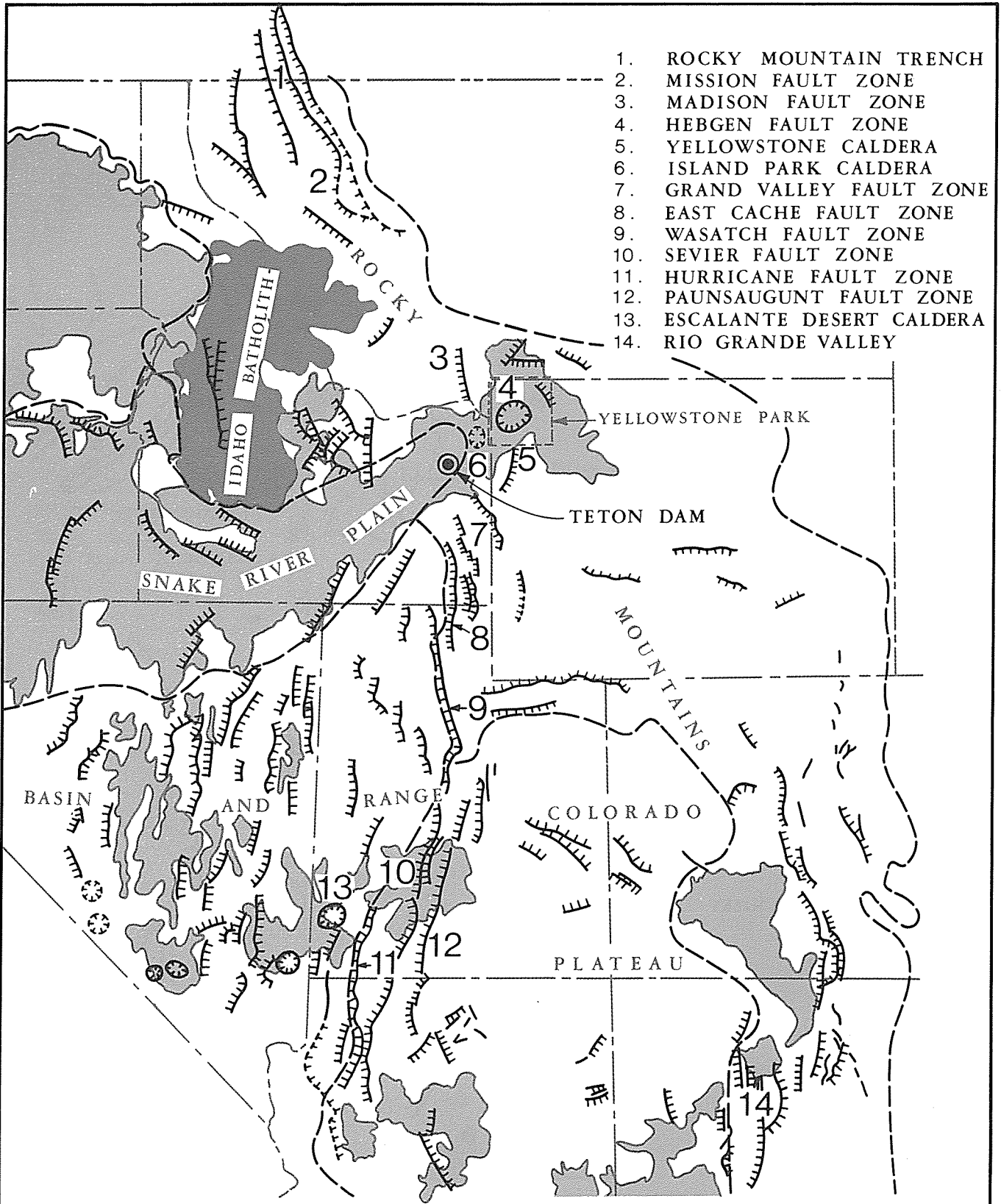
This map is based on the known distribution of damaging earthquakes and the M.M.* intensities associated with these earthquakes; evidence of strain release; and consideration of major geologic structures and provinces believed to be associated with earthquake activity. The probable frequency of occurrence of damaging earthquakes in each zone was not considered in assigning ratings to the various zones. See accompanying text for discussion of frequency of earthquake occurrence.

*Modified Mercalli Intensity Scale of 1931.

SEISMIC RISK MAP
OF THE UNITED STATES

FIG. 6-1.

U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

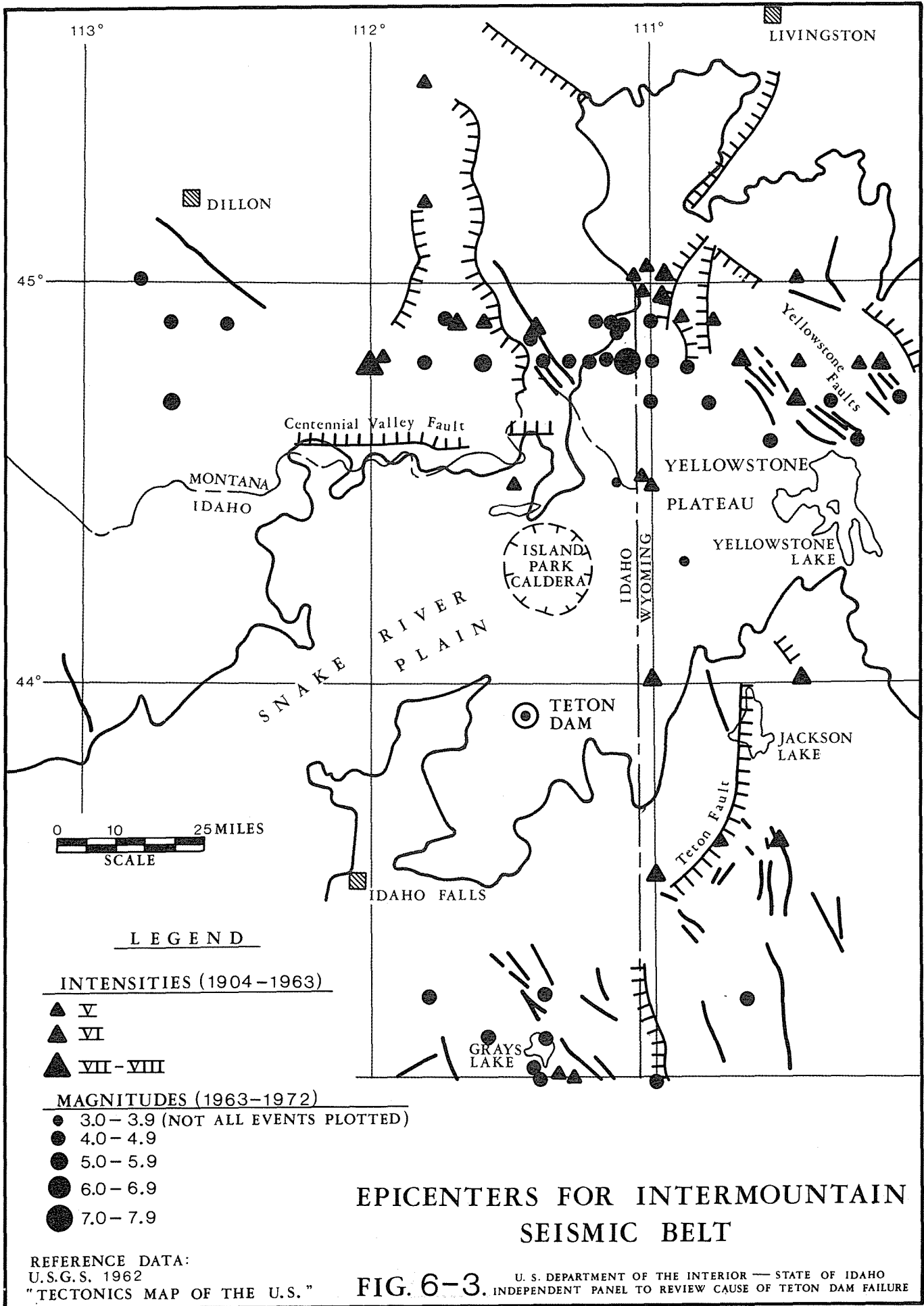


1. ROCKY MOUNTAIN TRENCH
2. MISSION FAULT ZONE
3. MADISON FAULT ZONE
4. HEBGEN FAULT ZONE
5. YELLOWSTONE CALDERA
6. ISLAND PARK CALDERA
7. GRAND VALLEY FAULT ZONE
8. EAST CACHE FAULT ZONE
9. WASATCH FAULT ZONE
10. SEVIER FAULT ZONE
11. HURRICANE FAULT ZONE
12. PAUNSAUGUNT FAULT ZONE
13. ESCALANTE DESERT CALDERA
14. RIO GRANDE VALLEY

REFERENCE DATA:
 KING'S TECTONIC MAP OF NORTH
 AMERICA (1969),
 U. S. GEOLOGIC SURVEY
 "TECTONIC MAP OF THE
 UNITED STATES" (1962)

GENERALIZED LATE MESOZOIC -
 CENOZOIC TECTONIC MAP OF
 INTERMOUNTAIN WEST

FIG. 6-2. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
 INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



approximately 60 miles from Teton Dam. In 1964 a 5.8 magnitude earthquake was recorded in Madison Valley 25 miles north of Hebgen.

Other large earthquakes in the belt have magnitudes ranging from 6.5 to 7. However, except for the Hebgen event, no earthquake within 50 miles of Idaho, or in Idaho, had a magnitude above 6.5. The nearest major fault to the dam is at Jackson Lake 35 miles to the east.

Fig. 5-1 in the previous chapter indicates a fault passing close to the dam. On the basis of low swales or flexures in the surface aeolian sediments it has been interpreted as passing between core holes DH-5 and DH-6. However, Bureau of Reclamation personnel (Klein and Boch, 1973) suggested that neither of these deep, inclined holes showed evidence of faulting.

Another fault, with a northeasterly inferred extension, has been mapped at White Owl Butte ten miles south of Teton Dam. It is doubtful if this inferred extension passes within several miles of the dam. Nevertheless, Greensfelder, in a recent seismicity study of Idaho, has suggested that the maximum acceleration at the damsite may reach a value as high as 0.25g.

SEISMOMETER ARRAY

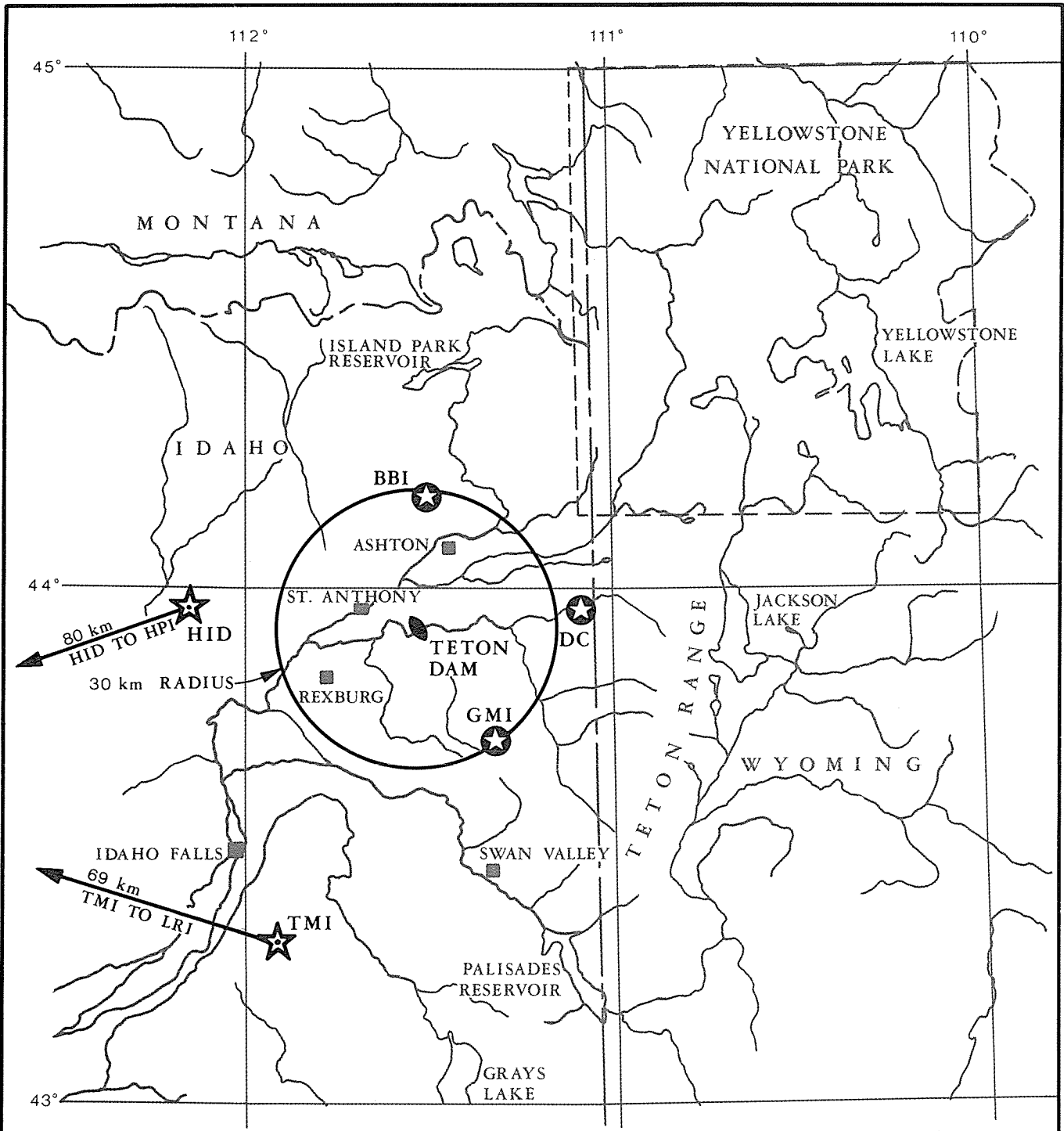
To observe effects of water impounded by the dam, plus the weight of the dam, the U.S. Geological Survey installed three stations: Big Bend, Dry Creek and Garns Mountain. Their locations are shown on Fig. 6-4 under the code designations BBI, DCI and GMI.



The Energy Research and Development Administration (ERDA) and the Idaho National Engineering Laboratory (INEL) had stations at Hamer (HID) and Taylor Mountain (TMI), localities indicated on Fig. 6-4.

An additional station was installed at the Teton Dam project office and a strong-motion accelerometer was placed in the Teton Dam powerhouse.

The results of the monitoring study by the U.S. Geological Survey can be summarized as follows:

1. The most sensitive Teton Dam seismic monitoring station has a magnitude threshold of $-0.1 M_L$ in the immediate vicinity of the dam.
2. No seismic events other than identified blasts were observed within a 30-km radius of the Teton Dam site during the period April 1, 1976 - June 9, 1976.
3. The closest and largest earthquake during this same period was located southwest of Victor, Idaho, at a distance of 60 km from the dam, and had a magnitude of $1.7 M_L$.
4. No seismic events of magnitude $2.2 M_L$ or greater were observed within a 30-km radius of the dam during the period June 16, 1974 - March 31, 1976. All of the events within 20 km of the dam have been confirmed as blasts.
5. No increase in seismicity around the dam was observed as the reservoir was filling.
6. For at least four hours, the seismic monitoring network recorded ground motion generated by the release of the flood waters and the breakup of the dam.
7. The strong-motion instrument at the base of the powerhouse has not been recovered.



-  TETON DAM PROJECT SEISMIC STATIONS
-  INEL SEISMIC STATIONS

LOCATION OF SEISMIC STATIONS

REFERENCE DATA:
U. S. GEOLOGIC SURVEY

FIG. 6-4. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

RECORDINGS MONITORING DAM FAILURE

Ground motions induced by flooding water during the failure of Teton Dam were recorded at all seven stations in the U.S. Geological Survey array including the most distant, 78 miles from the site.

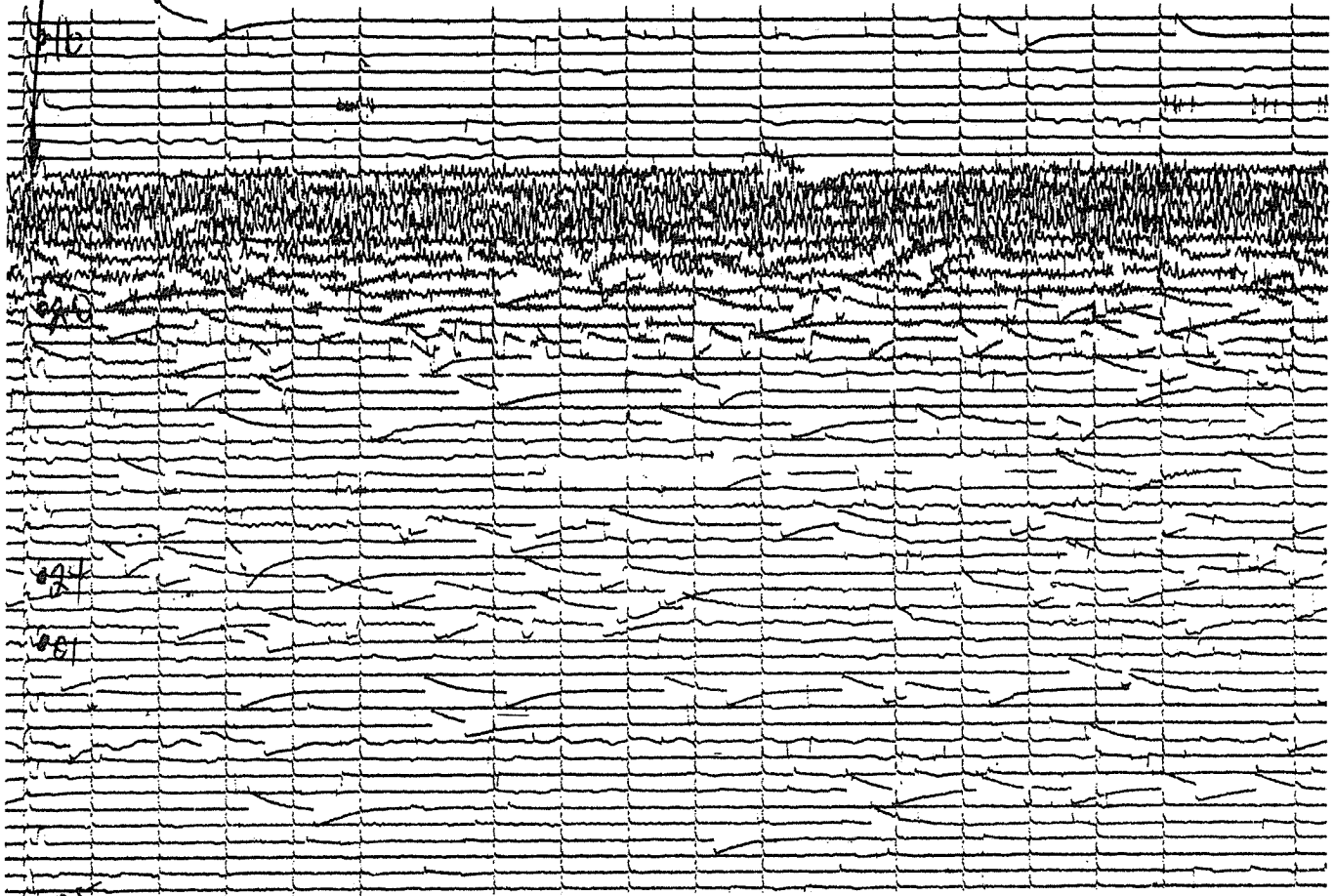
The initial recording began at 11:47 a.m. and vibrations continued for four hours. Fig. 6-5 is a portion of the seismogram from station GMI showing ground motion induced by the flood waters.

COMMENTS

Although the Teton Dam was constructed in a general region of high seismicity and a localized region of moderate seismicity, there were no seismic events triggered by filling the reservoir and there is no reason to believe that earthquakes were in any way responsible for the failure.

GROUND MOTION DUE TO
FLOODING COMMENCED
APPROXIMATELY 11:47AM JUNE 5, 1976

1 MINUTE



SEVEN-IN. LONG SEGMENT FROM 30 IN. CIRCUMFERENCE CONTINUOUS RECORD.
SUCCESSIVE HORIZONTAL LINES REPRESENT ONE COMPLETE REVOLUTION OF THE
SEISMOGRAPH DRUM, AN INTERVAL OF 15 MIN. AS REPORTED TO THE PANEL, BY A
REPRESENTATIVE OF THE USGS, THE GROUND MOTION RECORDED HERE IS
DISTINCT FROM THAT OF AN EARTHQUAKE.

PORTION OF SEISMOGRAM FROM STATION GMI

REFERENCE DATA:
U. S. GEOLOGIC SURVEY

FIG. 6-5.

U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

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Smith, Robert B. and Sbar, Marc L. (1974) "Contemporary Tectonics and Seismicity of the Western U.S. with Emphasis on the Intermountain Seismic Belt," Bull. Geol. Soc. of America, vol. 85, no. 8, pp. 1205-1218.

CHAPTER 7
CONSTRUCTION MATERIALS
(Panel Charge No. 4)

A general description of the Teton Dam embankment is included in Chapter 1. (Figs. 1-2 and 1-3.) The canyon width at river level is about 750 ft (El. 5050 ±) and at the canyon top the width is about 1,700 ft (El. 5320+). The abutments rise steeply 280± ft above the canyon floor.

The dam embankment consisted of five zones of the following approximate volumes:

Zone 1	5,186,327 cu yds
Zone 2	2,393,364 cu yds
Zone 3	906,560 cu yds
Zone 4	591,275 cu yds
Zone 5	793,675 cu yds

The upstream slope above El. 5185 was protected with a 3-ft thickness of basalt rock riprap measured normal to the surface.

BORROW AREAS

General.

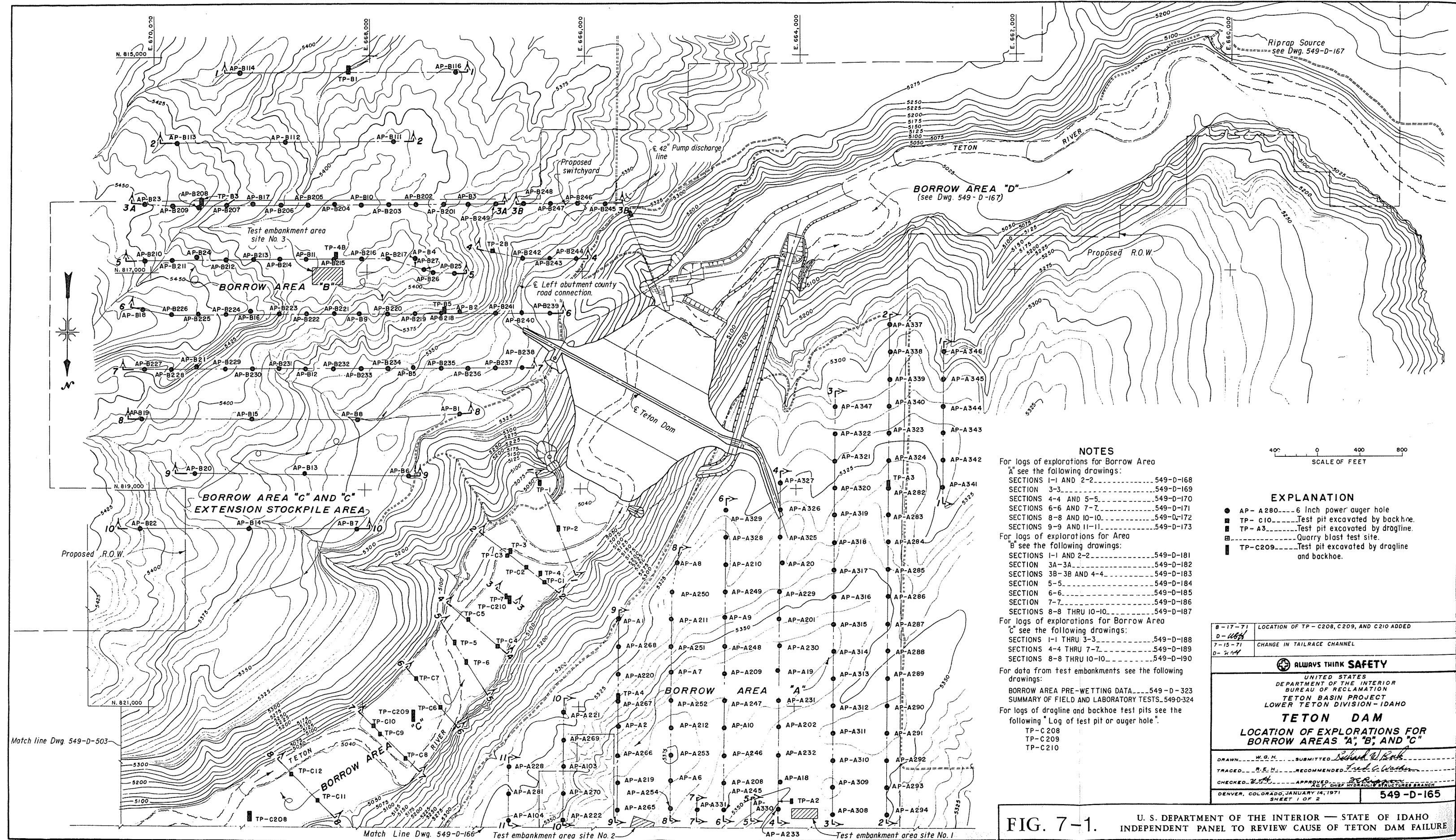
Embankment materials utilized at the Teton site consisted of the ML aeolian silts covering the uplands in Borrow Area "A" just north of the right abutment; GP and GW river-deposited gravel in the bottom of the Teton River Canyon immediately upstream of the Dam in Borrow Areas "C" and "C" Extension; and basalt from quarries developed about 3-1/2 miles north of the site. The borrow areas are shown in Figs. 7-1, 7-2, and 7-3. In addition maximum utilization was made of the material from the required excavations for embankment and structure foundations, the two outlet tunnels, the powerhouse, the tailrace channel, and the borrow pits.

Required Excavation.

The surface of the foundations occupied by the embankment and appurtenant structures was stripped of soil containing root concentrations, organic materials, and any unstable silts and clay. The stripped material was wasted in the disposal areas immediately upstream and downstream of the dam. The materials from foundation and structural excavation below the stripping were selectively placed in appropriate zones of the dam. River sands and gravels were placed in Zones 2 and 4. Loose rock and rock excavation were placed in Zone 5. Mixtures of clay, silt, sand, and rock fragments were placed in Zone 3. The construction separately of portions of Zones 2, 3, 4, and 5 was permitted where possible to minimize the stockpiling of required excavation.

Borrow Area "A".

Materials for Zone 1 and Zone 3 that were not available from required excavation were borrowed from Borrow Area "A". This area contained substantial deposits of caliche (a soil inclusion formed by cementation of material by calcium carbonate) and relatively hard cemented layers of overburden. Close control and selective excavation were required to avoid extensive deposits of caliche and cemented soils, as well as deposits of MH and CH lower-density soils. Occasionally, shallow layers of caliche and soils easy or moderately hard to excavate were blended in cut. Some relatively thick layers of caliche and cemented soils that would not break down readily under rolling had to be



NOTES

For logs of explorations for Borrow Area "A" see the following drawings:
 SECTIONS 1-1 AND 2-2..... 549-D-168
 SECTION 3-3..... 549-D-169
 SECTIONS 4-4 AND 5-5..... 549-D-170
 SECTIONS 6-6 AND 7-7..... 549-D-171
 SECTIONS 8-8 AND 10-10..... 549-D-172
 SECTIONS 9-9 AND 11-11..... 549-D-173

For logs of explorations for Area "B" see the following drawings:
 SECTIONS 1-1 AND 2-2..... 549-D-181
 SECTION 3A-3A..... 549-D-182
 SECTIONS 3B-3B AND 4-4..... 549-D-183
 SECTION 5-5..... 549-D-184
 SECTION 6-6..... 549-D-185
 SECTION 7-7..... 549-D-186
 SECTIONS 8-8 THRU 10-10..... 549-D-187

For logs of explorations for Borrow Area "C" see the following drawings:
 SECTIONS 1-1 THRU 3-3..... 549-D-188
 SECTIONS 4-4 THRU 7-7..... 549-D-189
 SECTIONS 8-8 THRU 10-10..... 549-D-190

For data from test embankments see the following drawings:
 BORROW AREA PRE-WETTING DATA..... 549-D-323
 SUMMARY OF FIELD AND LABORATORY TESTS..... 549-D-324

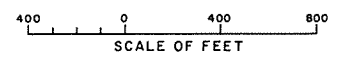
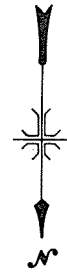
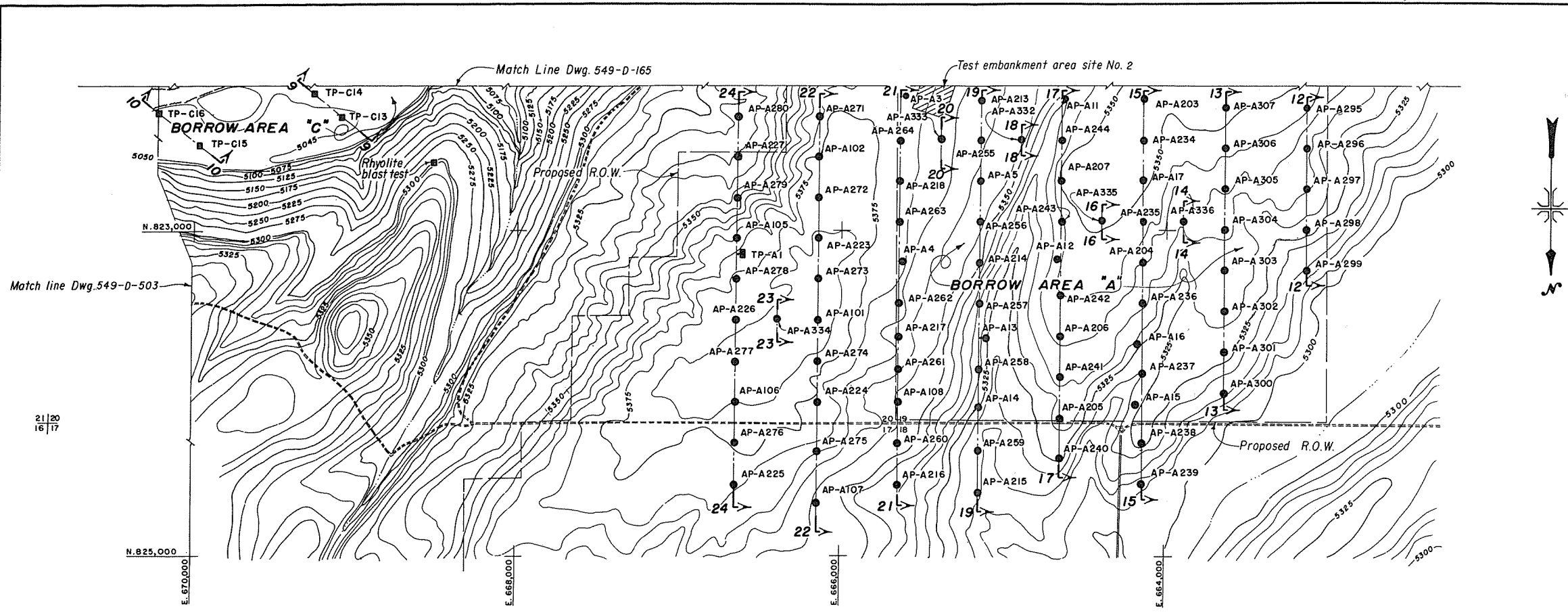
For logs of dragline and backhoe test pits see the following "Log of test pit or auger hole".
 TP-C 208
 TP-C 209
 TP-C 210

EXPLANATION

- AP-A 280..... 6 inch power auger hole
- TP-C 10..... Test pit excavated by backhoe.
- TP-A 3..... Test pit excavated by dragline.
- TP-C 209..... Test pit excavated by dragline and backhoe.

B-17-71	LOCATION OF TP-C 208, C 209, AND C 210 ADDED
D-11-71	
7-15-71	CHANGE IN TAILRACE CHANNEL
D-24-71	
ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION TETON BASIN PROJECT LOWER TETON DIVISION-IDAHO	
TETON DAM LOCATION OF EXPLORATIONS FOR BORROW AREAS "A", "B", AND "C"	
DRAWN..... W.G.H. SUBMITTED <i>Richard D. Park</i>	
TRACED..... R.E.H. RECOMMENDED <i>Fred C. Wain</i>	
CHECKED..... <i>[Signature]</i> APPROVED..... <i>[Signature]</i> AGT, CHIEF HYDRAULIC STRUCTURES BRANCH	
DENVER, COLORADO, JANUARY 14, 1971	
549-D-165 SHEET 1 OF 2	

FIG. 7-1. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
 INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



- EXPLANATION**
- AP-A300.....6 Inch power auger hole
 - TP-C13.....Test pit excavated by backhoe
 - TP-A1.....Test pit excavated by dragline
 -Quarry blast test site

- NOTES**
- For logs of explorations for Borrow Area "A" see the following drawings:
- SECTIONS 12-12 THRU 14-14.....549-D-174
 - SECTIONS 15-15 AND 16-16.....549-D-175
 - SECTIONS 17-17 AND 18-18.....549-D-176
 - SECTIONS 19-19 AND 20-20.....549-D-177
 - SECTION 21-21.....549-D-178
 - SECTIONS 22-22 AND 23-23.....549-D-179
 - SECTION 24-24.....549-D-180
- For logs of explorations for Borrow Area "C" see the following drawings:
- SECTIONS 8-8 THRU 10-10.....549-D-190

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UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TETON BASIN PROJECT
LOWER TETON DIVISION - IDAHO

TETON DAM
LOCATION OF EXPLORATIONS FOR
BORROW AREAS "A", "B", AND "C"

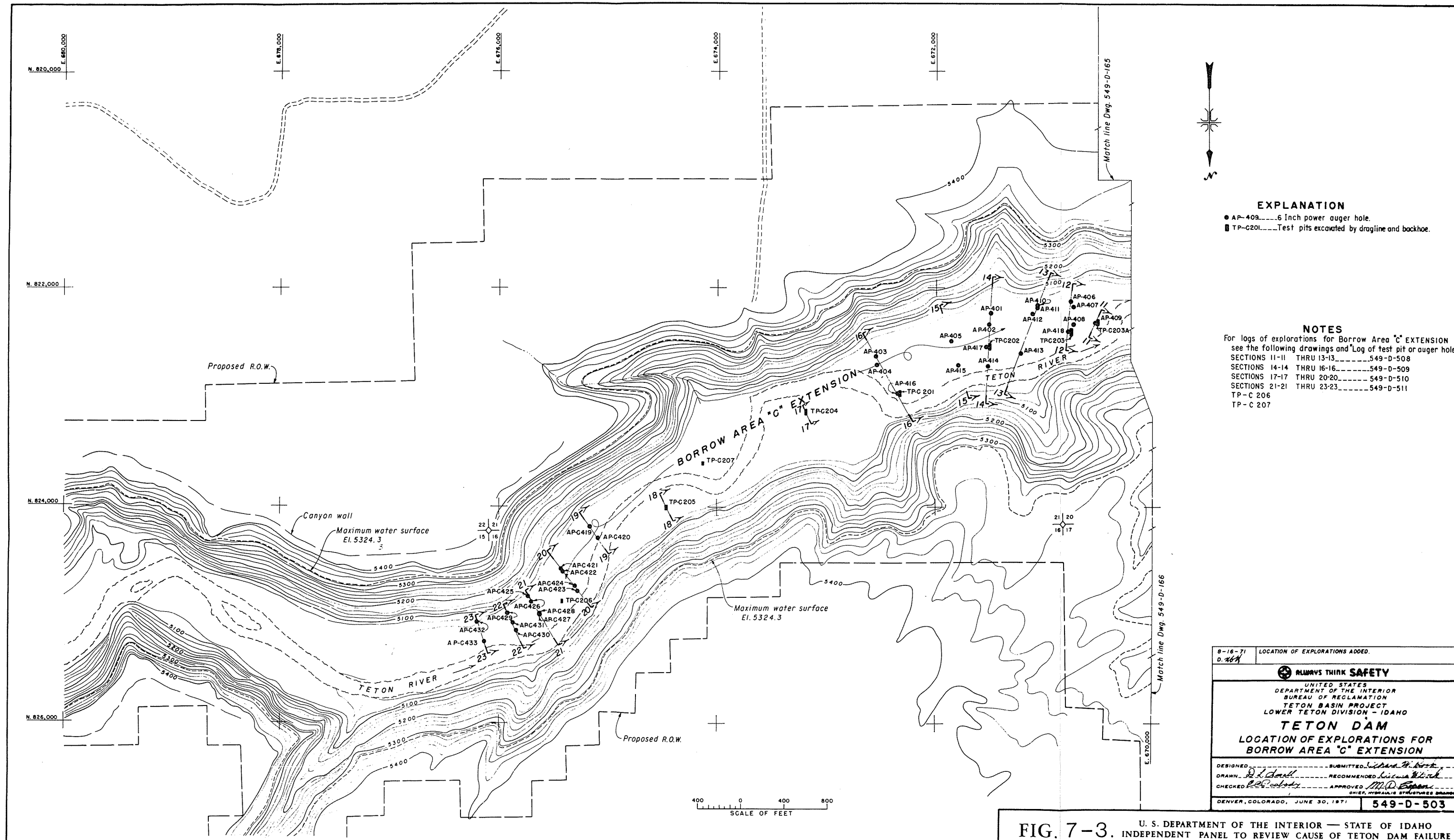
DRAWN: W.G.H. SUBMITTED: *Richard W. Beck*

TRACED: R.E.H., P.M.K. RECOMMENDED: *L.C. W. H.*

CHECKED: *W.H.* APPROVED: *D.E. Rogers*
ASST. CHIEF, HYDRAULIC STRUCTURES BRANCH

DENVER, COLORADO, JANUARY 14, 1971 SHEET 2 OF 2 **549-D-166**

FIG. 7-2. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



EXPLANATION

- AP-409.....6 Inch power auger hole.
- TP-C201.....Test pits excavated by dragline and backhoe.

NOTES

For logs of explorations for Borrow Area "C" EXTENSION see the following drawings and Log of test pit or auger hole.

- SECTIONS 11-11 THRU 13-13.....549-D-508
- SECTIONS 14-14 THRU 16-16.....549-D-509
- SECTIONS 17-17 THRU 20-20.....549-D-510
- SECTIONS 21-21 THRU 23-23.....549-D-511
- TP - C 206
- TP - C 207

8-16-71 D. 167	LOCATION OF EXPLORATIONS ADDED.
ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION TETON BASIN PROJECT LOWER TETON DIVISION - IDAHO TETON DAM LOCATION OF EXPLORATIONS FOR BORROW AREA "C" EXTENSION	
DESIGNED: <i>D. J. Small</i>	SUBMITTED: <i>W. A. H. Burt</i>
DRAWN: <i>D. J. Small</i>	RECOMMENDED: <i>W. A. H. Burt</i>
CHECKED: <i>D. J. Small</i>	APPROVED: <i>M. D. Egan</i>
CHIEF, HYDRAULIC STRUCTURES BRANCH	
DENVER, COLORADO, JUNE 30, 1971	549-D-503

FIG. 7-3. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

excavated to uncover the required quantity of suitable underlying Zone 1 type materials. Such required excavations and those that were unsuitable for Zone 1 by reason of excess quantities of caliche or hardpan were used in Zone 3.

Borrow Areas "C" and "C" Extension.

Materials for embankment Zones 2 and 4 that were unavailable from required excavation were borrowed from Areas "C" and "C" Extension. Seventeen percent of the necessary material was obtained from required excavation. Zone 4 was designed to facilitate the construction of the diversion cofferdam. After diversion of the river, Borrow Areas "C" and "C" Extension were subject to flooding during spring runoff. To assure an adequate supply of Zone 2 material when needed, all but 400,000 cu yds of the required Zone 2 quantity was stockpiled above the canyon wall upstream of the left abutment. As construction progressed, the volume of Zone 2 anticipated from required excavation and Zone 2 shrinkage factors were continuously monitored. By varying slightly the boundary between Zone 1 and Zone 2, the embankment was completed with the material available in the stockpile.

MATERIALS AS PLACED

Zone 1.

Zone 1 is the impervious central core obtained from Borrow Area "A" located on the north side of the river canyon near the right end of the dam.

Material in Borrow Area "A" was about 5 to 6 percent dry of the specified moisture content. Water was added by either ponding or by ripping and sprinkling. Material was excavated either with a wheel excavator in an 8-ft cut, or by scrapers. It was compacted by 12 passes of the USBR standard sheepsfoot roller. Zone 1 was placed in 6-in. compacted lifts with moisture and density control tests determined by the rapid compaction control method (Earth Manual, 1974 — Designation E-25).

The earthwork construction control records show the following average values for Zone 1 material for 2,608 tests:

Ratio of fill dry density to maximum USBR dry density	= 98.2 percent
Optimum moisture minus field moisture	= 1.3 percent dry
Fill dry density, minus No. 4	= 99.5 pcf
Fill moisture content, minus No. 4	= 18.6 percent

Zone 2.

Zone 2 is the inner downstream shell and the upstream shell of the embankment and also forms the drainage blanket beneath Zone 3 and between Zone 3 and the abutment foundation surfaces. It consists of a mixture of fines, sand, gravel and cobbles obtained from required excavations and Borrow Area "C" and "C" Extension in the riverbed deposits upstream from the dam. Zone 2 was compacted in 12-in. lifts by either crawler-type tractors or vibratory compactors. Density requirements were controlled by the Relative Density Test (Earth Manual Designation E-12). The average relative density for 176 construction control tests was 94 percent.

Permeability tests were made during construction on 18 laboratory-compacted specimens. As reported by USBR, the coefficient of permeability ranged from 0.7×10^{-6} cm/sec to 39.3×10^{-6} cm/sec and averaged 9.4×10^{-6} cm/sec for 16 of the specimens. Two specimens tested at extremes of 2980 and 1784×10^{-6} cm/sec.

Coarse and fine fractions, as determined by record tests during construction were:

Fraction	Average	Range
Plus 3-in.	10 percent	0 to 30 percent
Plus No. 4	66 percent	45 to 76 percent
Minus 200	4.5 percent	2 to 12 percent

Zone 3.

Zone 3 is an intermediate downstream shell utilizing material from Borrow Area "A", unsuitable for Zone 1, and Zone 1-, 2-, or 4-type materials. Zone 3 material was compacted in both 12-in. lifts with six passes of a 50-ton pneumatic-tired roller and in 6-in. lifts with 12 passes of a 4000-lb/ft sheepsfoot roller. Construction control methods used were the same as for Zone 1.

The earthwork construction control records show the following average values for 118 tests:

Ratio of fill dry density to maximum USBR dry density	= 97.4 percent
Optimum moisture minus fill moisture Plus No. 4	= 1.5 percent dry = 1.6 percent
Fill dry density, minus No. 4	= 97.5 pcf
Fill moisture content, minus No. 4	= 18.4 percent

Zone 4.

Zone 4 is the toe segment of the upstream shell utilizing the semipervious silty sands and gravels from the required excavations of the cutoff trench and Borrow Area "C" and "C" Extension, and also formed part of the cofferdam for river diversion. Downstream Zone 4 is the berm at the downstream toe of the dam utilized for storage areas near the power and pumping plant. Zone 2 type material from Borrow Area "A" was also placed in Zone 4.

Zone 4 material was compacted in 12-in. lifts with four passes of a 40,000-pound crawler-type tractor. Density requirements were controlled by the Relative Density Method (Earth Manual Designation E-12). The average relative density for 94 construction control tests was 98 percent.

Coarse and fine fractions, as determined by record tests during construction, were:

Fraction	Average	Range
Plus 3-in.	8 percent	0 to 31 percent
Plus No. 4	61 percent	12 to 75 percent
Minus No. 200	6 percent	2 to 15 percent

Zone 5.

Zone 5 is the outer layer of the upstream and downstream shells, utilizing cobbles, boulders and rock fragments from the required excavations of the cutoff trench, the foundation key trench, abutment cleanup, river outlet works, auxiliary outlet works, spillway, and Borrow Area "C" and "C" Extension. Zone 5 material was placed in 3-ft lifts and compacted by travel of the hauling and placement equipment.

PROJECT MATERIALS TESTING PROGRAM

Soil Samples.

Seventy-two soil samples were acquired by the Earth Sciences Branch, Denver Laboratory, during the project design period. The first 45 samples (51B-1 through 51B-45, Denver Laboratory index number) were acquired in May 1969. Thirty-nine were disturbed fine-grained materials from three prospective impervious borrow areas and six were gravel and sand from two prospective pervious borrow areas. Two samples (51B-X46 and 51B-X47) were composited of materials from representative test pits in Borrow Area "A" (TP-A2) and Borrow Area "B" (TP-B2).

Undisturbed 6-in.-diam Denison-type samples were obtained from the embankment during construction at approximately 10-ft vertical intervals while drilling hole DH-DNGP-1 for geophysical testing of the embankment and represent the interval between El. 5133.5 and El. 4940.5. The hole was located 100 ft upstream of Sta. 20+00. Twenty samples from that hole (51B-48 through 51B-67) were sent to the Denver Laboratory for both static and dynamic soil analyses. One composite sample (51BX68), representing the full length of the hole, was prepared for the various soil tests scheduled.

Four hand-cut undisturbed samples (51B-70 through 51B-73) were obtained from the cutoff trench during construction at the following locations:

Embankment ∇ Station	Elevation
19+00.7	4944.6
19+01.2	4945.9
19+04.7	4949.3
20+51.3	4944.0

Laboratory Tests on Borrow Materials.

Gradation and Atterberg limits were determined for each sample from Borrow Areas "A" and "B" and specific gravity and moisture-density relationships were determined for four selected samples, two from each borrow area. All samples from TP-A2, Borrow Area "A", were composited to form one sample (51B-X46). All samples from TP-B2, Borrow Area "B", were composited to form one sample (51B-X47). The composite samples were tested for gradation, Atterberg limits, specific gravity, moisture-density relationships, permeability, one-dimensional consolidation, triaxial shear strength, the modulus of deformation, and Poisson's ratio.

Sample No. 51B-X46, which classified as ML material, consolidated about 6 percent under a 300 psi loading, developed shear-strength parameters of $\tan \phi' = 0.64$ and $c' = 11.3$ psi under lateral confining pressures of 25, 50, and 100 psi. Permeability was 0.32 ft/yr under a 100 psi loading.

Sample No. 51B-X47, which classified as CL material, consolidated about 6.5 percent under a 300 psi loading and developed shear-strength parameters of $\tan \phi' = 0.58$ and $c' = 11.6$ psi under lateral confining pressures of 25, 50, and 100 psi. Permeability was 0.18 ft/yr under a 100 psi loading. All tests were made on specimens compacted near maximum dry density (12,500 ft-lbs/cu ft compactive effort) and at either 2 percent dry of optimum moisture content or at optimum moisture content.

The materials from borrow areas "C" and "D" are streambed gravels and sands. Two samples (51B-38 and 51B-39) from Borrow Area "C" were tested for gradation and relative density.

Laboratory Tests on Embankment Samples Obtained During Construction.

Gradation, Atterberg limits, field moisture, and density were determined for the 6-inch undisturbed samples obtained from Zone 1 in Drill Hole DH-DNGP-1. These materials classified as silt or silty clay

(ML or CL-ML). Sample 51B-X58 was composited and remolded using approximately one-half of each undisturbed sample, and gradation, Atterberg limits, specific gravity, moisture-density relationship, one-dimensional consolidation, and triaxial shear strength were determined. Specimens compacted to about 98 percent of USBR maximum dry density at optimum water content for one-dimensional consolidation and triaxial shear testing consolidated about 7 percent under a 300 psi loading and developed shear-strength parameters of $\tan \phi' = 0.70$ and $c' = 0.02$ psi under lateral confining pressures of 15, 30, 75, and 100 psi. All specimens were back-pressured for complete saturation and then sheared under consolidated-drained conditions.

The USBR maximum dry density was 102.9 pcf at an optimum moisture content of 18.2 percent. The average dry density and moisture content of all the undisturbed embankment samples were 105.1 pcf and 21.2 percent, respectively.

Laboratory tests to determine soil behavior under dynamic loadings scheduled for sample No. 51B-68 have not been made.

Samples 51B-70 through 51B-73 were tested for horizontal permeability in their undisturbed state. All were classified as silt or silty clay. Dry density varied from 85.7 to 89.4 pcf. The maximum horizontal permeability was 18.1 ft/yr under a hydraulic gradient of 43. Permeability decreased at higher gradients.

Summary of Laboratory Test Data for Zones 1 and 2.

A summary of permeability tests for Zone 1 materials is shown in Table 7-1 and a summary of triaxial shear tests on Zone 1 materials is provided in Table 7-2.

Riprap.

The quarry sites finally selected and used for riprap were known as Hobbs No. 2 and Hobbs No. 2 extension, located 3-1/2 miles north of the damsite. The rock, which is a massive vesicular to dense basalt, was exposed in bold outcrops with large talus blocks also present. The specific gravity ranged from 2.27 to 2.88. The loss by Los Angeles abrasion test was 7 percent at 100 revolutions and 31 percent at 500 revolutions.

Concrete Aggregate.

Sand and gravel for concrete aggregate were obtained from Falls River streambed deposits about 6 miles north of the damsite. Aggregate was processed at the pits into 3 in., 1-1/2 in., 3/4 in. and sand fractions. The gravel was subrounded with about 13 percent subangular and 4 percent flat particles. It was composed mainly of basalt, quartzite, and glassy volcanics with lesser amounts of schist, chert, and obsidian. The loss by Los Angeles abrasion test was 5 percent at 100 revolutions and 26 percent at 500 revolutions.

The sand was subangular to angular and contained the same rock types found in the gravel plus quartz, feldspar, amphibole, garnet, and magnetite. Generally, the fineness modulus was about 3.10. A considerable percentage of both the gravel and sand was alkali reactive.

PANEL'S INVESTIGATIVE ZONE 1 SOILS TESTING PROGRAM

As described in Chapter 3, undisturbed samples were obtained from Zone 1 during exploration of the right abutment remnant and tested for engineering properties in a number of laboratories in accordance with the schedule shown in Table 7-3. The locations from which the samples were obtained are shown in Fig. 3-5.

**TABLE 7-1
SUMMARY OF PERMEABILITY TESTS
FOR ZONE 1 MATERIALS FROM
BORROW AREA "A"**

Lab Sample Number	Field Designation	Depth Sampled (ft)	Smaller Than No. 200 Sieve (percent)	PI	Coefficient of Permeability (ft/yr)	USBR Exhibit (Appendix A)
Remolded* (Vertical)						
51B-X46	TP-A2	0-18.0	74	NP	0.32	1.3, Table 3
F-21	A-4	0-10.	68	3.1	0.37	1.3, Table 10
F-81	A-13	0-20.	68	10.0	0.01	1.3, Table 10

*USBR Designation E-13, 3" high sample in 8" permeability cylinder loaded to equivalent weight of fill.

Undisturbed* (Horizontal)						
51B-70	20+51.3	94	95	NP	12.3	24, Memo Chief, Earth Sciences Branch, 10/6/75
51B-71	19+00.7	93.4	93	6	13.0	
51B-72	19+04.7	88.7	88	NP	8.5	
51B-73	19+01.2	92.1	97	3	6.4	

*High-pressure permeability test apparatus under lateral pressure of 55 psi. In-place density, 85.7-89.4 pcf, Average for Zone 1, 100 pcf.

Remolded* (Vertical)						
146 Record Tests		See Exhibit			0.02- 3.57**	39, Earthwork Central Data

*USBR Designation E-13.

** Kav. = 0.47 ft/yr

All laboratories also made classification tests on the samples which they tested for other properties.

Classification Tests.

Table 7-4 shows the physical properties of representative samples taken from the Zone 1 material of the right abutment remnant.

Triaxial Compression Tests.

Five series of tests were made to determine the stress-strain characteristics of Zone 1 material for use in finite element stress analyses. These tests were made under consolidated-drained conditions at confining pressures of 15, 40, 70, and 100 psi on 1.4-in.-diam specimens, 3-1/2 in. long. Three series were made with the specimens at placement moisture content and two at saturated moisture content. One of the series at placement moisture and one at saturated moisture were conducted by stress-control techniques to investigate the creep characteristics under loads sustained for several days. Guided by the results of these tests and those of the Project Soils Testing Program, strength and stress-strain parameters were developed for use in the finite element analyses as discussed in Appendix D.

Permeability Tests.

Seven tests were made on specimens cut from undisturbed block samples for permeability in a horizontal direction. Three tests were made with the specimen saturated and four unsaturated. The overall average horizontal coefficient of permeability was 5×10^{-6} cm/sec.

Erosion and Dispersion Tests.

Although the highly erodible character of windblown silts of the type used in Zone 1 are well known to the engineering profession, the erodibility characteristics of these soils were determined by both quantitative and qualitative tests. The quantitative test procedures used were the flume test and the rotating cylinder test. The qualitative tests used were the crumb test, the dispersion ratio test and the pinhole test.

The flume and rotating cylinder tests which are specifically designed to test soil erodibility indicated clearly that the materials tested were highly erodible. The pinhole, crumb, and dispersion ratio tests, which are primarily designed to test the dispersive character of soils yielded mixed results. The results of the erosion and dispersion tests coupled with field observations of the material as it was excavated from the remnant on the right abutment, leave no doubt that the Zone 1 material was highly erodible.

Unconfined Compression Tests.

Unconfined compression tests were made on undisturbed specimens at placement moisture and on specimens compacted at various moisture contents to densities comparable with that of the undisturbed sample to assess the stress-strain relationships (brittleness) of Zone 1 material when placed at moisture contents both dry and wet of optimum. These tests confirm that Zone 1 material, when compacted dry of optimum, is brittle as evidenced by the shape of the stress-strain curves. The complete reports of tests are available in the Panel's records.

COMMENTS

Zone 1 – Core Material.

As has been described above, the Zone 1 material which formed the core comprised more than half of the volume of the dam. The material is a fine, wind-blown silt, primarily an ML material. As

TABLE 7-2
SUMMARY OF TRIAXIAL SHEAR TESTS
FOR ZONE 1 MATERIALS

Remolded*

Lab Sample Number	Field Designation	Depth Sampled (ft)	Smaller Than No. 200 Sieve (percent)	PI	Degree of Saturation (percent)	tan ϕ'	c' (psi)	K _o			USBR Exhibit (Appendix A)
								ϵ (in./in.)	E (psi)	μ	
51BX46	A-2	0-18	74	NP	70	0.64	11.3	0.005	5118	0.1306	1.3, Table 5 UU Test
								0.01	5676	0.1700	
								0.05	4856	0.2484	
								0.08	5367	0.2847	
51BX47	B-2	0-18	61	14	70	0.58	11.6	0.005	8081	0.047	1.3, Table 5 UU Test
								0.045	2529	0.1864	
								0.080	2663	0.2624	
								0.115	3287	0.2868	
51BX68	DH-DNGP-1	1.3-193	74	6	77	0.70	0.2			24, Table 3 2, pp 61, 62 El. 5133.5 Sta. 20+00 100 ft upstream CD Test	

*USBR Designation

2"x5" specimens

$$K_o = \frac{\sigma_3}{\sigma_1}$$

No tests were made on undisturbed samples.

**TABLE 7-3
POST-FAILURE TESTS PERFORMED FOR INDEPENDENT PANEL**

Laboratory	Samples Supplied	Tests Made
Northern Testing Laboratories Billings, Montana	5255D4-1	Drained triaxial shear at field moisture
	5250IR-U3-2	Drained triaxial shear at saturation moisture
	5270D3-1	Spare
Earth Sciences Branch USBR Denver, Colorado	5270U2-1	Drained triaxial shear at field moisture
	5755U4-1	Drained triaxial shear at field moisture
	5255D5-1	Drained triaxial shear at saturation moisture
	5270D4-2	Unconfined compression
	5270D5-1	Spare
	5270U5-1	Spare
Corps of Engineers Waterways Experiment Station Vicksburg, Mississippi	5240D5-1	Pinhole dispersion
	5255D4-2	Pinhole dispersion
	5270U3-1	Pinhole dispersion
	5250IR-D3-1	Pinhole dispersion
	5260IR-D2-1	Pinhole dispersion
University of California Davis, California	5240D5-2	Rotating cylinder erosion
	5255D4-3	Rotating cylinder erosion
	5270U3-2	Rotating cylinder erosion
	5250IR-D3-2	Rotating cylinder erosion
	5260IR-D2-2	Rotating cylinder erosion
	5240D5-3	Flume erosion
	5235IE-D4-1	Flume erosion
	5270U3-3	Flume erosion
	5255U3-1	Flume erosion
5255D3-1	Flume erosion	
University of California Berkeley, California	5270U5	Horizontal permeability
	5270D3-2	Horizontal permeability
Teton Project Laboratory Newdale, Idaho	All	Classification

TABLE 7-4
SUMMARY OF CLASSIFICATION TEST DATA
SAMPLES FROM REMNANT OF KEY-TRENCH FILL
RIGHT ABUTMENT

<u>Property</u>	<u>Mean</u>	<u>Approx. Std. Deviation</u>	<u>Usual Range</u>
Liquid Limit	26.4	0.8	23-31
Plasticity Index	3*	—	0-11
Water Content	22	5	14-32
% less than 200-mesh	80	6	55-95

No. of samples tested for each property was approximately 150

* 40% of samples non-plastic

Usual classification ML, occasional samples CL

compacted in the dam at less than optimum water content, it was very brittle and, because of its composition, extremely susceptible to erosion by flowing water. These two properties of the Zone 1 material, erodibility and brittleness, were prime factors which contributed to the failure of Teton Dam. The brittle nature of the Zone 1 material substantially increased the potential for cracking, and would sustain cracks once opened. The very high erodibility of the Zone 1 material permitted rapid piping of the material when subjected to flowing water through cracks in Zone 1 and along open joints at the foundation contact of Zone 1.

The strength of the Zone 1 material was a factor in the failure mechanism. The material, as compacted in the dam, permitted continuous erosion channels (pipes) to be formed in the core without any evidence of their existence becoming visible from the exterior of the dam. There is no doubt that channels had developed within the dam, in Zone 1 and possibly also in Zone 2, before the leaks were noticed at the downstream face of the dam on the morning of June 5th. The large volumes flowing from the leaks, the "loud burst" that was heard when the leak at El. 5200 broke out of the dam, and the observation of the 6-ft-diam tunnel in the dam reported by Mr. Robison, the Project Construction Engineer, all point to the probability that the Zone 1 material had been eroding for some time before the visible signs of failure appeared on June 5, and that the combination of erodibility and strength of this material had led to the formation of a major tunnel or pipe. Had the material been weaker, the internal erosion might have caused caving near the place where erosion initially took place, and continuing erosion might have resulted in a sink hole appearing on the surface of the dam.

Thus, it can be concluded that the nature of the Zone 1 material, and the manner in which it was utilized, were major factors leading to the failure of the Teton Dam.

Zone 2.

Zone 2 was intended to form ". . . a blanket and chimney drain in the downstream shell . . . to assure positive control of the phreatic line during periods of high reservoir levels and to provide drainage control of seeps through the foundation. . . ." (p. 24, USBR "Design Considerations," October 1971).

Record permeability tests made on remolded samples of Zone 2 material during construction raise some question about the suitability of this material for a drainage blanket and chimney drain. These test results suggest that much of the Zone 2 material may have been nearly as impervious as the Zone 1 material.

Grain-size analyses of large-volume samples of Zone 2 material taken from the remnant on the left side showed that between 5 and 11 percent of the material was finer than the No. 200 sieve. This suggests a low permeability. Further evidence that the Zone 2 material is less pervious than planned is the fact that Zone 2 of the dam remnant on the left is standing on very steep slopes, which a non-cohesive, free-draining sand and gravel would not do.

The low permeability of the Zone 2 material probably prevented the leakage which was occurring through the erosion channels in Zone 1 from making an earlier appearance at the downstream face of the dam.

CHAPTER 8
PROJECT DESIGN
(Panel Charges Nos. 5 and 7)

Teton Dam was designed under the direction of the Office of Design and Construction, U.S. Bureau of Reclamation, at the Denver Federal Center. Its Division of Design had responsibility for supervision and coordination of the design program.

DESIGN OF DAM

USBR design notes* indicate recognition of the difficulties presented by conditions at the damsite. Early consideration was given to control of seepage issuing from the foundation under the embankment and to prevention of piping due to cracks in the core. Some of the measures considered as the design evolved were: (1) excavation to groutable rock in key trenches; (2) excavation, dental treatment, special compaction, and slush grouting under the core outside the key trenches; (3) drains or pervious blankets on the downstream side of the cutoff trench; (4) blanket grouting, slush grouting, guniting, or special compaction of earthfill in the cutoff trench bottom; (5) downstream drain holes or tunnels; and (6) semipervious zones on the upstream and downstream sides of the core so that cracks in the core would not result in failure by piping. Not all of these measures were adopted in the final design.

The designer's notes identify the core material as a silt, or soil classification ML. It is labelled as abundant, inexpensive, strong in frictional resistance, low in permeability, erodible, and susceptible to cracking. Late in 1970, apparently to avoid damaging the environment by disturbing the alluvial materials downstream from the dam, the designers decided to use as much as possible of the silt, with a corresponding reduction of sand and gravel requirements.

The design finally adopted for the Teton Dam and its appurtenances is shown in Figs. 1-2 through 1-6. A description of this design follows:

The seepage barrier in the central part of the dam consisted of a wide impervious zone of silty material in the embankment proper and a grout curtain in the foundation rock. In the abutments it consisted of an impervious backfill of the same silty material in a key trench excavated to a depth of about 70 ft through the heavily jointed rock and, beneath the backfill, a continuation of the grout curtain. Beneath the spillway structure on the right abutment, the key trench was omitted and the grouted cutoff extended downward from the base of the structure.

A substantial transition zone of selected sand and gravel was provided upstream of the seepage barrier in the embankment (but not in the key trenches).

Downstream of the impervious core of the embankment proper was a drainage or filter zone of modest dimensions, consisting of selected sand and gravel. It extended to the bedrock of the valley floor and was then continued downstream beneath a random zone and a downstream rockfill zone. No transition material was placed between the impervious core and the alluvium on the bedrock valley bottom and no transition zone was provided in the key trench downstream of the grout cap

*Notes by W.G. Harber, USBR file, dated March 1967, November 1969, May 1970, and October 1970.

against the bottom or side of the bedrock trench. Instead, the core material in the key trench was placed directly against the rock by "special compaction" of a 2-ft thickness at a moisture content specified to be somewhat above that of the rest of the backfill and compacted by small hand-operated compactors or by rubber-tired equipment.

The specifications provided for grouting under pressure any faults, joints, shear zones, springs, or other foundation defects, when determined necessary by the contracting officer. There were no provisions for treatment of such foundation defects by the surface application of slurry grout.

During post-failure investigative excavation of the embankment remnant on the right abutment between the spillway and about Sta. 13+00, no evidence was found that joints had been treated on the bottom of the key trench in that area. Actual construction procedures are covered in Chapter 9.

No drainage system was provided for either abutment, and no downstream piezometers were called for.

Thus, the final design depended for seepage control almost exclusively on the impervious core, the key trench backfill and on the grout curtain. Although the upstream face of the impervious core in the embankment proper was protected by a transition zone, 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 fact, there is reason to question whether there was an effective downstream drainage zone anywhere since Zone 2 material does not appear to have been adequately permeable (Chapter 7).

Foundation Details.

The foundation treatment specified at Teton Dam was based upon an examination of drill hole data such as rock type, extent of fracturing, drill hole water losses, and on the pilot grouting done in 1969. It consists basically of four elements, as shown on Figs. 1-2 and 1-3:

- (1) 70-ft-deep, steep-sided key trenches on the abutments above El. 5100.
- (2) A cutoff trench to rock below El. 5100.
- (3) A continuous grout curtain along the entire foundation, extending about 1000 ft into the right abutment and about 500 ft into the left abutment. The grouting pattern consists of one row of grout holes with two outer rows of grout barrier holes, except in certain areas below El. 5100 where the foundation was less jointed and where only one or two lines of holes were placed. Actual techniques and patterns adopted during construction varied from those specified in design. (Chapter 9.)
- (4) Excavation to rock under Zone 1 on the abutments.

Stripping.

Stripping was required beneath all embankment zones outside of the cutoff trench. Stripping depths used to arrive at the specifications estimate were based on drill hole logs, test pit logs, and

descriptions and photographs in the design data. These data indicated that the foundation below El. 5040, with the exception of the existing river channel, would have to be stripped to an average depth of approximately three feet.

Outside the cut slopes of the cutoff trench and key trenches, the embankment foundation area was to be stripped to uncover material equal in strength to the overlying embankment materials. The valley floor downstream, underlying Zones 2 and 5, was to be stripped to expose Zone 2 type material to insure proper drainage characteristics in the downstream toe, and the abutments were to be shaped to provide a reasonably smooth surface that would permit adequate compaction of the embankment against the foundation with little or no special compaction.

In accordance with the USBR design practice of not requiring stripping to rock under coarse-grained zones, stripping specifications did not require removal of in-situ impervious soils from the abutments prior to placement of the Zone 2 blanket drain. As applied at Teton Dam, this requirement is based on the USBR's premises that:

- (1) "If the rock is open, any normal flow would be continued in the jointed rock."
- (2) "Concentrated normal flows which would surface could be handled by the Zone 2 gravel drain."
- (3) "Flows large enough to cause washing of silt beneath the gravel blanket into rock openings were not expected."

Stripping was not required under the El. 5041.5 downstream berm or under the fill on the southeast side of the powerplant tailrace channel.

Key Trenches.

A foundation key trench was to be excavated above El. 5100 on each abutment to intercept the more open rock joints and to reach a groutable horizon. The trench was to be excavated 70 ft deep, measured from the original ground surface, as shown in Fig. 1-3.

A bottom width of 30 ft was selected to provide space for construction equipment and for three lines of grout holes. In response to questions posed after the failure, the USBR stated that although the hydraulic gradient was recognized as somewhat higher across the key trench than the normal USBR standard, laboratory tests on the material did not indicate that there would be a problem with piping.

Spillway Treatment.

The key trench was omitted under the spillway. The bottom of the trench was sloped at 1.5:1 from the edges of the spillway cut down to the 70-ft trench depth. It was anticipated that blanket grouting, closely spaced closeout holes, and large-volume grout injection would be required to seal the foundation under the spillway.

The decision not to continue the key trench under the spillway was based on a desire to avoid differential settlement that might crack the spillway structure. The rock foundation under the spillway was judged to be adequate to carry the design load and was considered suitable for blanket

and curtain grouting. The cutoff beneath the spillway crest formed a portion of the grout cap for deep grouting of the right abutment.

Cutoff Trench.

The design width of the cutoff trench varies on the basis of a reference width of 30 ft at El. 4920, as shown on "Cutoff Trench Plan," (Fig. 1-3). The El. 4920 reference line was used to dimension the cutoff trench and does not indicate the bottom of excavation. Above El. 5030 on the abutments where overburden is shallow, the top of the cut slope for the cutoff trench is referenced to the intersection of outer slopes of Zone 1 with the bottom of stripping. This intersection line is shown in Fig. 1-3, and in "Typical Abutment Section A-A," Fig. 1-2.

Across the canyon floor the specifications estimate provided for excavation through recent river-deposited gravel and into the rock in the bottom of the cutoff trench. On the abutments the cutoff trench was to be excavated through weathered and loose or open-jointed rock to a firm relatively tight horizon. In addition, irregularities were to be removed and the remaining surface sloped to 0.5:1 or flatter.

Grouting.

Erosive seepage under the embankment was to be prevented by injecting the foundation with grout. Foundation investigations at the damsite indicated that large grout quantities would be required to produce a tight curtain, and that special procedures would be required to prevent excessive travel of the grout.

Water tests in core drill holes indicated the abutments above El. 5100 to be very pervious. In these areas, therefore, three rows of grout holes were to be provided, with the outer rows 10 ft upstream and 10 ft downstream from the center.

The pilot grouting program in 1969 demonstrated that conventional grouting procedures would result in grout travel far beyond the limits of the intended grout curtain. From a design standpoint, grout extending more than about 100 ft from a vertical plane through the grout cap was judged to serve no useful purpose and measures were taken to restrict treatment to the curtain area.

The stated purpose of the two outer rows of grout holes, spaced at 20 ft except in the basalt area, was to restrict grout travel "by pumping thick mixes and provide an upstream and downstream barrier to allow the centerline row of holes to be grouted effectively under pressure with less chance of traveling distances of several hundred feet downstream or upstream." The outer rows were "not expected to be completely solid barriers. . . ."

Except where interrupted by the auxiliary outlet works access shaft, a continuous grout cap was to be provided in rock formation for the full length of the dam. Concrete in the grout cap was to be placed approximately to the general level of the adjacent bottom of the trench after final cleanup. In estimating specifications quantities, grout cap excavation was assumed to range in depth from the 3-ft minimum up to 8 ft when crossing zones of intensely jointed rock. In the vicinity of the test areas for blanket grouting and curtain grouting, the centerline of the cutoff trench and the key trench was to be located so that the pilot grouting would become part of the final grout curtain.

Specifications permitted the contractor to construct the grout cap in the form of a stairway on the abutments. Steps, if constructed, were to be formed by extending the cap above the adjacent bottom of the trench.

The grout cap in the bottom of the key trench was placed in a notch with minimum specified cross-sectional dimensions of 3 ft deep by 3 ft wide. A detail is shown in Fig. 1-3. Excavation of this notch required some use of explosives, with the attendant probability of some fracturing of the rock adjoining the grout cap. Since this rock was already extensively jointed and fractured in its natural state, blasting for the grout-cap notch would have tended to worsen its condition.

Specific measures were not taken to assure sealing of the upper part of the rock under the grout cap, such as gravity grouting in closely spaced blanket holes.

Blanket grouting was to be provided for special treatment of open cracks, jointed areas, zones of high grout take, and other defects disclosed in the bottom of the cutoff trench or found during curtain grouting. Blanket grouting was not general, however. Blanket holes, if required, were to be located by the contracting officer as the work progressed.

Large open joints or cracks in the bottom of the key trenches and cutoff trench were to be treated by (1) cleaning out the crack with air and/or water jets, (2) setting grout pipe nipples in the crack, (3) sealing the surface by caulking and/or grout, (4) drilling, if required, and (5) low-pressure grouting through the nipples. Evidently little of this treatment was actually done, at least in the part of the key trench exposed by the Panel's investigations. Actual foundation treatment is described in Chapter 9.

The older alluvium beneath the intercanion basalt on the left side of the river bottom was investigated by water testing drill holes during the test grouting program in 1969 for the purpose of determining the groutability of the alluvium and overlying basalt. The results of the pilot grouting led to the conclusion that the estimated 5- to 15-ft thickness of alluvium between the basalt and rhyolite was groutable with a cement grout. For economic reasons, the alternative of chemical grouting was not specified.

Embankment Details.

The USBR volume titled "Design Considerations for Teton Dam," dated October 1971, documents the basic concepts of the embankment design. As seen on Fig. 1-2, the dam is composed of five zones. In addition, a thickness of 3 ft of riprap was placed on the upstream slope above El. 5185.

For economic and environmental reasons (primarily opposition to downstream channel borrow), consideration was given early in the design to building a nearly homogeneous dam predominantly composed of silt. However, for an embankment of the required height, the upland silts were found to have some undesirable characteristics, including a high percentage of fines, some caliche, a low maximum dry density of about 100 pcf, and a tendency to crack when subjected to differential settlement. Since Teton Dam would rest on about 100 ft of unconsolidated overburden and since it is in a seismically active region, the designers concluded that a homogeneous dam involved unacceptable risks and that the core of aeolian silt should be surrounded by sand and gravel for earthquake crack protection.

In addition to the Zone 1 silt and the Zone 2 sand and gravel, the design provides the third, fourth, and fifth zones to permit maximum utilization of required excavations for foundations and structures.

Embankment Zoning.

Zone 1 is the impervious core intended to form the water barrier of the dam. It was specified to consist of ML and CL type soils.

CH and MH type materials in the borrow areas were to be avoided or blended in the borrow cut with silty material to preclude layers of low-strength clay in the dam. Caliche and cemented soil that would break down under the roller could be blended and placed in Zone 1. Material predominantly composed of caliche and cemented hard layers of soil was allowable in Zone 3.

Zone 2 was intended to form a blanket and chimney drain in the downstream shell for the purpose of controlling the phreatic line during periods of high reservoir levels and to provide control of seepage through the foundation. The Zone 2 blanket was extended up the abutment so that there was a layer of Zone 2 between Zone 3 and the foundation in all sections of the embankment. Test pit logs in the borrow area indicated that the available material was predominantly gravels that tended to be deficient in fines. From the designers' standpoint, mixing the surface layer of silty sand with the underlying gravel was regarded as a desirable procedure when it could be accomplished in the normal excavation process; moreover, if concentrations of silty, sandy gravels were encountered in the borrow area, they could be utilized in Zone 2 provided they could be reduced to an acceptable moisture content. Such material was to be placed next to Zone 1 and in the upstream shell, reserving the more pervious gravels for the Zone 2 chimney next to Zone 3 and the blanket under Zone 3.

Zone 3 is composed of miscellaneous material placed in the downstream part of the embankment to accommodate material unsuitable for Zone 1 because of rocks larger than 5 in. or layers of caliche or hard-cemented materials that were excavated in the borrow areas. The top elevation of Zone 3 could be varied according to the quantity of material that became available from required excavation. Zone 1, 2, and 4 type materials were allowed in Zone 3.

Since Zone 3 was to provide structural stability, some degree of moisture control was regarded as essential. The best practicable placement moisture for the specified compaction effort was judged as probably slightly dry of optimum. It was recommended by the designers that placement moisture in Zone 3 be maintained approximately as recommended for Zone 1.

Zone 4 is a part of the upstream toe where silty sands and gravels were used to construct a cofferdam for river diversion. Zone 4 material was also used for the berm at El. 5041.5 at the downstream toe of the dam and for the storage areas downstream from the control and warehouse structure.

Zone 5 is the outer shell composed of rock from the required excavations in the cutoff trench, key trenches, abutment cleanup, river outlet works, auxiliary outlet works, spillway, and Borrow Area "C" and "C" Extension.

The riprap for upstream slope protection was basalt obtained from sources in the region near the dam.

Crest Details.

Teton Dam is located in Earthquake Zone 3 on the Seismic Risk Map of the United States (Fig. 6-1). At the crest, earthquake design considerations included heavy slope protection at the top of the embankment and sand and gravel fills around the silty core to provide filter blanket protection in the event the core was cracked. A 35-ft-wide crest was adopted to provide space for zones meeting these criteria. The crest was cambered 3.0 ft to allow for settlement.

Special Compaction.

"Special compaction" was required in the bottom of the cutoff trench and in the key trenches. For the specifications estimates, an average vertical depth of 12 in. of "specially compacted" material was assumed in the bottom of the cutoff trench below El. 5040 and above El. 5150 and in the bottoms of

the key trenches; however, the depth was expected to vary considerably, depending on the roughness of the foundation. Where a smooth surface was exposed, consideration was to be given to obtaining compaction at the embankment-foundation contact by placing the material adjacent to bedrock at optimum or slightly wet of optimum moisture content, by using a thicker initial layer, up to 12 in., and by increasing the number of roller passes to obtain compaction. An average of 24 in. of specially compacted earthfill, measured horizontally, was to be placed against the slopes of the key trench; also 24 in. of such compacted material measured horizontally would be required in the bottom of the cutoff trench against steep abutment slopes between Els. 5040 and 5150.

The Panel's investigation did not indicate that the fill against the rock in the key trench was wet of optimum.

Stability Analyses.

The embankment was analyzed for stability by the USBR standard procedure for stability analysis with the least factor of safety being determined by a computer program using an automatic search technique.

Design parameters for friction and cohesion were adopted as follows:

Material	Cohesion	Tan ϕ'
Zones 1 and 3	11.6 psi	0.58
Zones 2, 4 and 5	0	0.70

The parameters used for Zone 2 were also used for the alluvial foundation. The permeabilities of the materials were determined from laboratory tests. Pore water pressures for the high-level steady state and for the drawdown stage were computed from flow nets.

The analysis indicated safety factors considered by the designers to be conservative for rapid drawdown, high-level steady state, and the construction condition. Safety factors were calculated by the USBR to be as follows:

Construction Condition	1.47
High-Level Steady State	1.69
High-Level Steady State With Earthquake (with pseudo static factors, Horiz. = 0.1, Vert. = 0.0)	1.32
Rapid Drawdown	2.36

The Independent Panel has not reviewed these stability analyses further, since they are not regarded as pertinent to the failure.

Filter Criteria.

In USBR practice, filter criteria are used to design narrow filters but may be relaxed when zones of pit-run sand and gravel are incorporated in the embankment, as was the case at Teton Dam. The Bureau filter criteria for subrounded particles are:

$$R_{50} = \frac{50 \text{ percent size filter material}}{50 \text{ percent size base material}} = 12 \text{ to } 58 \text{ and}$$

$$R_{15} = \frac{15 \text{ percent size filter material}}{15 \text{ percent size base material}} = 12 \text{ to } 40$$

Although much of Zone 3 material was the same as Zone 1 silt, a filter was not included between Zone 3 and the Zone 5 rockfill since Zone 2 upstream from Zone 3 was expected to control the phreatic line and prevent Zone 3 from becoming saturated.

Consideration of Differential Settlement.

Design consideration was given to possible differential settlements and subsequent cracking of the low plasticity Zone 1 fill due to the steep rock abutments and deep key trenches. However, the USBR has stated in response to questions after the failure, that its experience with such material had not indicated that tension would develop in the embankment, even with such foundation configuration.

Instrumentation.

According to the statements made after the failure, devices for measuring internal movements and water pressures were considered by the USBR to be unnecessary at the Teton Dam because instrumentation was "not normally used for structures which are constructed of materials previously instrumented at other dams and for which [there are] satisfactory performance records." Performance records were said to be available for dams "constructed with similar material and on similar foundations."

DESIGN OF AUXILIARY OUTLET WORKS

The auxiliary outlet is a concrete-lined tunnel through the right abutment, with a diameter of 6 ft upstream of the gate chamber and 7 ft 6 in. downstream from that point. (The latter diameter was increased from 7 ft 3 in. after the design was done.) Its centerline coincides with the projected centerline of the spillway for most of its length. Details of this facility are shown in Fig. 1-4. The auxiliary outlet works were designed to accommodate streamflow for the period from October 1 to April 30, during which time the river outlet works were to be completed, and for passing riverflow while any future repairs or inspection of the river outlet works was taking place. A discharge rating curve is shown in Fig. 1-2.

The tunnel and adit, the gate and shaft chambers, and the access shaft were excavated in densely welded ashflow tuff for their entire distances.

DESIGN OF RIVER OUTLET WORKS

The river outlet works through the left abutment include a 111-ft-high, 13.5-ft-diam intake structure, a 2,127-ft-long, 13.5-ft-diam tunnel; and a 320-ft-high, 18.5-ft-diam gate chamber shaft. The tunnel was built to serve as the main outlet works and as the intake to the powerplant. Details of this facility are shown in Fig. 1-5. A discharge rating curve is provided in Fig. 1-2.

Most of the tunnel was driven, and the intake and gate chamber access shafts were excavated, in densely welded ashflow tuff.

DESIGN OF SPILLWAY

Details of the spillway at Teton Dam are shown in Fig. 1-6.

The spillway design was based on requirements for passing a flood with a peak inflow of 22,400 cfs and a 15-day volume of 200,000 acre-ft with a reservoir level at El. 5324.3. A discharge rating curve is shown in Fig 1-2.

COMMENTS

From a design standpoint, the appurtenant structures of the dam had no direct relationship to the failure. On June 5, 1976, the reservoir water had entered the approach channel of the spillway, but did not reach its crest. The spillway therefore was not required to function. Nothing in the operation of the auxiliary outlet works indicated any deficiency in design. Construction of the river outlet was incomplete, so its design was not tested during the emergency.

The capability of rapid lowering of a reservoir during a crisis is an important consideration in sizing outlet facilities. There are no widely accepted rules for satisfying this general requirement. In fact, many important dams have no facilities to permit emptying the reservoir quickly. In the case of Teton Dam, the combined capacity of the two outlets, if both had been operable, was approximately one-third more than necessary to pass reservoir inflows during the ten days preceding the disaster. The capacity in excess of this requirement was enough to enable lowering the reservoir level about one foot per day. This indicates that a moderate emergency capability was designed into the outlet system.

Comments on the relationship of the design to the failure are presented in Chapter 12.

CHAPTER 9
PROJECT CONSTRUCTION
(Panel Charges Nos. 6 and 8)

CONTRACT AND SUBCONTRACT AWARDS

The final design of Teton Dam was completed in early 1971, and the drawings and specifications were issued under Specifications No. DC-6910, Volumes 1 to 4. Invitations for construction bids to cover all items of the dam and associated facilities, except major electrical and mechanical items at the power and pumping plant, were issued on July 22, 1971. Bids were received on October 29, 1971. Contract award was made to the joint venture of Morrison-Knudsen-Kiewit on December 13, 1971, and notice to proceed was given on December 14, 1971. The contract award totalled \$39,476,142.

The contractor awarded a number of subcontracts during the progress of the project, such as the one to McCabe Bros. Drilling Company for drilling and foundation grouting for the dam.

SPECIFIED CONSTRUCTION SCHEDULE

The required overall schedule is given in Par. 15 of the General Conditions and the schedule details are deferred to a contractor-prepared schedule under Par. 17. The basic requirement was that all work be completed within 1800 days of notice to proceed. Accordingly, the contract completion date was November 17, 1976. Because of change orders, the date was extended to October 27, 1977.

The detailed schedule, as mutually amended and agreed to between the contractor and the USBR in early 1976 has been examined only to the extent necessary to evaluate compliance with critical dates for certain features required for handling water storage and controlled release. A comparison of required and attained dates is given below:

Item	Completion Date	
	Contractor's Approved Schedule	Actual
Auxiliary Outlet Works (In Service)	Aug. 31, 1975	Oct. 3, 1975
Embankment (Essentially Complete)	Nov. 15, 1975	Nov. 26, 1975
Spillway (Essentially Complete)	May 30, 1976	June 4, 1976
River Outlet Works	Mar. 31, 1976	(Incomplete June 5, 1976)

The significance of the delay in completion of the river outlet works is discussed in Chapter 10.

DIVERSION AND CARE OF RIVER

Diversion of the river was a responsibility of the contractor, and was initially handled by a 70-ft-high embankment cofferdam located under the upstream toe of the main dam. The contract required a 13.5-ft-diam lined river outlet tunnel through the left abutment. The cofferdam was constructed on river alluvium with underseepage controlled by pumped wells downstream. The crest elevation was El. 5100, providing a diversion capacity of about 5300 cfs. Diversion through this left abutment tunnel,

later to be converted to serve as the permanent river outlet works, was commenced on June 8, 1973, following a 14-month tunnel construction period.

Immediately thereafter, construction was started on the auxiliary outlet works tunnel through the right abutment. This 6.0 to 7.5-ft-diam lined tunnel, with intake invert at El. 5047, was completed in September, 1975; and on October 3, 1975 the river outlet tunnel entrance was closed by placement of stoplogs and all subsequent diversion of river flows past the damsite was made through the auxiliary outlet. Since these works had a maximum rated capacity of slightly more than 850 cfs, river flows in excess of that capacity, a common occurrence in late winter and spring, could only partially be passed, with storage of the remaining flow.

SITE PREPARATION

Clearing.

The specifications required clearing and stripping at the dam and powerhouse sites only.

Excavation.

In regard to the damsite, the specifications required stripping of all loose topsoil and organic materials from the entire site, excavation of alluvium from the riverbottom cutoff trench, and excavation of blasted rock from the two abutment key trenches. From photographs of construction, it is clear that stripping of the abutment areas under Zone 1 was taken down to the bedrock surface, but under the other embankment zones relatively shallow stripping was done, leaving slopewash and talus materials in place. Excavation volumes originally planned were estimated as follows:

Abutment Stripping	100,000 cu yds
Abutment Key Trenches	350,000 cu yds
Cutoff Trench	650,000 cu yds

This work was carried out essentially as planned, except that it was apparent from as-excavated cross sections that the planned 1/2:1 key trench side slopes specified in the design were somewhat impracticable in the initial 20 to 40 ft depths below bedrock surface because of the loose, intensely jointed nature of rock at those depths. Accordingly, it was necessary to lay back the slopes of the upper third to half of the trench depth to inclinations varying from 1:1 to 2:1, substantially increasing the rock excavation volume.

Damsite excavation was initiated on April 17, 1972, working at the left end of the cutoff trench area, and was essentially completed by late 1973.

Drainage.

Excavation of the cutoff trench through alluvium to a depth of about 100 ft in the river bottom portion of the site between about Stas. 17+00 and 24+00 required extensive drainage of seepage from beneath both upstream and downstream cofferdams and from the abutments. The control facilities used consisted principally of pumped wells in the alluvium and pumped sumps on the cutoff bottom at rock surface. These systems have not been studied in detail because of their unlikely relationship to the failure of the dam.

PROJECT SURVEYING RECORDS

Information available to the Panel indicates that the project was provided with conventional second order surveying controls for both horizontal and vertical reference points. The local horizontal controls were tied to first order geodetic surveys at distance of 15 to 30 miles from the project. Vertical controls were based on available U.S. Coast and Geodetic Survey bench marks. Details of project survey data relating to settlements and deflections are presented in Chapters 5 and 11. The Panel believes that the surveying work was conventional and acceptable.

FOUNDATION GROUTING AND TREATMENT

General.

It is clear from the great volume of records, summaries, reports and data that, during construction, special emphasis was placed on foundation grouting. Because it is impracticable to attach all this information, a summary is given here. A more extensive description is given in the paper by Peter P. Aberle, published by the American Society of Civil Engineers in *Rock Engineering for Foundations and Slopes*, 1976, Vol. 1, and entitled: "Pressure Grouting Foundation on Teton Dam." The total grouting program entailed drilling 118,000 lin ft of grout hole and injecting nearly 600,000 cu ft of cement, sand and other materials, at a contract cost of \$3,800,000.

Site Conditions.

From preconstruction geologic evaluation of the damsite, it was apparent that much of the foundation bedrock to depths of at least up to 100 ft was highly pervious, and that curtain grouting would be difficult, extensive and expensive. To obtain a quantitative assessment of the problem, a Pilot Grouting Program was carried out on the left abutment in 1969, as described in Chapter 4. This program showed that it would be extremely costly to attempt to curtain-grout the upper 70 ft of foundation bedrock for the dam above El. 5100. Accordingly, the decision was made to excavate the relatively ungroutable rock to a depth of about 70 ft on both abutments from El. 5100 upward to the ends of the dam, and to begin the grout curtain under a concrete grout cap in the center of the excavation. The trenches, for economy, were designed to be deep, narrow and steep-sided.

In addition to the major adjustment to site conditions of locally substituting a key trench filled with impervious Zone 1 for a grout curtain through highly jointed, pervious bedrock, the designers concluded that extensive curtain grouting beneath key and cutoff trenches would be required. To indicate the scope, the bid items included provision of 55,000 barrels of cement, and 1,700 cu yds of sand, together with 260,000 cu ft of pressure grouting. Actual quantities of cement injected were over twice the bid quantities.

Grout Curtain.

The drawings and specifications called for three rows of deep grout holes along most of the axis of the key and cutoff trenches, with wide latitude retained by the USBR to direct and modify specific details. The center row of grout holes, intended to form the impermeable curtain, was provided with a concrete grout cap, nominally 3 ft wide by 3 ft deep in a drilled and blasted notch in rock. In the key trenches, the specified grouting sequence was: First, the downstream row of holes on 20-ft centers; second, the upstream row of holes on 20-ft centers; and third, closure along the center row of holes working through the grout cap. These center holes were spaced on 10 ft centers, with split spacing where the primary holes did not indicate a tight curtain. It is important to recognize that, as this procedure was actually carried out, neither the upstream nor the downstream rows constituted grout "curtains" as the term is conventionally understood. Actually full closure along the two outer

rows was neither attempted nor attained. Both the grouting procedures and hole spacing along the outer rows were such that gaps could be judged to be inevitable. The outer rows were intended to be only semi-pervious grout barriers against which the center row of grout holes could reasonably be fully and successfully grouted.

Accordingly, it is the Panel's view that a triple or 3-row grout curtain was not constructed. Instead it should be termed a single-row curtain.

The Panel has reviewed and analyzed the grouting records and reports, including review of the specifications, reports and records, sufficiently to assess the methodology and scope and to judge the effectiveness. Perhaps more importantly, in what were judged to be the critical reaches of the grout curtain between key-trench Stas. 3+00 and 15+00, the Panel had coring and water pressure testing carried out along the center row of grout holes to assess directly the probable effectiveness of the grouting. The testing has been reported in Chapter 3, with results summarized in Tables 3-2, 3-3 and 3-4.

With particular reference to those tests in the general failure area between Stas. 13+50 and 14+26, the results show that more than 40 percent of the 30 water loss tests, run at depths up to 34 ft below key-trench invert, exceeded 0.1 gpm/ft of hole. Twenty percent of the tests exceeded a loss of 0.5 gpm/ft, and 7 percent exceeded 1.0 gpm/ft. Accordingly, the tests indicate that in the critical key-trench area the grout curtain was not fully closed.

The Panel's water loss tests under the spillway at depths of up to 145 ft, as more fully reported in Chapter 3, and in two properly positioned holes in the grout curtain near the right end of the dam at depths up to 300 ft, showed satisfactory grouting. A third hole near the right end showed high water losses but in a subsequent survey was found to be located out of the grout curtain.

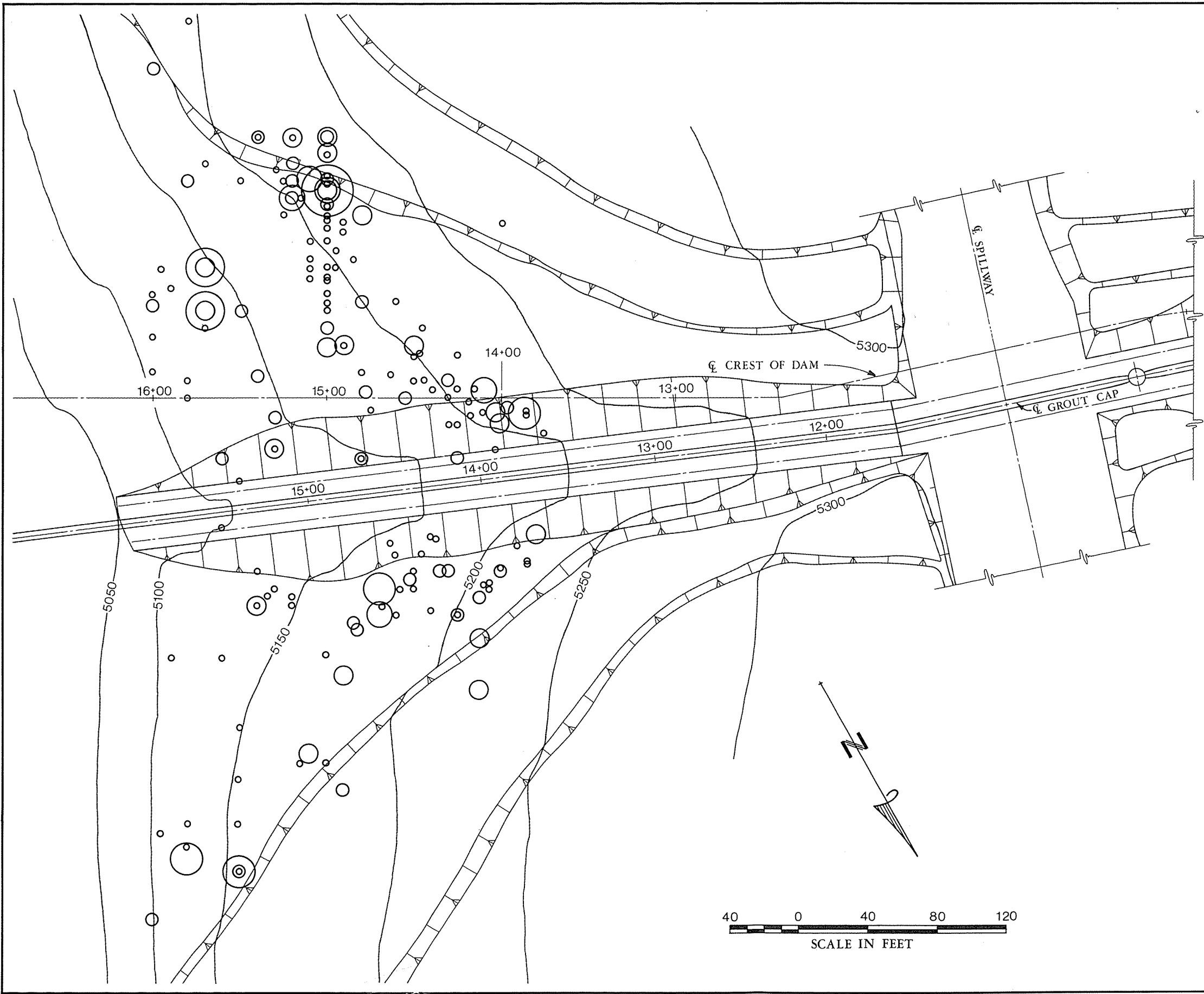
Blanket Grouting.

Par. 100 of the specifications requires "blanket grouting" in the key trench and cutoff trench, as directed by the USBR. No definition of the term is given, but it appears that it entailed drilling and grouting both uniformly and randomly spaced and angled holes to shallow depths (20 to 35 ft) to intercept and plug open joints. The results of the work are shown on USBR drawings. The scope of the blanket grouting done was limited, and the areas so treated were almost exclusively in the bottom of key and cutoff trenches, and at only a few local spots. A major exception was at the spillway crest structure where a close pattern of 80-ft-deep "blanket" grout holes is shown under the entire structure.

Slurry Concrete.

A review of the drawings and specifications has failed to show that it was expected to treat open bedrock joints at the Zone 1-to-bedrock contact with slurry concrete. USBR Project Office records show, however, that a total of about 1830 cu yds was placed at the instigation of that office (Fig. 9-1). This was accomplished principally by pouring slurry into open joints and the more obviously open cracks. This procedure was discontinued above about El. 5210. Fig. 9-1 illustrates the extent of slurry grout or concrete placement on the side slopes of the key trench and in the stripped bedrock areas upstream and downstream from the key trench on the right abutment, principally beneath Zone 1. It is particularly evident on Fig. 9-1 that, in the failure area, significantly large open joints existed at the top of the downstream face of the key trench at axis Sta. 14+00 and near the downstream toe of Zone 1 opposite Sta. 15+00, where slurry takes totaled 100 or more cu yds.

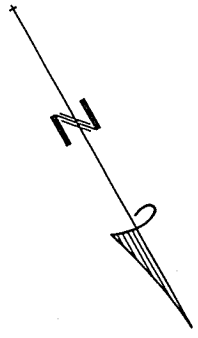
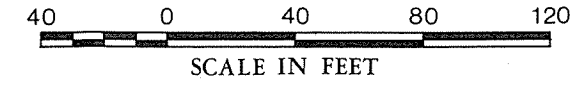
These large takes under gravity placement conditions identified the rock as being extremely pervious, indicating that grave incompatibility existed between the highly erodible Zone 1 fill and its underlying intensely jointed foundation.



LEGEND

- 0-5 CUBIC YARDS
- 6-10 CUBIC YARDS
- 11-20 CUBIC YARDS
- 21-30 CUBIC YARDS
- 31-40 CUBIC YARDS
- 41-50 CUBIC YARDS
- GREATER THAN 50 CUBIC YARDS

REFERENCE DATA:
TETON DAM FAILURE EXHIBIT NO. 4



**SLUSH GROUT DISTRIBUTION AND
DENSITY ZONE 1 FOUNDATION
STA. 11+41 TO 16+00**

FIG. 9-1.

U. S. DEPARTMENT OF THE INTERIOR - STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

River Alluvium.

At the maximum sections of the dam, where the bedrock canyon was filled with up to 100 ft of alluvium, the design permitted constructing the upstream and downstream shells of the dam on this material, except for stripping away organic and loose materials. A cutoff trench down to bedrock, with bottom width actually constructed a minimum of 80 ft wide, was required to control underseepage.

In view of the generally coarse-grained nature of this alluvium, which from upstream areas provided the "silt, sand, gravel and cobbles" utilized for fill as Zone 2, there was no engineering justification for fully removing all alluvium under the outlines of the dam.

It is believed that there were no aspects of construction activities involving treatment of the river alluvium left in place under the dam that had any significant influence on the failure.

Talus and Overburden on Abutments.

The Drawings and Specifications, as interpreted, permitted slopewash materials, overburden and talus to be left in place under the dam, in all areas outside of the Zone 1 fill (Figs. 9-2, 9-3, 9-4 and 9-5). Under Zone 1, stripping to bedrock was required. Stripping of overburden was subject to the Construction Engineer's judgment as to the amount necessary to uncover reasonably strong materials.

It seems conclusive, from examination of as-excavated cross sections taken at 10-ft intervals along the axis between Stas. 14+50 and 15+50, as well as construction photos and oral inquiry, that a substantial thickness of talus was left under the dam downstream from the Zone 1 fill. This conclusion is based on interpretation of the original topography, as against the belief that steep bedrock cliffs existed under that topography together with an admittedly subjective evaluation of the cross sections. No direct evidence remains, all overburden and talus in the vicinity, together with a large but unknown volume of the blocky bedrock cliffs having been washed away following the failure. However, supplemental inquiry of the project construction staff confirms that substantial volumes of talus were left in place under the outer zones of the dam.

If, as is believed, a large volume of bouldery talus existed along the canyon wall under the downstream slope of the dam in the vicinity of axis Sta. 15+00 to Sta. 17+00, it could have provided an exit conduit for the initial, relatively restricted leakage across or under the key trench at about Sta. 14+00, and increasing leakage flows could have had explainable exits at the groin of the dam at El. 5200 and El. 5045 where the large flows of muddy water were seen on June 5.

Quality Control.

The Panel has not undertaken a detailed review of construction quality control procedures or personnel, except indirectly through examinations of reports on physical characteristics of the construction materials used in the dam, of the Construction Engineer's reports, and through review of certain design and construction decisions.

Based on its contacts with the USBR project construction staff, the Panel considers that the project was properly staffed with knowledgeable, interested, supervisory personnel, and that all required aspects of quality control were faithfully carried out. If any substantive questions regarding construction quality control could be raised, it would seem that it would not be in the areas of tests or reports which are clearly designated by long standing USBR practice, but rather in the areas of reaction to and exercise of judgment in matters more related to fundamentals of conceptual design than to execution of construction.



Fig. 9-2 Foundation formation and contact beneath zone 2 and zone 5



Fig. 9-3 Foundation formation and contact beneath zone 2 and zone 5

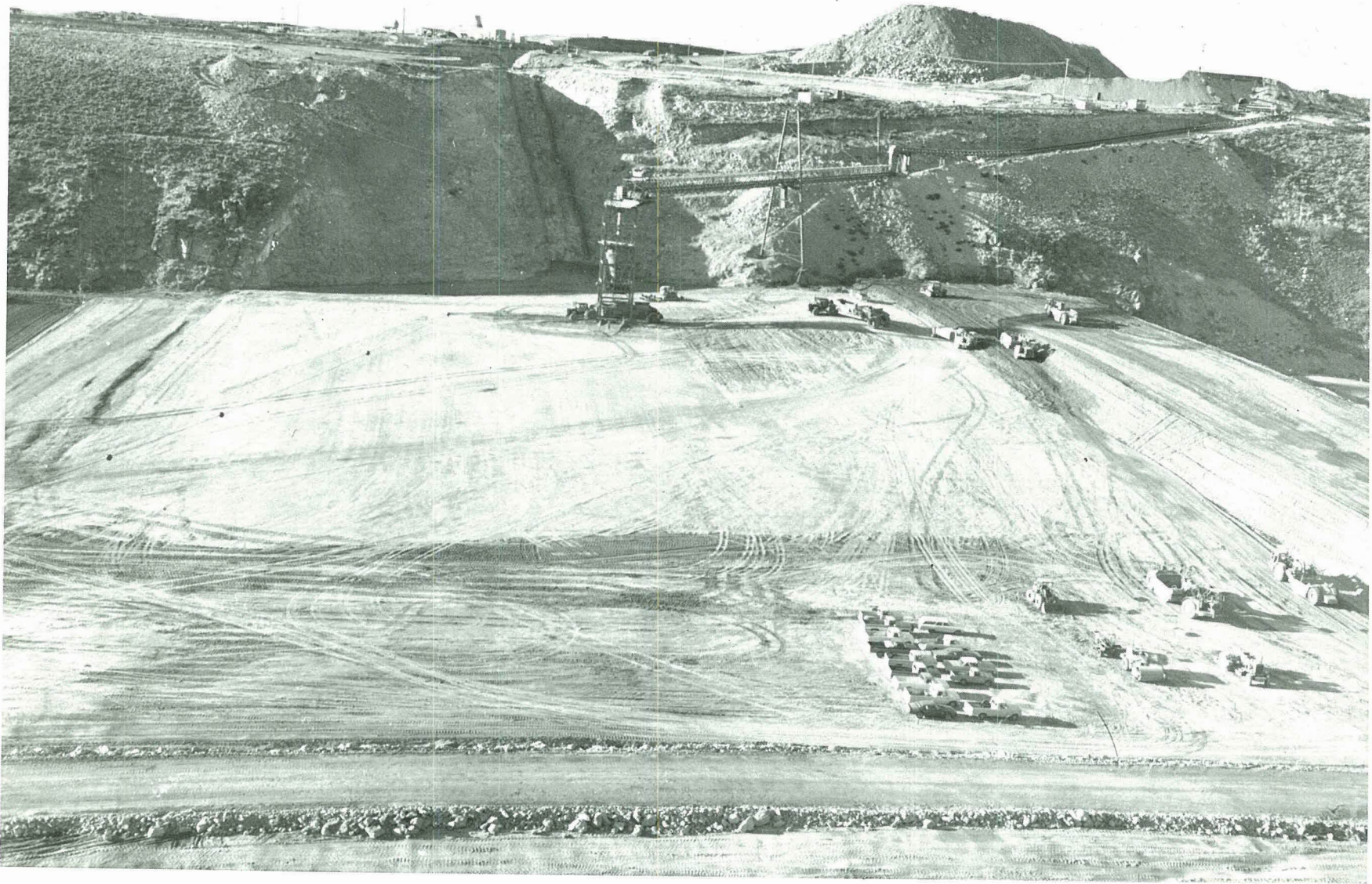


Fig. 9-4 Right abutment excavation showing the variable qualities of foundation for the various embankment zones and for the key-trench invert. Zone 1 is approximately El. 5145.

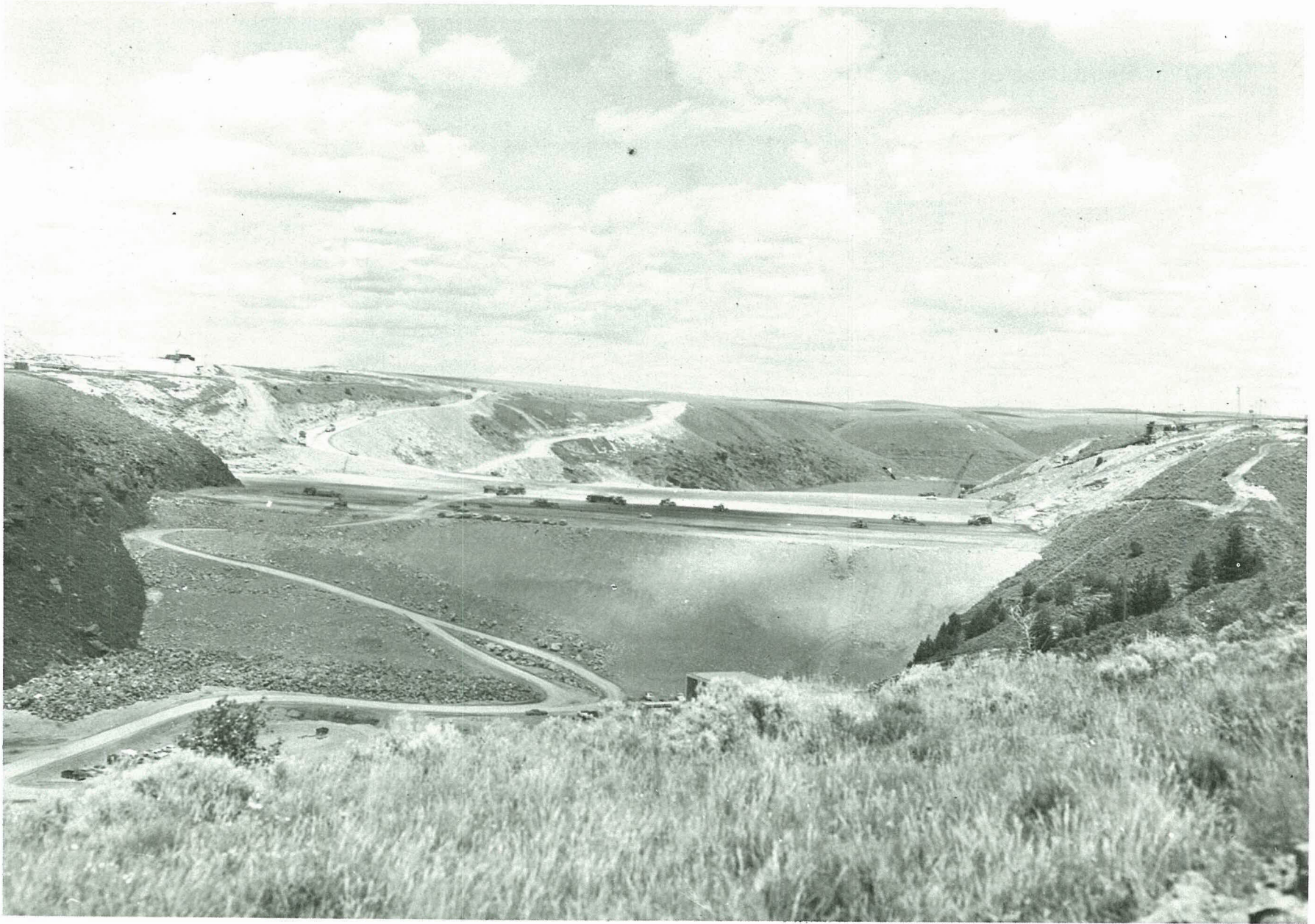


Fig. 9-5 Right abutment (to left in photo) showing minimal stripping (vegetation only) in preparing Zones 2 and 5 foundation. Zone 1 is approximately El. 5170.

DAM CONSTRUCTION

General embankment construction started in late June 1972 with placement of Zone 2 downstream from the cutoff trench near the left abutment. It was essentially completed by November 26, 1975. The necessary operations of stripping, excavation, dewatering, grouting, borrow pit and quarry development, and fill placement generally progressed concurrently throughout that period. Key dates throughout the construction were as follows:

Chronology.

Feb. 22, 1972	First construction equipment arrived.
Mar. 17, 1972	Started excavation of river outlet works.
Apr. 17, 1972	Started excavation of left key trench.
Apr. 25, 1972	Started fill placement downstream from cutoff trench.
July 11, 1972	Started excavation of right key trench.
July 17, 1972	Started stripping at spillway.
Oct. 20, 1972	Started drilling and grouting operations in left key trench.
Dec. 1, 1972	Fill operations suspended.
Dec. 13, 1972	Set first steel liner in river outlet works.
Apr. 10, 1973	Fill operations resumed.
June 5, 1973	River outlet works tunnel grouting completed.
June 8, 1973	River diverted through river outlet works.
June 25, 1973	Started excavation of auxiliary outlet works and upstream cofferdam at El. 5100.
Sep. 17, 1973	Grouting begun in cutoff.
Oct. 18, 1973	First Zone 1 fill in cutoff.
Nov. 7, 1973	All fill operations shut down.
Dec. 12, 1973	Completed grouting left abutment.
Apr. 4, 1974	Resumed fill placement.
Nov. 27, 1974	Fill operations shut down, majority at El. 5130.
Apr. 29, 1975	Resumed fill placement.
May 9, 1975	Completed concreting auxiliary outlet works tunnel.
Oct. 3, 1975	Closed river outlet works and diverted river to auxiliary outlet.
Oct. 21, 1975	Completed Zone 1 placement.
Nov. 26, 1975	Dam essentially complete, 200 ft of fill placed in 6.7 months.
Oct. 1975- June 1976	Modifications to river outlet works to permit project use as power and irrigation water outlet, including concrete elbow, trash racks, gates and repainting liner. Repainting incomplete as of 6/5/76.

Embankment Materials.

The sources and physical characteristics of the embankment materials are presented in Chapter 7.

Placement Procedures.

The five zones of the dam embankment were constructed in accordance with accepted practices and their placement was controlled by the project inspection forces exercising the specification provisions.

Specification requirements for placement are abbreviated in Table 9-1.

Permissible variations in compaction equipment were made by also using vibrating rollers on Zone 2, sheepsfoot rollers on Zone 3, and 40,000-lb crawler tractors on Zone 4.

The sources and engineering properties, excavation, hauling, stockpiling, and handling of the embankment materials, and the achievement of the density requirements as revealed by the earthwork construction control records are presented in Chapter 7.

The plans and specifications contain a provision for specially compacted earthfill Zone 1 at steep and irregular abutments and on rough and irregular embankment foundations to include "...each layer... shall be compacted by special rollers, mechanical tampers, or by other approved methods... moisture and density shall be equivalent to that obtained in the earth fill placed in the dam embankment..." This provision was nothing more than a requirement for a method of compaction in confined locations. For example, no special treatment regarding increased moisture content was specified. The project office did require selective placement of material having a slightly higher moisture content but the average was only 0.7% wetter than that for the normally compacted fill and was still 0.5% dry of optimum moisture. Wheel rolling by loaded trucks, and power tamping by gasoline or air hammers were used.

Embankment was placed during four construction seasons, 1972 through 1975, commencing June 15, 1972 and terminating November 26, 1975. Embankment was not placed during the cold winter months.

AUXILIARY OUTLET WORKS

The auxiliary outlet (Fig. 1-4) is located in the right abutment of the dam. The concrete-lined tunnel is 6 ft in diam upstream of the gate chamber and 7 ft 6 in. downstream. A 7-ft-6-in.-diam adit and an 8-ft-diam shaft provide access to the gate chamber.

The auxiliary outlet diverted the Teton River while the river outlet was being modified and completed for use as a permanent power and irrigation water outlet.

Tunneling Experience and Geology – Right Abutment.

The auxiliary outlet tunnel, adit, and gate chamber access shaft were driven in consistently hard, lightweight, densely welded, crystal rich, rhyolitic ash flow tuff. Tunneling conditions were excellent throughout. Shears and numerous joints subparallel with the tunnel alignment were present. No water flows were encountered.

In some areas, hydrothermal alteration adjacent to joints and locally disseminated through the rock has discolored the tuff reddish brown to brick red, but had little or no other apparent effect on the character or quality of the welded tuff.

Two prominent, well-developed joint sets were observed. The joints of one set generally strike on an average of about N30°W. Joints of the other set generally strike on an average of about N60°W. The joints of both sets are vertical to steeply inclined. Joints of the N30°W striking set appear to be the strongest, most consistent of the two sets except between tunnel Stas. 21+40 and 27+80 where joints of the N60°W striking set not only are very strongly developed and generally closely spaced, but form virtually all of the joints present.

TABLE 9-1
EMBANKMENT PLACEMENT SPECIFICATIONS

Zone	Compacted Lift Thickness	Moisture Control	Compaction Equipment	No. of Passes	Density Control
1	6 in.	1/2 to 1-1/2% dry of optimum	5-ft x 5-ft 4,000 lb/ft sheepsfoot	12	Average compaction not less than 98% of std. AASHO
2	12 in.	Thoroughly wetted	40,000-lb crawler-type tractor or approved alternate	4	Relative density not less than 65% but no more than 20% of tests less than 70%
3	12 in.	Most practicable	50T pneumatic-tired roller	6	Best practicable degree
4	12 in.	Optimum amount required	50T pneumatic-tired roller	6	Maximum obtainable by the specified procedure
5	36 in.	None	Hauling and placement equipment	Equipment routed	Maximum amount possible

Only minor seeps or drips occurred locally from joints and rock bolts in the tunnel crown and from joints in the crown of the adit near its intersection with the gate chamber. However, numerous seeps ranging in volume from heavy drips to wet or damp rock reportedly developed between Stas. 29+20 and 34+11.5 (the downstream portal) during the spring runoff following the completion of the tunnel excavation.

It was reported that water and a small volume of grout leaked into the tunnel at Sta. 17+73 from a spillway blanket grout hole located 18 ft upstream from dam centerline Sta. 11+04 (dam C.L. Sta. 11+06.5 = spillway Sta. 18+30), and a very small amount of grout also leaked into the tunnel at Sta. 16+81 from a grout hole 18 ft upstream from dam C.L. Sta. 10+78.

Control Works.

Flow through the auxiliary outlet is controlled by two tandem 4-ft x 4-ft hydraulically operated high pressure slide gates located beneath the gate chamber. Control stations are located in both the shaft house at El. 5332 and the gate chamber at El. 5048. The control cabinet equipment is located in the shaft house. At the time of failure the permanent electric power facilities, including duct banks in the crest of the dam, had not yet been installed. However, a temporary power source was available and the gates were open at that time.

Means for unwatering the tunnel upstream of the control gates are provided by sectionalized steel stoplogs which were designed to be set in position by a barge-mounted crane under balanced head conditions.

Schedule.

Excavation for the auxiliary outlet commenced April 13, 1973. All concrete was completed July 28, 1975. On October 3, 1975 the river flow was diverted from the river outlet to the auxiliary outlet.

Post-failure Condition.

As discussed in Chapter 3, no unusual conditions were found during the Panel's inspection of the unwatered auxiliary outlet tunnel on October 4, 1976.

There is no known aspect of the auxiliary outlet works construction that is believed to have a bearing on the failure of the dam.

RIVER OUTLET WORKS

The river outlet works (Fig. 1-5) are located in and on the left abutment of the dam. They consist of a 13.5 ft diam lined tunnel, controlled at mid-length by an hydraulically operated, 10.5 ft by 13.5 ft wheel gate. The gate shaft extends from the crest of the dam vertically to the gate chamber. The downstream end of the tunnel, as finally completed, branches into four steel pipes, each being controlled by a 4 ft by 4 ft outlet gate.

The river outlet works were constructed in two stages: The first stage consisted only of tunnel and shaft excavation and lining. At the end of this stage, the tunnel was utilized as a free-flow conduit for initial diversion of the river around the damsite. The tunnel was begun in March 1972 and went into service for diversion in June 1973. This service extended to October 1975. At that time the auxiliary outlet works were put in service. Whereupon, the upstream end of the river outlet tunnel was plugged and second stage construction began. That stage principally included constructing a permanent intake, installing gates and associated operating equipment, constructing penstocks, and recoating the

steel liner in the downstream half of the tunnel with coal tar enamel. This work was not quite complete on June 5, 1976, although the required completion date was March 31, 1976.

Control Works.

The 10.5-ft by 13.5-ft wheel gate previously mentioned was completely installed and in operating condition by May 17, 1976. There was reservoir pressure on its upstream face at all times after May 14.

The 4 ft by 4 ft outlet gates were in operating condition on June 5, but it is indicated that they had not yet been connected to the available power source. They were fully open at that time, the openings serving as access and ventilation for the tunnel liner coating work still in progress.

Schedule.

The only item of Stage 2 River Outlet Works construction which was incomplete on June 5 is believed to have been coating the steel tunnel liner. This work must have been virtually complete, since communications from the project office stated that completion had been expected by June 10.

Post-failure Condition.

It has not yet been possible to unbury, dewater and inspect the river outlet works.

SPILLWAY

No detailed discussion is offered of spillway construction because it has not been found to have any influence on the dam failure. Two items of interest are noted, however: (1) the spillway was operational on June 5; and (2) according to oral reports by project personnel, there was no indication at any time of discharge of groundwater from the spillway underdrains, before, during, or after the dam failure.

COMMENTS

For construction of the grout curtain, the Panel considers that reliance on a single curtain with nominal hole spacing of 10 ft and with holes inclined in only one direction was unduly optimistic. The use of smaller hole spacing, cross-angled holes, and multiple curtains would have been justifiable in the light of known rock conditions. It is not suggested, however, that even these measures would have provided adequate closure for the embankment as designed.

In view of the known presence of a maze of open joints in the bedrock under all of Zone 1 embankment, the Panel would not have concurred with the decision to limit blanket grouting essentially to the bottoms of the key and cutoff trenches.

In going forward with the initial filling of the reservoir, the USBR clearly had arrived at the judgment that the Contractor's construction was completely acceptable from the standpoint of the structural safety of the dam. Probably the only significant aspect of project construction wherein a failure to meet design requirements may be judged to have occurred was the contractor's failure to meet the approved construction schedule for completion of the river outlet works.

CHAPTER 10
RESERVOIR FILLING EXPERIENCE
(Panel Charges Nos. 9 and 12)

The Teton River drains an area of about 1,000 square miles on the west side of the Teton Range in Wyoming and Idaho, as shown in Fig. 10-1. It is the largest tributary of the Henrys Fork of the Snake River. The drainage area above the Teton Dam is 853 square miles. Principal tributaries of the Teton above the damsite are Canyon, Bitch, Badger, Leigh, and Teton Creeks. Canyon Creek enters Teton River within the reservoir area five miles upstream from the dam. The other tributaries and the headwaters of Teton River drain the west slopes of the Teton Range and provide most of the flow of the river.

The Teton River basin includes a broad agricultural valley cut by a narrow 20-mile-long canyon. Elevations range from 4800 to more than 13,000 ft.

Precipitation in the Teton basin varies from about 13 in. per year at Sugar to over 40 in. per year in the Teton mountains. Average annual precipitation at Driggs in the upper valley is about 15 in. Most of the annual precipitation occurs in the form of snow.

HYDROGRAPHIC RECORD PRIOR TO 1976 WATER YEAR

Characteristics of flows at the gaging station Teton River near St. Anthony, about five miles downstream of Teton Dam, are shown on the summary hydrograph in Fig. 10-2. Approximately 37 percent of the annual runoff in an average year occurs in May and June as a result of snowmelt. The average annual runoff at this station is about 580,000 acre-ft.

Frequency curves of runoff volumes for the months of April and May are shown on Fig. 10-3. A frequency curve of springtime flood peaks is shown in Fig. 10-4. The two largest flows of record occurred in the winter as a result of rain and snowmelt on frozen ground in the lower areas of the basin. Instantaneous peaks were:

Feb. 12, 1962	11,000 cfs
Feb. 3, 1963	7,280 cfs

These floods were of short duration compared with the spring snowmelt floods.

RESERVOIR OUTLET WORKS

Low-level reservoir discharge was to be accomplished through the river outlet works in the left abutment and by the auxiliary outlet works in the right abutment. The design capacities of these facilities at a maximum water surface elevation of 5324.3 ft were 3,400 cfs and 850 cfs, respectively. The area and capacity curves of the reservoir and discharge ratings for the spillway and outlets are shown on Fig. 10-5.

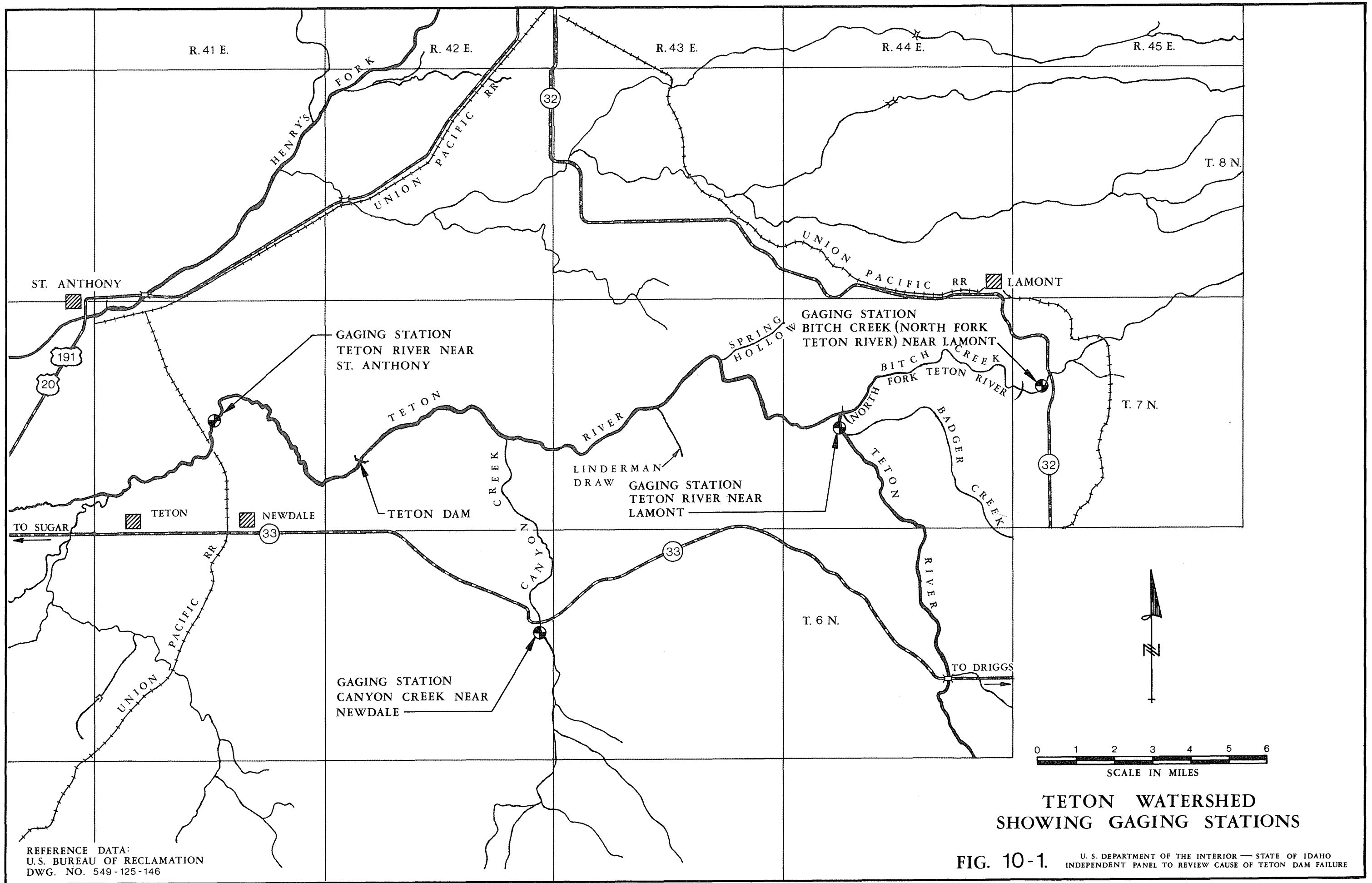
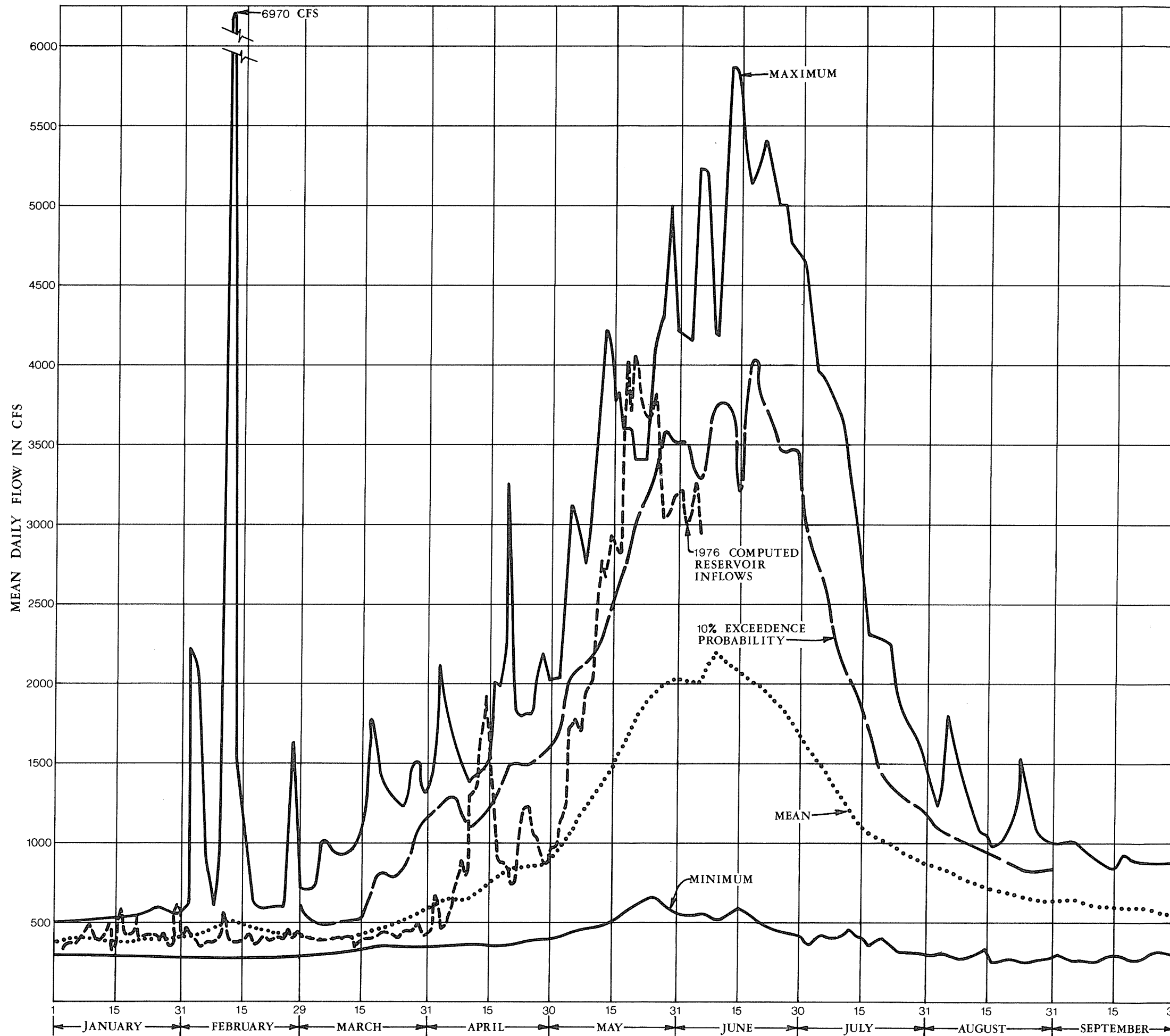


FIG. 10-1.



NOTE:

MAXIMUM LINE REPRESENTS
 MAXIMUM FLOW RECORDED
 FOR EACH DAY OF THE YEAR
 FOR THE PERIOD OF RECORD.
 10% LINE INDICATES FLOWS
 THAT WERE EXCEEDED 10%
 OF THE TIME.

RECORD:

1891 - 1893
 1903 - 1909
 1920 - 1973

REFERENCE DATA:
 DEPARTMENT OF WATER RESOURCES
 STATE OF IDAHO

**SUMMARY HYDROGRAPHS
 TETON RIVER NEAR ST. ANTHONY**

FIG. 10-2

U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
 INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

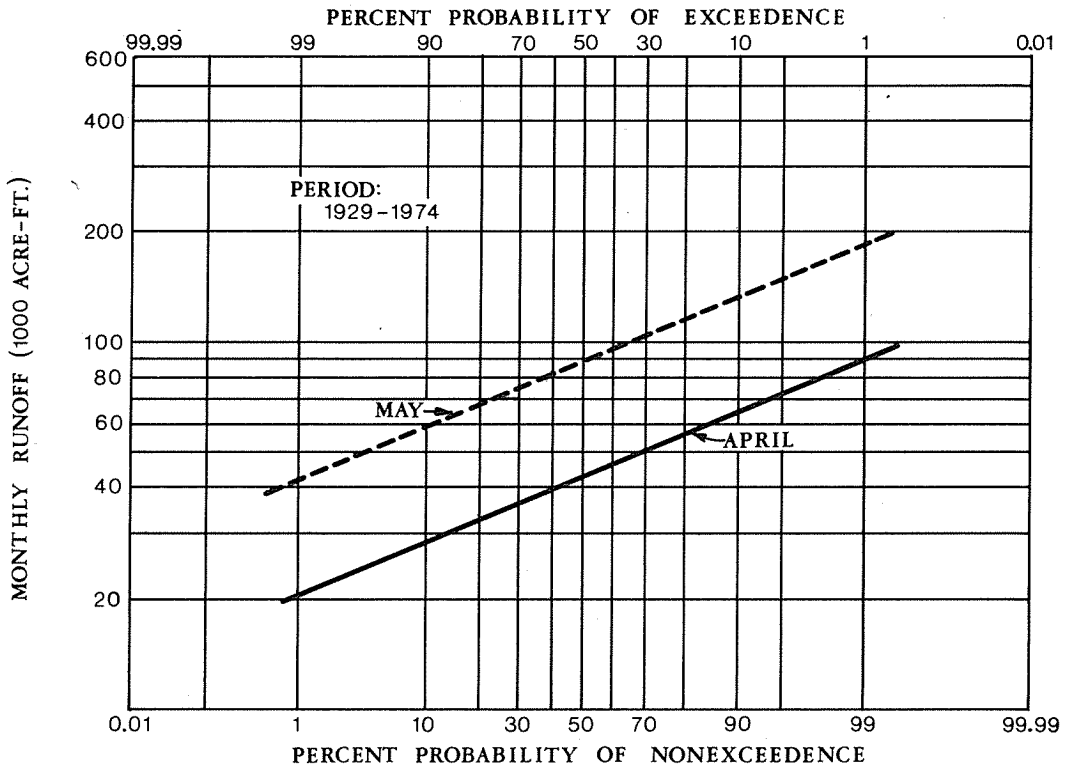


FIG. 10-3. FREQUENCY CURVES OF MONTHLY RUNOFF VOLUMES, TETON RIVER NEAR ST. ANTHONY

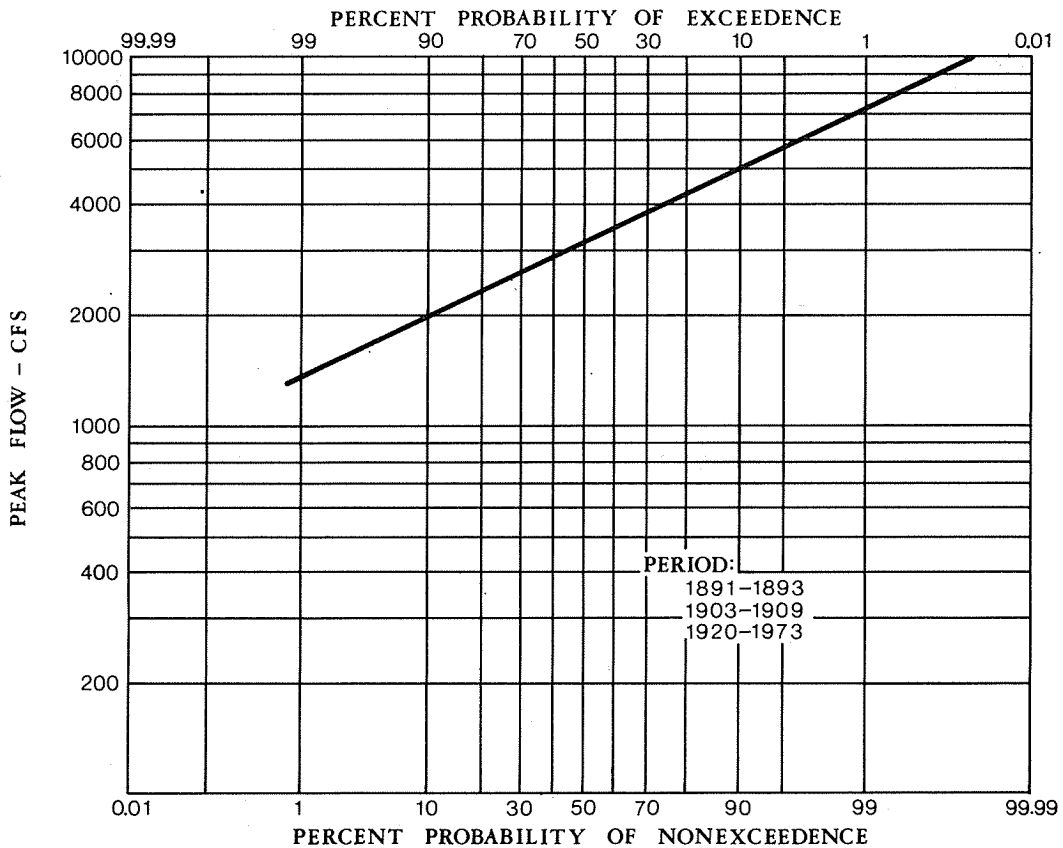
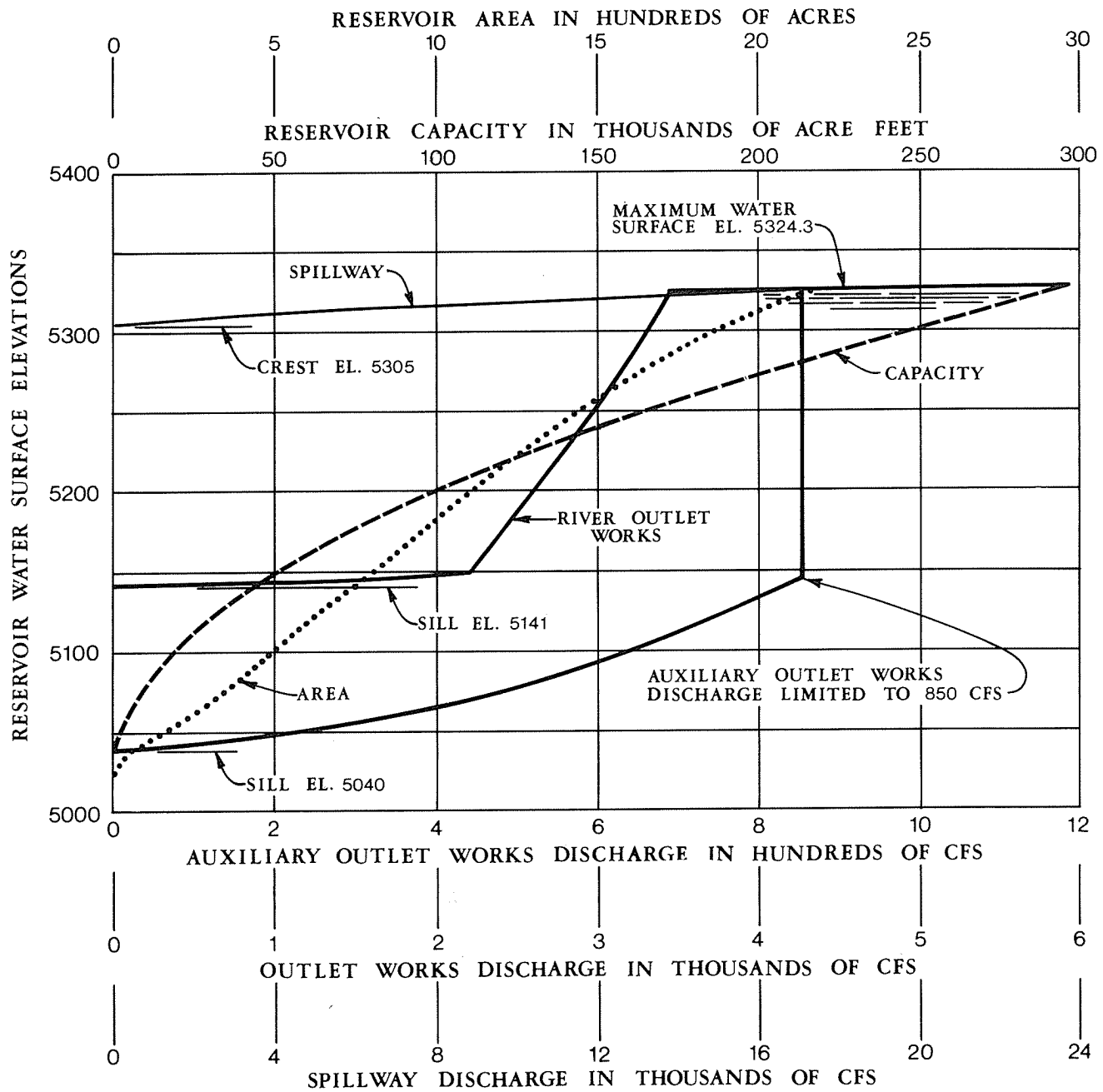


FIG. 10-4. FREQUENCY CURVE OF SPRING FLOOD PEAKS, TETON RIVER NEAR ST. ANTHONY

REFERENCE DATA:
DEPARTMENT OF WATER RESOURCES
STATE OF IDAHO

U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



AREA-CAPACITY-DISCHARGE CURVES

REFERENCE DATA:
 U. S. BUREAU OF RECLAMATION
 DWG. NO 549-D-8

FIG. 10-5. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
 INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

COMPARISON OF RESERVOIR FILLING RATES

Reservoir Filling Anticipated.

The following is taken from "Design Considerations for TETON DAM," by the U.S. Bureau of Reclamation, Denver, Colorado, October 1971:

The performance of the foundation, of the abutments, and of the embankment of Teton Dam during initial filling and reservoir operation is extremely important. Instructions for observing and reporting performance of the structure will be issued in "Designers' Operating Criteria." It is most likely, however, that initial filling or partial filling, and some reservoir operation will occur prior to issuance of the operating criteria. The instructions contained here are tentative and are applicable until such time as the final criteria are issued.

During the initial filling for periods when the reservoir surface is either rising or falling more than 1 foot per day, and for at least the first year of reservoir operation, frequent inspections of the embankment, of the abutments, and of foundation areas should be made to check for seepage or for significant rises in the water table downstream from the dam.

Measurable seepage should be collected and measured; seepage areas should be mapped and photographed; and reservoir water surface elevation and other pertinent data should be recorded. Generally, the results of the inspections should be reported monthly to the Director of Design and Construction. . . . Adverse seepage conditions may require more frequent reports, and any unusual developments noted during any inspection should be reported immediately. In this event further instructions will be furnished by the Director of Design and Construction. . . .

After the completion of the required portions of the river outlet works and the installation of protection for various parts of the work, the Teton River would be diverted through the river outlet works by the construction of the upstream cofferdam. A downstream cofferdam will also be required to exclude high tailwater from the powerplant area, tailrace, and spillway stilling basin.

In routing the 25-year spring flood through the diversion tunnel and channels as described above, the water surface in the reservoir would rise to elevation 5075 and the discharge would be 4,200 cfs. The 25-year spring flood has a peak of 5,000 cfs and a 15-day volume of 111,000 acre-feet. The tailwater in the 110-foot minimum downstream diversion channel would be at about elevation 5029 for a discharge of 4,200 cfs. A hydraulic jump will occur in the 19-foot-wide channel lining for a discharge of 4,200 cfs.

After the spillway, auxiliary outlet works, and dam embankment have been completed as required by the specifications, the river outlet works should be closed on October 1 of the final winter period. This should be accomplished by installing the intake bulkhead gate at elevation 5141, opening the 24-inch slide gate in the diversion inlet, and installing the diversion inlet stoplogs. The stoplogs were designed to seat in flowing water with a depth up to about 9 feet. Before the stoplogs are installed all rocks, gravel, and debris of all kinds must be removed from the seats and sill in order to give a watertight contact.

When the river outlet works is closed the auxiliary outlet works must be fully open and must be kept fully open until the river outlet works has been completed ready for service. . . .

After May 1 the flow in the Teton River exceeds the maximum allowable release from the auxiliary outlet works of 850 cfs, and the capacity of the river outlet works is needed in order to control the rate of filling in the reservoir.

It is anticipated that with the river outlet works completed for service by May 1, storage in the reservoir could be commenced. . . .

Unless adverse performance develops, unrestricted filling rates will be permitted to elevation 5200. Above elevation 5200 initial filling should not exceed 1 foot per day. . . .

This quotation shows that a flood of the magnitude of the actual spring flow of 1976 was anticipated in designing the outlet facilities. The possible consequences of operating with a single outlet during the spring runoff were known.

The following is an excerpt from U.S. Bureau of Reclamation letter dated August 4, 1976 (after the failure) from Acting Regional Director, Boise, Idaho, to Director of Design and Construction, Engineering & Research Center, Denver, Colorado, subject: Teton Forecast Information as Requested (Re: Faxogram Dated July 30, 1976):

Runoff volume forecasts of the inflow to Teton Reservoir were made as soon after the first of each month as data was available from January 1, 1976 to June 1, 1976. These forecasts [Table 10-1] and the flood control rule curve [Fig. 10-6] were used to determine what reservoir space would be needed to regulate downstream flows to 2500 c.f.s., the safe channel capacity. [Table 10-1] presents the volume forecasts, the flood space required by the rule curve on May 1, and the space available on the date of the forecast. [Fig. 10-6] is used to convert the forecast to a May 1 estimated residual on those months prior to May 1. Historically, runoff volumes of this magnitude would produce daily inflows of 4000-4500 c.f.s. per day for several days during the height of the snowmelt season. Anticipating that the main river outlets would be available during that period, it was estimated that the filling rate would be from 2 to 2.5 feet per day during the peak inflow. Had the main river outlets been available the fill rate prior to June 5 would have been limited to approximately 2 feet per day on or near the 18th of May, the time of peak inflow to the reservoir. . . .

Reservoir Filling Experienced.

Construction of the dam was started in February 1972. Initially, river flow was directed through the middle of the canyon so that excavation could proceed on both abutments.

Construction of the river outlet works tunnel was begun in June 1972. During construction, diversion was made through a channel at the right abutment. This channel permitted excavation of the cutoff trench to the left of the channel.

Diversion through the river outlet works was commenced on June 8, 1973. This diversion enabled construction of the cutoff trench on the right side and placement of embankment in the trench.

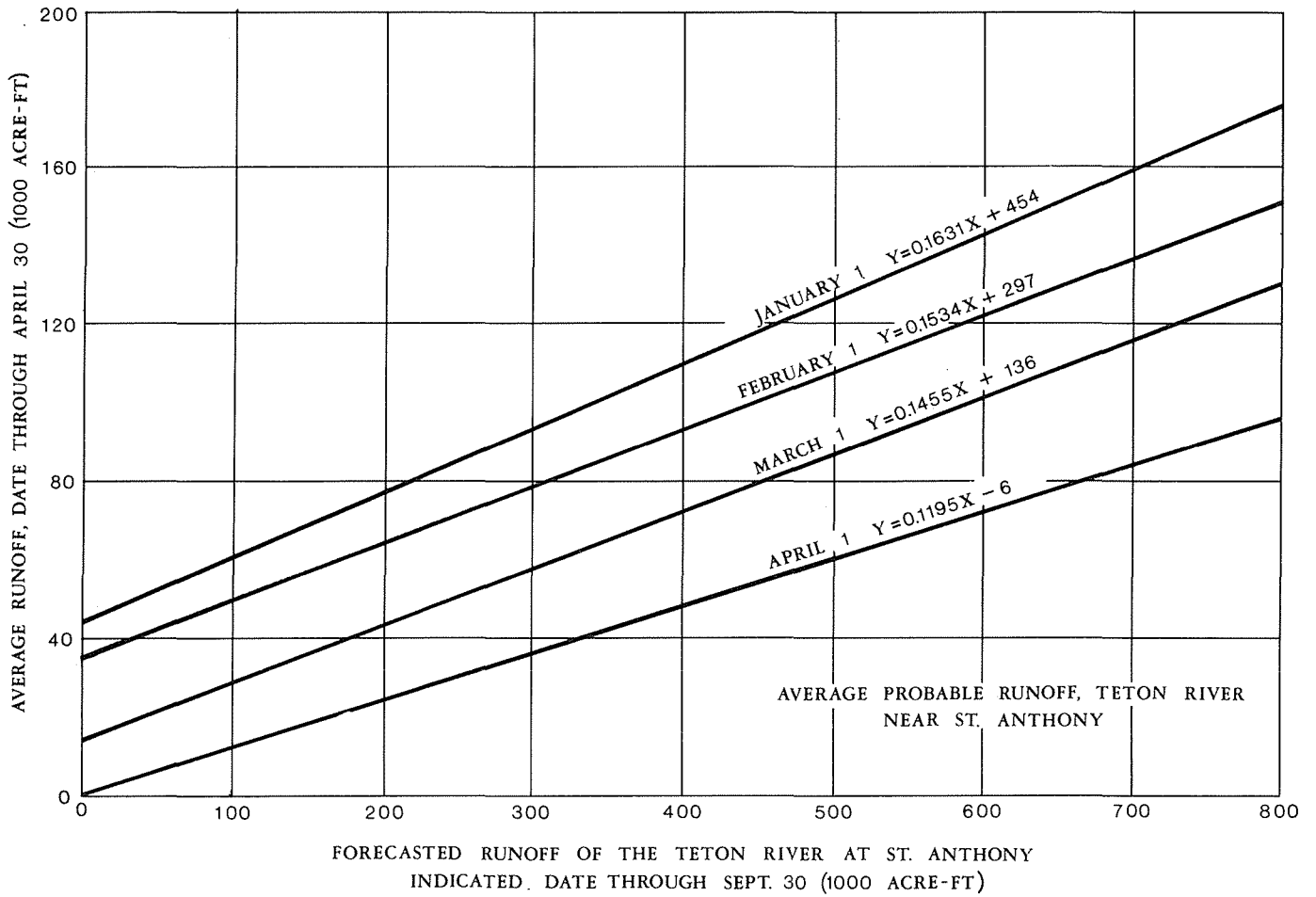
At the end of water year 1975 (October 1, 1974 through September 30, 1975), Teton River flows were at about the normal rates for that time of year. However, reservoir storage in the Snake River system was well above normal as a result of unusually high 1975 runoff. Runoff of the Teton River continued near normal through the fall and early winter months.

TABLE 10-1
RUNOFF VOLUME FORECASTS OF INFLOW
TO TETON RESERVOIR IN CALENDAR YEAR 1976
(Volumes in Thousands of Acre-ft)

<u>Forecasting Period</u>	<u>Forecasted Volume</u>	<u>% Normal</u>	<u>May-Sept. Est. Residual Forecast</u>	<u>Flood Control Space Required on May 1</u>	<u>Flood Control Space Available</u>
Jan.-Sept.	616	122	470	125	238.2
Feb.-Sept.	612	128	489	140	230.6
Mar.-Sept.	591	130	492	140	224.1
Apr.-Sept.	617	146	544	175	217.4
May-Sept.	584	157	584	200	175.2
June-Sept.	391	140		95 ¹	53.8

¹Flood control space required on June 1.

Reference: USBR Communication dated August 4, 1976.



REFERENCE DATA:
BUREAU OF RECLAMATION

FLOOD CONTROL RULE CURVE
FIG. 10-6

U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Diversion continued through the river outlet works until October 3, 1975 when the auxiliary outlet was put into operation to enable placement of second-stage concrete in the intake and gate chamber of the river outlet works and to install outlet gates, pipe, penstock manifold, and metalwork and equipment. From that date until the failure of the dam, all diversion was through the auxiliary outlet works. Chapter 9 discusses the details and schedules of outlet works construction.

Snow surveys in the Teton River watershed which were begun in January 1976 indicated heavy snow accumulations. Table 10-2 shows 1976 snow course measurements in percentages of normal for two Teton basin snow courses.

TABLE 10-2
Snow Water Equivalents as Percent of 1958-76 Averages

	<u>Pine Cr. Pass</u>	<u>State Line</u>
	(%)	(%)
Jan 1	191	191
Feb 1	142	129
Mar 1	141	141
Apr 1	141	136
May 1	193	233

As of March 1, 1976, the reservoir water surface was at El. 5164.7 ft and the auxiliary outlet was discharging 295 cfs. The reservoir level was rising about 0.2 ft per day.

A memorandum from the USBR Project Construction Engineer to the Director of Design and Construction dated March 3, 1976, when the reservoir stood at El. 5165.1, requested approval to exceed the 1-ft-per-day filling rate. Pertinent excerpts from the memorandum follow:

. . . have been monitoring the observation wells. . . . These observations show that there are several of the wells in which the water level is rising. Drill Hole No. 5 [No. 14 in Fig. 5-6] indicates a significant rise in the water table, but we do not feel that this well is truly representative of the area in which it is located. . . .

In addition to the well readings, daily inspections of the constructed works, including the auxiliary outlet works and the river outlet works access shafts, the river outlet works tunnel, and the area below the dam, are made. No leaks have been detected to date along the abutments or along the embankment downstream of the fill. However, moisture had become apparent on the walls of the river outlet works shaft up to elevation 5130.

There are no significant leaks to date and total leakage in the shaft is estimated to be ½ gallon per hour emitting through hairline cracks in the shaft lining. Water is leaking through the concrete lining upstream of the gate shaft in the river outlet works tunnel at approximately 2 gallons per minute.

In the auxiliary outlet works shaft, beginning on February 9, we have detected small leaks and they are presently apparent up to elevation 5067 with an estimated flow of ¼ gallon per hour. . . .

. . . request . . . approval to deviate from the 1 foot per day filling rate set forth in the design considerations. . . . feel this . . . desirable for the following reasons:

1. The reservoir filling curve and snow forecast information shows that if we do get permission for this deviation, we will be able to fill the reservoir this coming runoff period. This would allow our clearing contractor to sweep the reservoir and complete his work. This would make it possible to open the reservoir to the public for recreation in the summer of 1977.
2. The . . . construction schedule includes testing the turbines and generators. . . . A near full reservoir will permit tests at the higher operating heads.
3. Filling the reservoir . . . would enable us to observe the effectiveness of the curtain grouting.
4. A full reservoir would make it possible for full power generation this year subsequent to testing the generators.
5. We have in the past experienced flows considerably above that which we can release from the auxiliary outlet works during March and April. If we experience this again this year before the river outlet works is complete, we will not be able to maintain the recommended 1 foot per day filling rate.

In addition to reading the wells as we have in the past, we will continue to provide daily inspection of the downstream area of the dam; and upon melting of the ice on the reservoir, we will initiate a bi-weekly reservoir reconnaissance to detect any outflows into fissures or vents which might occur as the water rises behind the dam.

Our present releases from the reservoir average 300 cubic feet per second. From the available information on inflows, it appears that approximately 6 percent of the water is being lost either to seepage or to bank storage. . . .

On March 23, 1976, the Director of Design and Construction sent a memorandum to the Project Construction Engineer which stated:

The Design Considerations for Teton Dam, published in October 1971, restricted the fluctuation of the reservoir to 1 foot per day during the first year of operation to observe performance of the foundation and abutments of the dam. In May 1975, a program for monitoring ground-water conditions at the damsite and reservoir prior and during initial filling was established. This program, consisting of 19 observation wells, is superior to the normal monitoring program as it would give advance warning of the development of unusual ground-water conditions. A review of the ground-water monitoring from September 1975 to February 1976, contained in your referenced memorandum, indicates a predictable buildup of the ground-water table for Teton Dam and reservoir.

The preliminary reservoir filling curves developed for the 1976 runoff season indicate that the reservoir filling will exceed 1 foot per day with required releases only during the month of May when the maximum daily increase will be approximately 2 feet.

The normal development of the ground-water table to date and the well system being used for monitoring will allow relaxing the filling rate from 1 to 2 feet per day.

Daily inspection of the embankment, abutments, and foundation areas should be continued during filling.

Teton River flow in 1976 is shown in Fig. 10-2 for comparison with the historic river flows. These 1976 flows are identified as computed reservoir inflows and were determined by adding reservoir storage changes to outflows. This hydrograph differs from natural flows of the Teton River near St. Anthony by the amount of reservoir loss to bank storage.

Beginning in early April, river flows rose well above average and continued well above normal until the dam failure on June 5. During two periods, April 12-14 and May 17-23, the flows exceeded all previous flows for those dates. April and May 1976 runoff volumes (approximately 61,000 acre-ft and 170,000 acre-ft on a computed basis) are estimated to have 15 percent and 2 percent probabilities of exceedence (Fig. 10-3), respectively.

Storage commenced at the beginning of October 1975. The reservoir filled slowly and steadily until April 5, 1976, when increasing inflows accelerated the rate of fill. Fig. 10-7 shows the reservoir filling sequence from January 1 to June 5. Outflows were held constant at about 300 cfs until early May when they were increased to about 800 cfs and subsequently to a maximum of 963 cfs on May 28.

The 2-ft-per-day rate was exceeded on April 13, 14, and 15 during warm weather in the Teton River drainage area. The reservoir rises on these days were 2.6 ft, 3.1 ft, and 2.3 ft, in sequence. From April 16 through May 10, the 2-ft-per-day criterion was not exceeded except for a 2.1-ft rise on May 5. From May 11 until June 5, the 2-ft-per-day requirement was exceeded with an average daily rise of 3.0 ft and a maximum rise of 4.3 ft on May 18. During the period of May 12 to June 5, the auxiliary outlet was discharging at a rate higher than its capacity of 850 cfs.

On May 14, 1976 when the reservoir level stood at El. 5236.9, the USBR Project Construction Engineer sent a Faxogram to Director of Design and Construction, Denver, reporting current status of the reservoir filling and of the river outlet works construction. The following are excerpts:

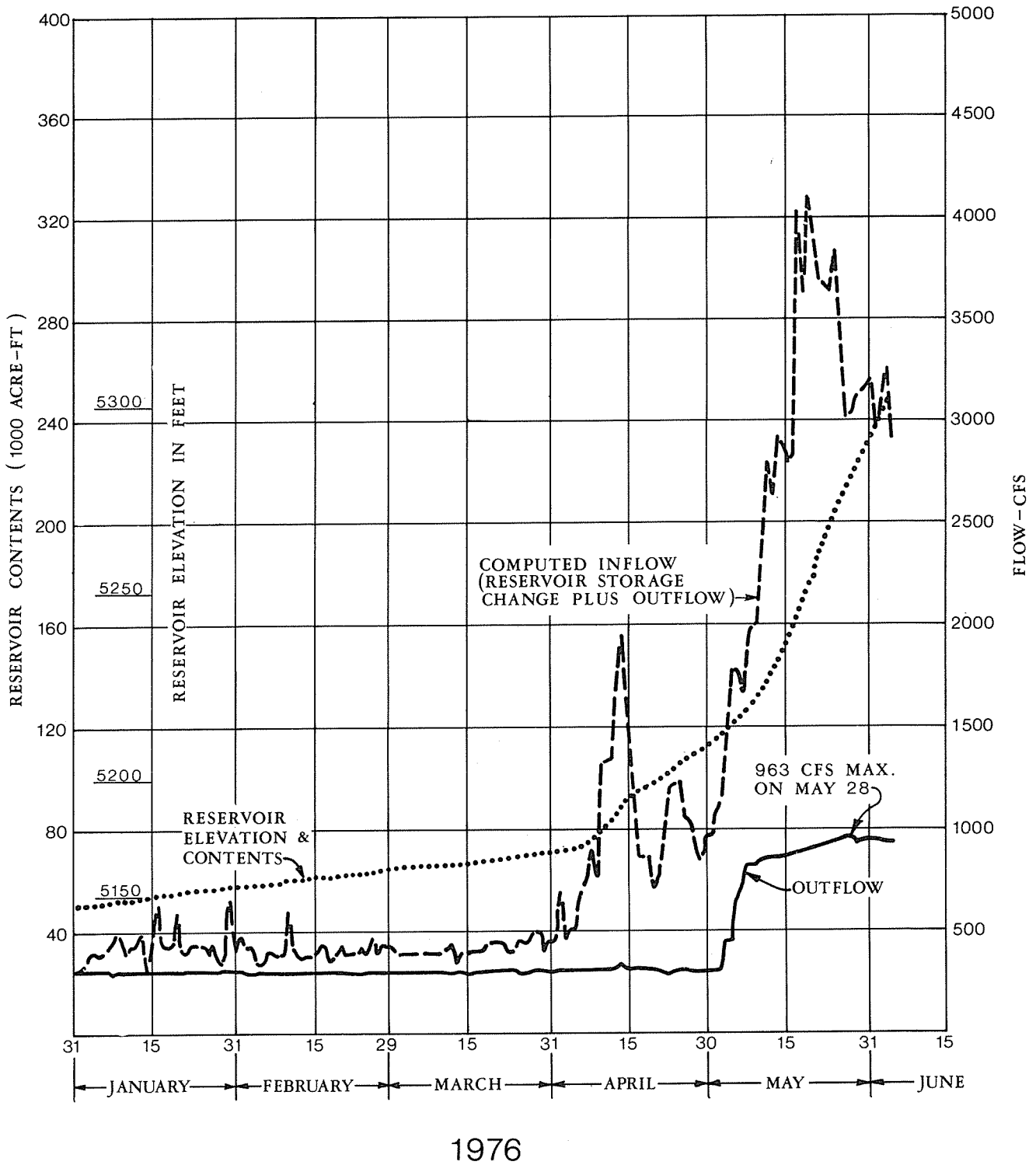
1. . . we do not expect that painting of the tunnel liner downstream of the wheel-mounted gate will be completed before June 10, 1976. Further acceleration of the painting to enable earlier opening of the river outlet works is not feasible.
2. The spillway gates are in place but not fully operational. Completion . . . for possible control of the reservoir at capacity is not expected before June 1, 1976.
3. . . Should the need for any water release through the river outlet works become imperative before completion of the painting, the resultant interruption would involve claims by the contractor for delay and additional cleanup and sandblasting. . .

We request your comments for flood control operation.

Response to this message is dated June 4, 1976. The message was in the mails on the day of failure and said in part:

We . . . conclude that the river outlet works need not be used prior to completion of the painting unless problems directly related to filling of the reservoir develop in the foundation, embankment or structures. It is imperative however that both the spillway and river outlet works be made operational as soon as possible.

In a statement before the Subcommittee on Conservation, Energy and Natural Resources of the Congressional Committee on Government Operations, on August 5, 1976, R.R. Robison, Project Construction Engineer, said:



TETON RESERVOIR INFLOW, OUTFLOW, AND CONTENTS, JANUARY 1 TO JUNE 5, 1976

FIG 10-7 U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

REFERENCE DATA:
DEPARTMENT OF WATER RESOURCES
STATE OF IDAHO

During the period from October 3, 1975, to May 3, 1976, releases through the auxiliary outlet works were limited to required downstream flows, 300 ft³/s, and all flows in excess of 300 ft³/s were taken in storage resulting in a reservoir depth of 185 feet. From May 4 through May 11, 1976, the flow through the auxiliary outlet was gradually increased to 850 ft³/s. From May 4, 1976, until the time of dam failure, the auxiliary outlet works was operated at or above its design capacity of 850 ft³/s.

On March 3, 1976, I requested that the filling rate be relaxed – at this time, there was a depth of 135 feet in the reservoir (Elevation 5170). On March 23, 1976, I was given permission to increase the filling rate to 2 feet per day because of the normal development of the ground water table at that time and no springs had developed below the dam, indicating the filling was not causing problems. Results of the monitoring of the ground water conditions received by the Engineering and Research Center for the period through May 13, 1976, did not indicate a radical change in the water level in the wells inconsistent with the rise in reservoir elevations, and no springs had developed. The decision was then made to fill the reservoir to the spillway. (The rapid rise of the right abutment wells subsequent to that date was not viewed as indicating emergency conditions inasmuch as the left abutment wells downstream from the dam were not affected, so the readings were transmitted routinely and reached the E&R Center after the failure.)

H.G. Arthur, Director of Design and Construction, stated before the same Subcommittee on August 6, 1976:

With regard to the filling of the reservoir, discussed yesterday by Mr. Robison, the design considerations required that above elevation 5200, the reservoir was not to be filled faster than 1 foot per day. This criterion has been used for many years by the Bureau for *initial* filling of reservoirs and it is considered a desirable rate for testing the abutments and the embankment. With structures available to control to this rate, time would be available to take remedial measures if problems developed. If desirable for project purposes, the initial rate is exceeded when the dam performs satisfactorily.

Exceeding the initial filling criterion at Teton Dam was not an unusual procedure. Initial reservoir filling criteria were increased in the initial filling of 36 instances on Bureau reservoirs. In eight instances, the reservoir filling rates experienced exceeded the maximum rate of 4.3 feet which occurred during filling of Teton Reservoir. No problems were encountered in any instance due to relaxation of the initial reservoir filling criterion.

Hypothetical Filling Rate.

The first day in the spring of 1976 when the actual rise in reservoir exceeded one foot per day was April 8. The water level on the preceding day had been recorded at El. 5175.5. Assuming hypothetically that both outlets had been fully operable and that a maximum rise of 1 ft per day had been observed each day thereafter, the reservoir water surface would have risen to about El. 5234 ft by June 5, which was 59 days later.

The USBR criterion actually allowed unlimited filling rates up to El. 5200, which was reached on April 22. If it had been possible to adhere to the 1-ft-per-day rule after this date, the reservoir would have risen to El. 5244 by June 5, as compared with El. 5301.7 measured just prior to failure.

The maximum inflow rates occurred in the period May 18-21, inclusive, with flows of 4044 cfs, 3664 cfs, 4111 cfs, and 3947 cfs, respectively. The combined capacity of the two outlet works for these

reservoir levels would have been about 3600 cfs which would have been more than adequate to maintain the 1-ft-per-day limit in reservoir rise. Under such an hypothetical operating regimen the safe downstream channel capacity, estimated by the USBR at 2500 cfs, would have been exceeded by as much as 30 percent for short periods.

COMMENTS

The 1976 spring flow was within the probabilities considered in design. Runoff forecasts based on snow surveys gave warning that river flows would be of such above-normal volume that both outlets would be required to hold the reservoir rise to the prescribed rate. Actual flows compared closely with these forecasts.

The river outlet works and auxiliary outlet works of Teton Dam were designed with a total capacity of 4250 cfs at maximum water level. However, even though the approved construction schedule required construction to be completed by March 31, 1976, only the auxiliary outlet works were in operation through June 5, 1976. This resulted in virtual non-control of the reservoir filling rate during the late spring of 1976.

The records of Teton River hydrology were well known to the Bureau of Reclamation. The design criteria recognized that it would be necessary to have the river outlet works in operation after May 1, 1976 in order to control the rate of filling so as not to exceed a 1-ft-per-day increase when the reservoir surface elevation was above 5200 ft. This design fill rate was relaxed to 2 ft per day on March 23, 1976, but the new rate was exceeded on three days in April and during the entire period from May 11 to June 5.

If both outlets had been operable on March 31, 1976, as required by the specifications, their combined discharge capacities would have been enough to control reservoir filling to the originally prescribed rate after that date. However, because information on the internal condition of the dam and its foundation was minimal, there can be no assurance that Project staff would have been able to see any reason to modify the March 23 operating plan.

The paucity of instrumentation and the decision to allow an increased rate of filling had no demonstrable influence on the failure. The short time within which the chain of events occurred that culminated in the catastrophe suggests that there would have been insufficient reaction time to take advantage of instrumental warnings. Nevertheless, the possibility exists that a more conservative approach to instrumentation and rate of filling could have averted the disaster. Had the rate of filling not exceeded 1 ft per day, and had foundation piezometers been located downstream of the cutoff, the piezometers might have given early warning of rapidly rising piezometric levels while the hydraulic gradients causing erosion were relatively small. Time would then have been available for lowering the pool and investigating the phenomena. It is equally possible, however, that the slower rate of filling would only have delayed the date of the failure.

CHAPTER 11
MEASURES TAKEN TO MONITOR SAFETY OF DAM
(Panel Charge No. 10)

SURVEILLANCE PLAN

Procedures.

Instructions for observing and reporting performance of the structures at the Teton damsite were to be issued in the "Designers' Operating Criteria." It was seen as likely that some reservoir operation would occur prior to issuance of these guidelines. Tentative instructions provided in the "Design Considerations for Teton Dam," USBR, October 1971, were in effect until such time as the final criteria were issued. Those criteria had not been issued at the time of the Teton Dam failure.

As stated in the preceding chapter, during the initial filling for periods when the reservoir surface was rising or falling more than 1 ft per day, and for at least the first year of reservoir operation, frequent inspections of the embankment, of the abutments, and of foundation areas were to be made to check for seepage or for significant rises in the water table downstream from the dam.

These tentative instructions of 1971 also required that measurable seepage should be collected and measured; seepage areas should be mapped and photographed; and reservoir water surface elevation and other pertinent data should be recorded, and that the results of the inspections should be reported monthly to the Director of Design and Construction. Any unusual developments noted during any inspections were to be reported immediately. In such an event, further instructions were to be furnished by the Director of Design and Construction.

Assignment of Responsibility.

The Project Construction Engineer had primary responsibility for surveillance of the dam and reservoir. He was required to report unusual conditions to the Director of Design and Construction in Denver. As reservoir filling began the Project Construction Engineer gave general instructions to members of the field forces to be alert for any adverse developments.

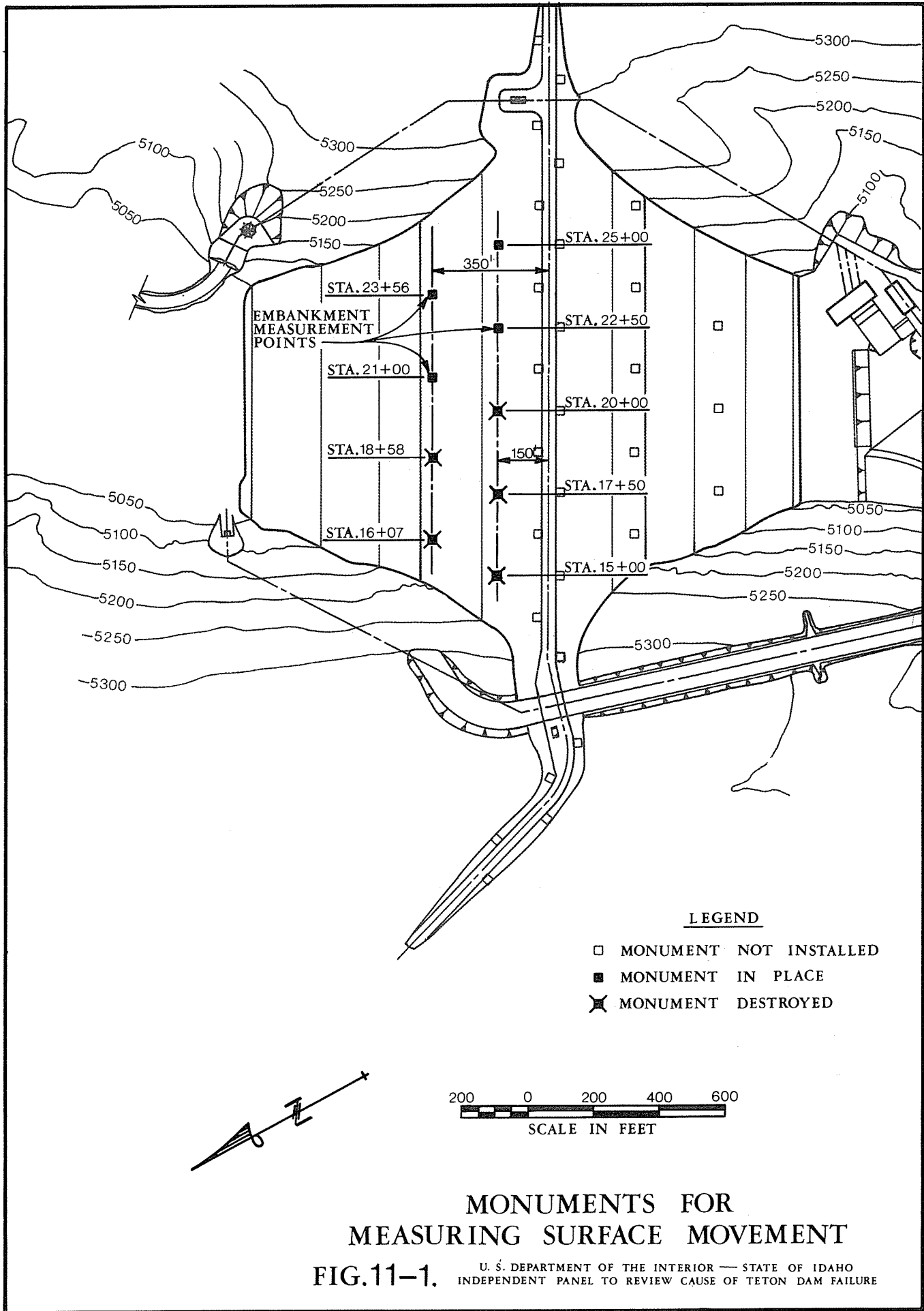
INSTRUMENTATION

Monuments.

Measurement points were to be installed by the contractor in rows parallel to the dam axis upon completion of the outer surfaces of the dam embankment to an elevation 10 ft above each of the measurement points. Planned spacing in each row was approximately 250 ft as shown in Fig. 11-1 with numbers and elevations as shown in the table below:

<u>Location</u>	<u>Approximate Elevation</u>	<u>Between Stations</u>	<u>Number of Points</u>
350 ft upstream	5199	16+07 and 23+56	4
150 ft upstream	5279	15+00 and 25+00	5
22.5 ft upstream	*	6+25 and 31+25	10
22.5 ft downstream	*	5+00 and 30+00	11
250 ft downstream	5215.75	16+25 and 26+25	5
500 ft downstream	5127.17	17+50 and 22+50	3
Total			38

*Camber determines elevation



**MONUMENTS FOR
MEASURING SURFACE MOVEMENT**

FIG. 11-1. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Under his contract the contractor was required to furnish materials for and to place the embankment measurement points where shown, or as designated by the Government.

The Government was to assist the contractor by determining the approximate locations for the measurement points and on completion of each row of points was to establish them as benchmarks for elevations and targets for horizontal control. Periodic Government surveys to 0.01 ft on these points were to be started for the measurement of cumulative settlement and for horizontal deflection of each with respect to the centerline of crest. Further details of the embankment measurement point installation are described in Designation E-32 of the USBR *Earth Manual*, first edition, revised 1968.

On June 5, 1976, only nine of the upstream points had been installed and were being monitored. Five of these points were destroyed when the dam failed. As shown in Fig. 11-1, none of the downstream points were placed.

Some settlement was expected along the spillway walls. To determine the magnitude of such settlement and any deflection of the walls, readings were obtained on measurement points as soon as the structures were constructed, prior to backfill placement, and periodically as construction progressed. As previously stated in Chapter 7, no movement was significant.

Flow Measuring Facilities.

Devices such as weirs to measure seepage flows downstream from the dam had been planned but had not yet been constructed at the time of failure. The reported flows were estimated from visual observation.

Groundwater Measuring Devices.

Nineteen exploratory borings located in a large region surrounding the dam were used as observation wells (Fig. 11-2). These holes, however, were not specifically located for the purpose of surveillance of the dam. They did serve to indicate rise in water level in the area in the vicinity of the reservoir, but their value in monitoring the safety of the dam was incidental and minimal.

Abutment Seepage Instruments.

No provision was made for monitoring seepage flows inside the abutments.

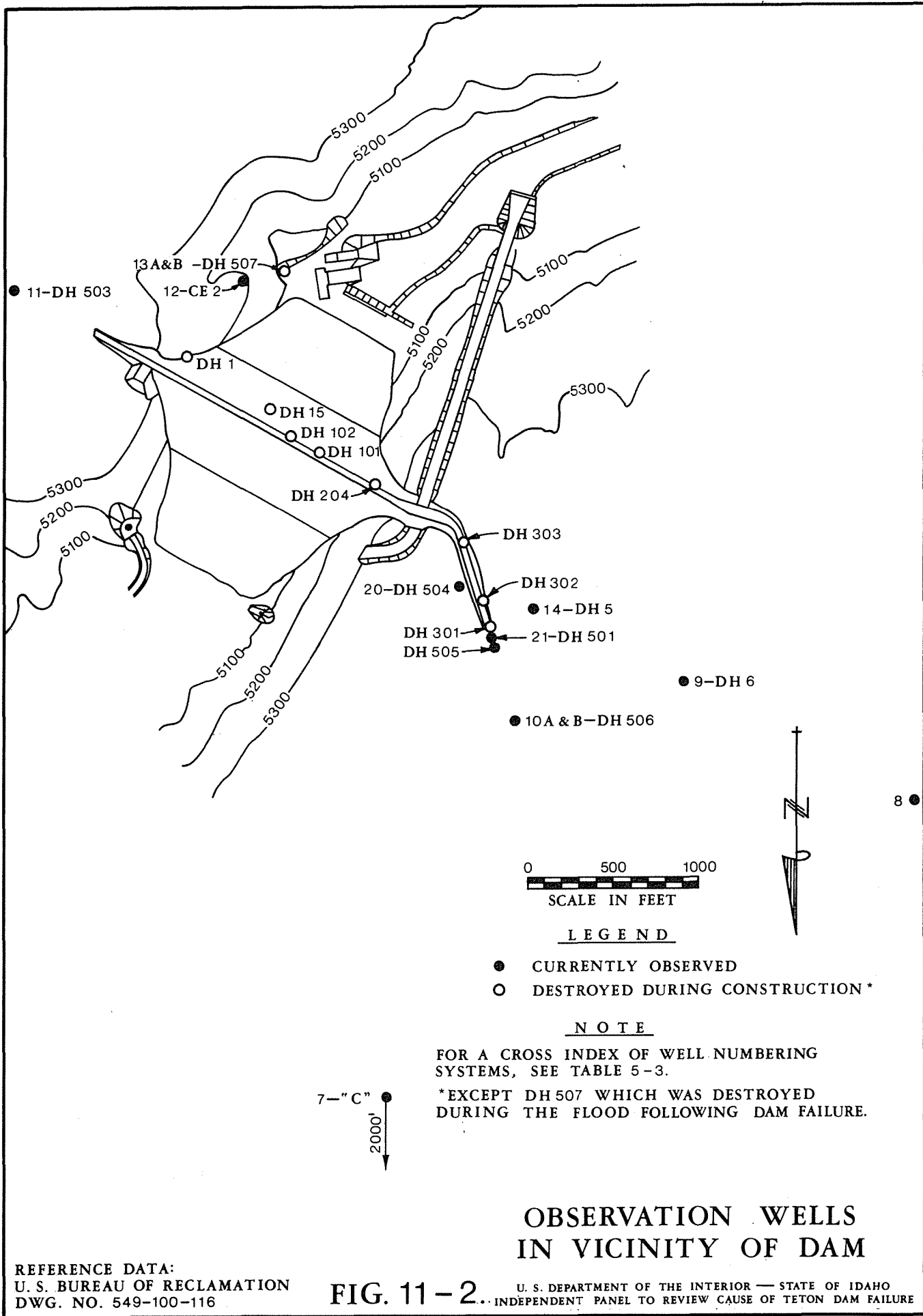
Strong Motion Instruments.

The U.S. Geological Survey had a strong motion accelerometer installed in the Teton Dam powerhouse which was destroyed at the time of failure. For other seismometer installations refer to Chapter 6.

MEASUREMENTS

The water table in the right abutment was observed through drill holes and wells for several years before construction of the dam. In May 1975 a program for measuring groundwater conditions at the damsite and reservoir prior to and during initial filling was established. This program was expected by the USBR to give advance warning of the development of unusual groundwater conditions. Readings of water levels in the wells were taken weekly until the spring of 1976, when the frequency of readings was increased to about twice a week. The results are discussed in Chapter 5.

Surveys were made periodically of horizontal and vertical movement of the surface monuments which had been installed on the dam. The results are shown in Table 11-1.



REFERENCE DATA:
 U. S. BUREAU OF RECLAMATION
 DWG. NO. 549-100-116

FIG. 11-2.

OBSERVATION WELLS
 IN VICINITY OF DAM

U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
 INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

**TABLE 11-1
MEASUREMENTS OF MONUMENTS ON DAM**

Reservoir Water Surface Elevation	Date	Centerline Station								
		15+00	16+07	17+50	18+58	20+00	21+00	22+50	23+56	25+00
Elevation in Feet										
5050±	10/2/75*		5197.82		5197.07		5198.00		5198.97	
5119.0	11/21/75	5278.97*	5197.77	5279.58*	5197.01	5279.21*	5197.94	5279.33*	5198.92	5278.90*
5162.7	2/23/76	5278.91	5197.77	5279.47	5197.00	5279.07	5197.93	5279.22	5198.92	5278.85
5171.4	3/29/76	5278.92	5197.78	5279.47	5197.01	5279.06	5197.94	5279.21	5198.92	5278.84
5249.2	5/18/76	5278.90		5279.44		5279.04		5279.17		5278.82
5056±	6/19/76						5197.78	5278.98	5198.87	5278.75
Offset Upstream from Centerline in Feet										
5050±	10/2/75*		350.23		349.91		350.01		350.00	
5074±	10/18/75		350.26		349.93		350.04		350.03	
5119.0	11/21/75*	150.00		150.00		149.91		149.95		149.68
5162.7	2/23/76	149.99	350.23	149.95	349.88	149.88	350.01	149.02	350.00	149.73
5171.4	3/29/76	150.02		149.93		149.86		149.89		149.67
5171.5	3/30/76		350.25		349.88		350.01		350.02	
5261.0	5/21/76	149.99		149.96		149.92		149.99		149.83
5056±	6/22/76						350.18	149.78	350.29	149.69

*Original

See Fig. 11-1 for location of points.

INSPECTION

Daily inspection was made of the constructed works, including the auxiliary outlet works and the river outlet works access shafts, the river outlet works tunnel, and the area below the dam. Patrol examination was made of the abutments and the canyon walls downstream in search of leakage.

Surveillance inspections were made during reservoir filling as a part of the regular duties of the inspectors. In addition, the project photographer who recorded the observation well readings also inspected the abutments at least twice each week.

A daily report dated August 2, 1976 by Inspector Lyman G. Rogers, USBR, states:

As construction work started up this spring and reservoir was filling, Inspector Gary Larson was instructed to check for leaks in A.O.W. shaft and sides of spillway and spillway drains. Inspector Frank Emrick was instructed to check for leaks in R.O.W. shaft. Inspectors Alfred Stites and Ken Hoyt were instructed to check for leaks along left and right abutments downstream of face of dam. This work was done along with their regular inspection duties. I also checked along the abutments at least twice a week.

In a memorandum of November 9, 1976 to the Project Construction Engineer from Field Engineer Peter P. Aberle, the following description of surveillance procedures was given (Appendix B, under letter to Robert Jansen dated November 12, 1976):

... The area downstream of the spillway area was observed from across the river on a daily basis by the inspection forces and myself and leaks of any consequence could be detected by watching for water flows from the drain downstream of the spillway along the right abutment into the river. All inspectors were instructed to be aware for leakage and to report these leaks immediately.

During the month of May, the contractor (MK-K) cut a small hole into a water storage pond which was located high on the right abutment for the purpose of draining it. Water from this pond drained into the gully located to the right of the spillway. This water was detected almost immediately by the inspection forces and reported which shows the awareness of the program.

About ten days prior to June 4, I received a call from Mr. Duane Buckert, Project Manager for MK-K, stating that their Office Engineer, Vince Poxleitner, thought he saw a leak downstream of the spillway. This was checked out by the inspection forces and found to be negative.

After well no. 6 [DH-6] showed an exceedingly rapid increase of the water level, I made an inspection of the right abutment about 1200 to 1700 feet downstream of the dam and the gully in this area. This inspection was made on or about June 1, 1976, and no leaks were noted.

On Thursday, June 3, 1976, when the two leaks were found downstream of the spillway, I checked along the canyon downstream of these leaks an additional 500 feet and found no leaks.

On the morning of June 5, as I drove to the powerhouse area, I again visually checked the spillway drains and the gully to the right of the spillway and saw no leaks.

At least once a week I instructed the shift inspector to remind all inspectors to watch daily for possible leaks. These reminders were also made by myself several times during the weekly safety meeting held by the inspection forces each Monday morning.

As soon as the ice cleared from the reservoir area, the reservoir was inspected two to three times per week. The shoreline was patrolled near the damsite and potential land slides were noted and reported throughout the reservoir area. . . .

After the failure, Aberle reported that the small springs in the right abutment downstream from the dam "warranted monitoring by visual observation as frequently as routine inspections of the entire operation at the dam."

Such inspections were made. On the morning of June 4, 1976, for example, engineer W.H. Andrew and inspector A.D. Stites walked around the right abutment area at the toe of the dam looking for leaks. Andrew reported that they were doing this because "one or two spring leaks had developed" farther down the stream in the abutment wall "about the day before."

Later on June 4, until dark, inspector Stephen Elenberger made several observations of "both the downstream side and the upstream reservoir." He says that he had been alerted to pay particular attention for possible leaks because there were small spring-like areas of water on the north side of the canyon below the toe of the dam.

Regarding the seepage downstream in the north abutment wall, Project Construction Engineer R.R. Robison says that "I felt the area should be monitored by sight inspections and other mechanical means, the latter of which were never put into effect." Robison inspected this seepage late in the afternoon of June 4, 1976 and examined the dam itself, both upstream and downstream.

ANALYSIS

The need for monitoring and analyzing water levels in wells and drill holes at and near the damsite was apparently recognized by the Bureau of Reclamation, and attention was given to study of the data from this effort. The procedures for processing of well observation data were transmitted in a memorandum attached to a letter to Robert Jansen dated November 12, 1976:

. . . It was our intent that the readings should be compiled and forwarded to the Regional Office and Denver office personnel interested in this data at least once each month. Subsequent to the water being stored in the reservoir, Mr. [Keith] Rogers would receive the data and plot them on the charts as they were submitted in the field. I would review them periodically at least once each week to see if there was any significant changes in the water level shown. Approximately once each month we would assign a cutoff date that the charts would be brought up to date, the readings would be recorded and the data sent off to the various offices. The reservoir started filling so rapidly in the spring that recordings were read at more frequent intervals. Periodically we would meet with Mr. Robison and would look over the readings and discuss the changes noted. The last month before the dam failure, we noted a significant rise in water level shown on these readings. This was discussed with representatives of Director of Design and Construction, Denver. It was decided to make a special effort at this time to compile the data from the observation wells in order to forward them at a closer interval than in the past. . . .

As can be seen in Fig. 11-2, two observation wells were located on the left abutment downstream from the dam and one was nearby in the canyon bottom. However, there was only one drill hole downstream from the dam axis on the right abutment to serve as an observation well. While more holes in this vicinity would have facilitated analysis, the Bureau regarded the monitoring system as adequate.

REPORTING

Surveillance activity was conducted as part of the general inspection program, and any observations related to safety of the project structures were made on that basis. Adverse developments, however, were reported separately from the routine inspection reports. Construction personnel were instructed to make oral reports to their supervisors if they observed any questionable conditions at the dam.

The Project Construction Engineer was expected to make monthly inspection reports to the Director of Design and Construction in Denver. Unusual observations were to be reported immediately. Project Construction Engineer Robison did this either by telephone, faxogram, or mail.

COMMENTS

For a dam of this size and complexity, facilities for measurement usually would include surface monuments for gaging vertical and horizontal movement, cross-arm settlement devices and/or slope indicators to measure internal embankment movement, piezometers to monitor water pressures within the fill and in its foundation, weirs or other devices to measure seepage, wells for observation of water levels in the reservoir environs, and instruments such as accelerometers to measure earth tremors.

Inspectors responsible for visual observation should be provided with standard operating instructions to guide them in their regular patrols. These observers would be trained to interpret potentially adverse conditions and to report significant findings promptly. The assignment of responsibility should be well documented and well understood by all concerned personnel.

Several key members of the construction force made inspections on a regular basis, supplementing their construction assignments. Definition of responsibilities was apparently adequate. Information from pertinent observations was communicated reliably through the chain of supervision. The major deficiency in the surveillance program was in instrumentation. While considerable attention was given to well measurement in the vicinity of the dam, most of the wells were too far away to give direct indication of its performance. None of the wells was located specifically to monitor behavior of the embankment or its abutments. Without good instrumentation in the foundation and with none inside the dam itself, observers at the site were limited in the judgments that they could make related to safety.

In summary, project staff complied sensibly with preliminary operating criteria specified by USBR designers. Most instructions given at the site were oral, but it is believed that they were followed conscientiously. However, the dam and its foundation were not instrumented sufficiently to enable the Project Construction Engineer and his forces to be informed fully of the changing conditions in the embankment and its abutments.

CHAPTER 12 CAUSE OF FAILURE

REVIEW OF SURFACE MANIFESTATIONS

Any satisfactory explanation of the failure must be in accordance with the known chronology and eyewitness accounts. The facts are summarized as follows:

Before June 3, no springs or other signs of increased seepage were noticed downstream of the dam. On June 3, clear-water springs appeared at distances of about 1300 and 1500 ft downstream, issuing from joints in the rock of the right bank.

During the night of June 4, water may have flowed down the right groin from about El. 5200, inasmuch as a shallow damp channel was noticed early on the morning of June 5. Shortly after 7:00 a.m. when the first observations were made on June 5, muddy water was flowing at about 20 to 30 cfs from talus on the right abutment at about El. 5045, and a small trickle of turbid water was flowing from the right abutment at El. 5200. Both flows were at the junction of the embankment and the abutment, referred to as the groin, and both increased noticeably in the following three hours.

At about 10:30 a.m. a large leak of about 15 cfs appeared on the face of the embankment, possibly associated with a "loud burst" heard at that time, at El. 5200, about 15 ft from the abutment and adjacent to the smaller leak previously observed at the same elevation. The new leak increased and appeared to emerge from a "tunnel" about 6 ft in diameter, roughly perpendicular to the dam axis approximately opposite dam axis Sta. 15+25, and extending at least 35 ft into the embankment. The tunnel became an erosion gully developing headward up the embankment and curving toward the abutment, as shown in Fig. 2-1 and in the photographs, Figs. 2-11 and 2-13.

At about 11:00 a.m., a vortex appeared in the reservoir at about Sta. 14+00, as shown in Fig. 2-1, above the upstream slope of the embankment. At 11:30 a.m., a small sinkhole appeared temporarily, ahead of the gully developing on the downstream slope, near the crest of the dam. Shortly thereafter, at 11:55 a.m., the crest of the dam began to collapse at a point between the vortex and the head of the enlarging gully (Fig. 2-14). The failure then continued as a simple enlargement of the discharge channel by the reservoir.

From the time observers arrived at the site and first observed the small muddy flows, to the breaching of the dam, was about five hours. If it is assumed that such flows began on June 4, immediately after Inspector Elenberger's last visit at about 9 p.m., the surface manifestations of the developing erosion channel could not have existed more than 15 hours before the final breaching of the dam.

No other pre-failure observations are known except for the rise in water levels in various drill holes being used as observation wells. This information is shown in Fig. 5-7.

PHYSICAL CONDITIONS ALONG FAILURE PATH

Introduction.

The path along which the erosion developed is defined in plan with a considerable degree of certainty by the surface manifestations described previously. The subsurface conditions along this path played a vital part in determining the nature of the failure. In the following comments, the term "failure

section" is frequently used. This term is intended to refer to that section of the dam generally between dam axis Stas. 13+50 and 15+25.

Conditions Upstream from Grout Curtain and Key Trench.

The open-jointed nature of the welded tuff in the right abutment, both upstream and downstream of the axis of the dam, is described in detail in Chapter 5. In the early stages of design, during the test grouting program, it was concluded that the upper 70 ft of the rock on both abutments was too open for successful grouting; consequently the key-trench design was adopted. The open nature of the joints on the upstream face of the right abutment key trench was confirmed by the Panel's investigation after removal of the key-trench fill. At the failure section, part of the abutment rock has been removed by the erosive action of the escaping floodwaters. There is no reason to believe that the eroded rock was less open-jointed than that which remains. Hence, there can be no doubt that reservoir water had ready access to the entire upstream face of the key trench, including the portion adjacent to the failure section.

Beneath the level of the base of the key trench, the rock was also jointed and permeable, as judged by the water tests in exploratory drill holes and by the grout takes in the curtain. Since the curtain was confined to the key trench, there is no doubt that the rock at depth, upstream of the curtain, was permeable, albeit possibly less so than that in the upper 70 ft. Inspection of the face of rock remaining along the right abutment after the erosion by the escaping floodwaters disclosed many open joints below key-trench level, some partly filled with grout, both upstream and downstream of the grout curtain.

Conditions at Key Trench.

The geometry of the key-trench excavation is shown in Figs. 3-1, 3-3, and 3-13. In addition to the steep sides of the general excavation, many local irregularities are present. These include near-vertical faces and occasional overhangs.

The geometry of the local steep faces and overhangs is related to the jointing. Concentrations of joints, largely trending N30°W, exist between Stas. 13 + 00 and 13 + 50, and in the vicinity of Sta. 14 + 00 where erosion has removed the grout cap and exposed the rock beneath. The jointing is described in Chapters 3 and 5 and in Appendix E.

The grout curtain and grout cap have been described in detail in Chapters 8 and 9. The ponding or joint transmissibility tests conducted by the Panel, described in Chapter 3, demonstrated that water can flow readily beneath the grout cap at several locations near the failure zone. Water tests in drill holes on the center line of the grout curtain near the failure section also demonstrated the existence of passages through which water emerged downstream.

The key-trench fill was investigated extensively as the remnant on the right abutment was excavated. The material, as indicated in the specifications, consisted of windblown clayey silts. As described in Chapters 7 and 9, it was compacted generally on the dry side of optimum, contained occasional lenses or layers more plastic than the rest, and was placed against the rock walls of the key trench with no rock treatment or transition. Loose zones were noted beneath occasional overhangs or against open joints.

Conditions Downstream from Key Trench.

The jointing in the rock exposed in the downstream face of the key trench appears generally less prominent and open than upstream, except in the vicinity of Sta. 14+00 where several sets of major, throughgoing joints are apparent. These joints are located just to the right of the mass of rock eroded by the floodwaters; others undoubtedly existed within the eroded mass.

The right abutment was originally partly covered with products of rock weathering and accumulations of talus. These materials were removed where Zone 1 was in contact with the abutment, but not at the foundation contact with Zones 2 and 5. As described in Chapter 9, pervious talus existed beneath the groin of the dam on the right abutment downstream of Zone 1. The downstream toe rested on alluvium. A stockpile of riprap existed downstream of the toe along the right abutment at the time of failure. Thus there was considerable pervious material in contact with the right-abutment rock wall into which moderate quantities of clear or muddy water could have escaped for a limited time without detection.

The geometry of the mass of rock bounded by the downstream face of the key trench and the face of the right abutment immediately downstream of the key trench deserves attention. The outer part of the mass was removed by the floodwaters. It was transected by nearly vertical joints, of which the lower portions still remain (Figs. 3-9 and 3-11), which constituted a short path from the key trench to the portion of Zone 2 resting on the right-abutment talus at about El. 5200.

UNTENABLE FAILURE HYPOTHESES

Seismic Activity.

As indicated in Chapter 6, there is no evidence of seismic activity at the time of failure. The only earth motions recorded were those clearly caused by the escape of the turbulent floodwaters from the reservoir. Therefore seismic activity was not a cause of failure.

Settlement.

Settlement of the dam and its surroundings, possibly associated with compression of the Miocene lake and stream sediments, has been suggested as a cause of cracking leading to penetration of water and erosion.

Settlement observations were made on surface reference points on the embankment (Table 11-1). Because the reference points were established when the dam was almost completed, the records do not include the immediate settlements due to the weight of the embankment. They do include, however, post-construction settlements due to this weight plus the settlements associated with filling the reservoir and any distortions due to the subsequent failure. The maximum observed movements prior to the failure were on the order of 2 in., a value by no means unusual during first filling of reservoirs. The settlements of a large number of successful earth dams have been many times this value. Indeed, one of the advantages of earth dams is their ability to accommodate large settlements of the foundations and of the fill itself, as well as large differential settlements among various parts of the dam and its appurtenant works. Furthermore, had appreciable abutment settlements actually occurred, distress would have been visible in the concrete lining of the auxiliary outlet tunnel beneath the spillway structure. The tunnel, when inspected after the failure, was in excellent condition and displayed no cracks that could be attributed to causes other than normal shrinkage.

A resurvey of fourteen principal control points used in construction of the dam was made after the failure (Table 5-5). One control point was located as far as 6500 ft from the axis of the dam, beyond the influence of the weight of the dam; two were located close to the ends of the dam; others were at intermediate distances. Indicated differences in elevation of reference points before and after failure of the dam ranged from a settlement of 1.5 in. to a rise of 0.5 in. These differences might be considered to be within the tolerances for construction surveys run across a canyon, and not to be significant. It would be reasonable to conclude that a movement of as much as 0.5 in. might have occurred as a consequence of removing the weight of the failed portion of the dam. On the

assumption that the thickness of the Miocene sediments is 100 ft, the constrained modulus of elasticity of the sediments, to account for a rise of 0.5 in. on removal of 100 ft of embankment, is about 500,000 psi. This value corresponds to the in-situ modulus of many basalts and other flow rocks on first loading. Hence, settlement due to compressibility of the Miocene sediments should have differed little from that which would have occurred if they had been replaced by rhyolite. It should be noted that since the Miocene sediments are thicker than 100 ft, their calculated modulus is even greater.

On the basis of the preceding discussion the Panel concludes that settlement of the dam or its foundation did not contribute to the failure.

Reservoir Leakage.

High water losses in various exploratory drillholes near the dam and along the reservoir rim have been cited as a factor indicative of potential failure. Large losses of water beneath or around earth dams or through the reservoir rim may have an influence on the economics of a project but they have no relation per se to the safety of the dam, provided that the foundation and abutment treatment, to be discussed, is designed and executed in accordance with good practice. Therefore, the Panel does not consider the question of reservoir losses to be pertinent to the cause of failure of the dam.

Seepage Around End of Grout Curtain.

As concluded in Chapter 5, seepage around the end of the grout curtain was not a cause of failure.

MOST PROBABLE STEPS IN DEVELOPMENT OF FAILURE

Appearance of Springs.

As the reservoir level rose, more and more water gained access to the joints in the rhyolite, joints that increased in width in a general way with increasing elevation, especially above about 5200 ft. As a result of flow beneath and around the ends of the grout curtain, as well as through the "windows" existing in it, a general rise of groundwater levels occurred downstream of the dam. This rise led to the appearance of clear-water springs 1300 to 1500 feet downstream of the toe a few days before the failure. These springs, although predating the failure only slightly, were not in themselves indicators of developing defects, but were normal accompaniments of reservoir filling.

Development of Erosion Tunnel.

Between about Stas. 13+40 and 15+00, particularly unfavorable conditions existed, as described previously: (1) a geometry of the key trench especially favorable to arching, to poor compaction, and to cracking of the Zone 1 material; (2) significant water passages through the rock just beneath the grout cap and possibly through the grout curtain at greater depth; (3) a concentration of throughgoing joints beneath and alongside the key trench; and (4) an erodible fill within the key trench and in contact with the jointed rock downstream from the key trench. As a result of these conditions one or more erosion tunnels formed across the bottom of the key trench permitting water to flow readily from the open joints upstream to those downstream of the key trench and grout curtain. The manner of formation of the initial tunnel deserves detailed discussion and is treated in the next section.

As erosion enlarged the tunnel or tunnels, the discharge of water increased. The discharge, being of increasing amount and containing eroded silty soils, could escape only through passages of appreciable size. Some of the outflow undoubtedly entered the generally interconnected joint system downstream of the cutoff and spread through the rock mass, but a large part passed nearly horizontally, near El. 5220, through or around the narrow block of rock between the downstream

face of the key trench and the right abutment wall. Flow through this rock mass was facilitated by the concentration of joints intersecting the downstream wall of the key trench between Stas. 13+40 and 14+00. Part of the flow emerged from the rock against the Zone 1 fill on the right abutment, turned downstream, and flowed along the interface. Since the silty fill beneath minor overhangs and along near-horizontal joints was sheltered from overburden pressure, it too was vulnerable to erosion. The water and suspended silt continued along the interface until it reached Zone 2 or the pervious surficial soils and talus left beneath Zone 2. Another part of the flow remained in the rock near the abutment, where weathering and relaxation left more open joints than at greater depth, and then emerged into Zone 2 or the talus beneath it. Once the pervious zones were reached and as long as the outflow did not exceed their capacity, water flowed through the pervious materials near the groin of the right abutment and through the riprap stockpile at the toe.

Development of Erosion Gully.

During the night of June 4, however, the leakage began to exceed the capacity of the pervious materials, whereupon it emerged at El. 5200 and flowed briefly down the surface. Dampness and slight erosion were noted along the groin the next morning. Early in the morning, as flow continued to increase, muddy springs appeared at both El. 5045 and El. 5200. Soon the spring at El. 5200 was seen to be the mouth of an erosion tunnel extending along the rock at the base of the earthfill close to the groin. Progressive erosion led to continued increase in size of the tunnel until finally at about 10:30 a.m. the water pressure was great enough to break suddenly and violently through the Zone 2 fill and erupt on the face of the dam. Thereafter the erosion tunnel became an erosion gully, working headward first up the groin and then along the initial passage through the key trench. The gully extended upstream by successive collapses of the roof of the tunnel, including the sinkhole that appeared briefly at El. 5315, toward the vortex over the upstream end, culminating in collapse of the roadway at the crest of the dam.

INITIAL BREACHING OF THE KEY-TRENCH FILL

Conditions Favoring Erosion and Piping.

It was recognized by the USBR early in design, and confirmed by the Panel's investigation, that the Zone 1 material placed in the key trench and against the abutments was highly erodible. Wherever this material would be subjected to the action of flowing water, it would be attacked and washed away rapidly. Seepage through the material could also produce backward erosion due to grain-by-grain removal at points of emergence of flow lines where such points consist of voids unprotected by filters. The latter process develops slowly. Hence, it is unlikely to have played a significant role in the failure of Teton Dam, because the failure developed with remarkable rapidity.

Therefore the initial breaching of the key-trench fill can be attributed to erosion by direct contact with flowing water. This contact could have occurred under two conditions: Where the fill was in contact with open joints through which water was flowing, and through cracks in the fill itself. The physical conditions in the vicinity of Sta. 14+00 were conducive to both possibilities, and it is possible that both existed simultaneously.

The erodibility of the fill material itself is, moreover, dependent on its density and state of stress. Where loose, as in local zones where compaction is difficult or impracticable, the erodibility is substantially greater than where the fill is dense. Where the intergranular pressures are low, erosion can take place more readily than where they are high. Furthermore, if the water pressure exceeds the intergranular pressure, tension develops in the soil skeleton, and if the tension exceeds the tensile strength of the soil, the soil may crack by the process known as hydraulic fracturing. If the total

stress in the soil at a soil-rock interface is less than the water pressure in a joint at the interface, the soil may separate from the adjacent rock as a consequence of hydrostatic pressure, and the separation tends to propagate and allow greater access of the water to the soil.

Some or all of these conditions occurred near the base of the key trench near Sta. 14+00 and, separately or in combination, were responsible for the original breach of the key-trench fill. The evidence is reviewed, for clarity of presentation, under the following two headings, although the two topics cannot, in fact, be completely separated.

Evidence of Attack on Fill by Flow in Rock Joints Along the Contact.

The mechanism of erosion under these conditions is illustrated by Fig. 12-1, in which is depicted an idealized joint in the bottom of the key trench. The joint is not sealed by dental concrete or slush grout; consequently, horizontally flowing water under pressure would attack the base of the fill and begin to form a pipe. If the joint occurred at a step in the rock surface, Fig. 12-2, the erosion would occur even more readily because of the reduction of stresses in the reentrant corner due to arching, as will be discussed in greater detail in the next section, and because of the likelihood of poor compaction of the fill in the corner. Furthermore, under high water pressure, the pipe is likely to enlarge by separation of the fill from the rock surface, as illustrated in Figs. 12-2 and 12-3. Conditions corresponding to Figs. 12-1 and 12-2 have been observed in the key trench near Sta. 14 + 00 as documented in Chapter 3; indeed, several instances of overhanging as well as near-vertical steps were noted.

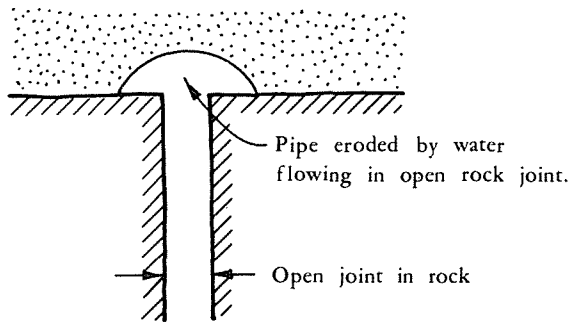
In reality, the key trench at the failure section contained a grout cap overlying a single-line grout curtain flanked by two other lines of grout holes intended to contain the flow of grout from the curtain grouting. The investigations carried out at the request of the Panel, described in Chapter 3, demonstrate clearly that openings or windows existed in the grout curtain near the failure section, particularly at shallow depth beneath the grout cap. These conditions are illustrated diagrammatically in Figs. 12-4 to 12-6. The diagrams show how the initial formation of pipes along transverse joints, as illustrated in Figs. 12-1 to 12-3, associated with even modest seepage beneath the grout cap, can develop (Fig. 12-5) into larger cavities upstream and downstream of the grout cap, and how the cavities may unite under the high hydraulic gradient between them to form a single erosion tunnel. After this occurs, enlargement of the tunnel is restricted only by the capacity of the adjacent joints to deliver and carry away the through-flowing water.

The Panel's investigations leave no doubt that all the conditions for creation of the initial breach by this mechanism existed between about Stas. 13+40 and 15+00. The results of ponding tests on throughgoing joints demonstrated the existence of zones of ready leakage beneath the grout cap at the places shown in Fig. 3-17 and the existence of deeper windows in the grout curtain is indicated in Table 3-2.

Furthermore, the topography of the bottom of the key trench in the vicinity of the failure showed a concentration of steps and overhangs conducive to arching and poor compaction.

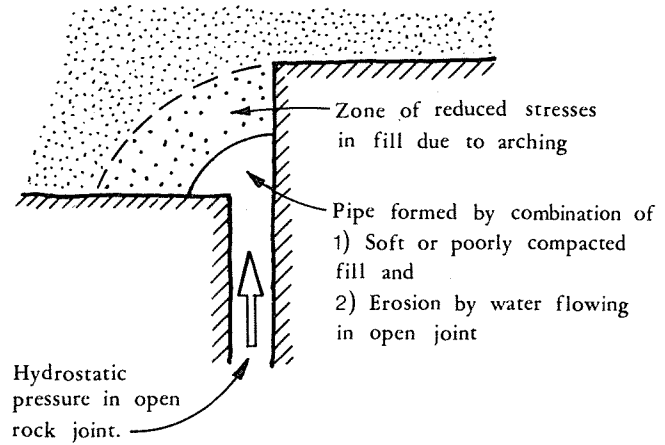
Evidence of Attack on Fill Through Cracks in Fill.

Cracking of cohesive soils in the impervious sections of earth dams is a well known phenomenon associated with tensile strains due to differential settlements among portions of the dam or between the embankment and its foundation. A variety of defenses is available to the designer to reduce the potential for cracking and to render harmless those that occur. The mere presence of cracks, therefore, is not an indication of unsatisfactory design or performance. At Teton Dam, however, it is apparent that cracks through the key trench would inevitably lead to rapid erosion and would thus constitute remarkably efficient avenues for breaching the seepage barrier and initiating failure.



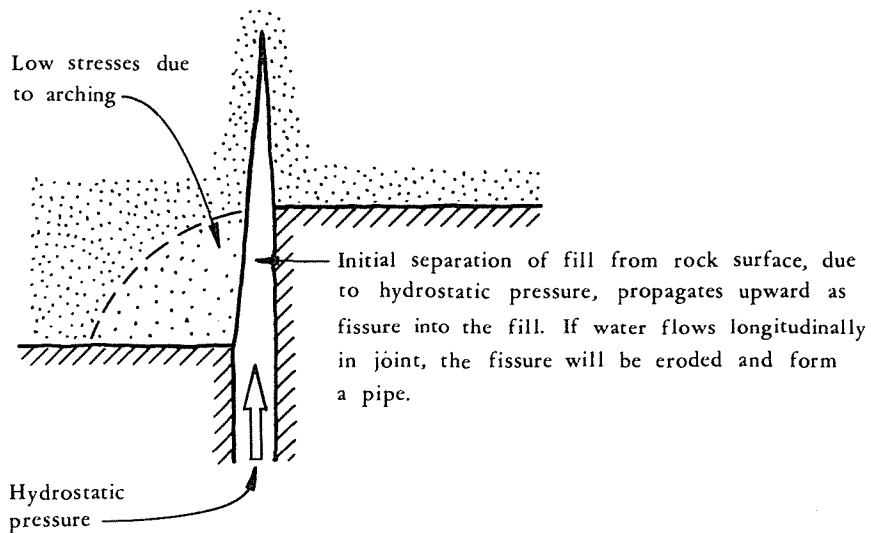
Pipe eroded by water flowing along open joint

Fig. 12-1



Pipe eroded by water flowing along re-entrant step in rock

Fig. 12-2



Fissure in fill produced by hydrostatic pressure in rock joint along re-entrant step

Fig. 12-3

EROSION DIAGRAMS

FIGS. 12-1. THROUGH 12-3.

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INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

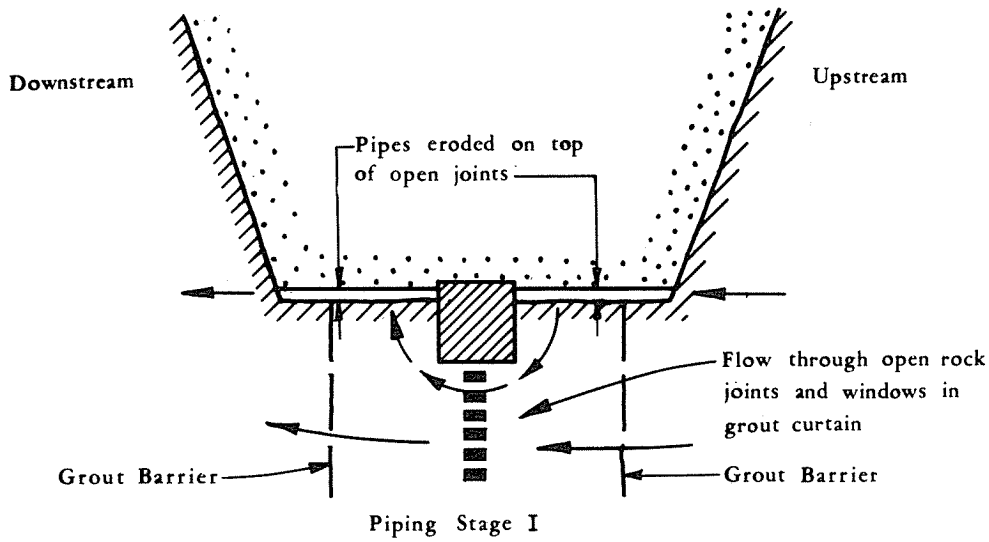


Fig. 12-4

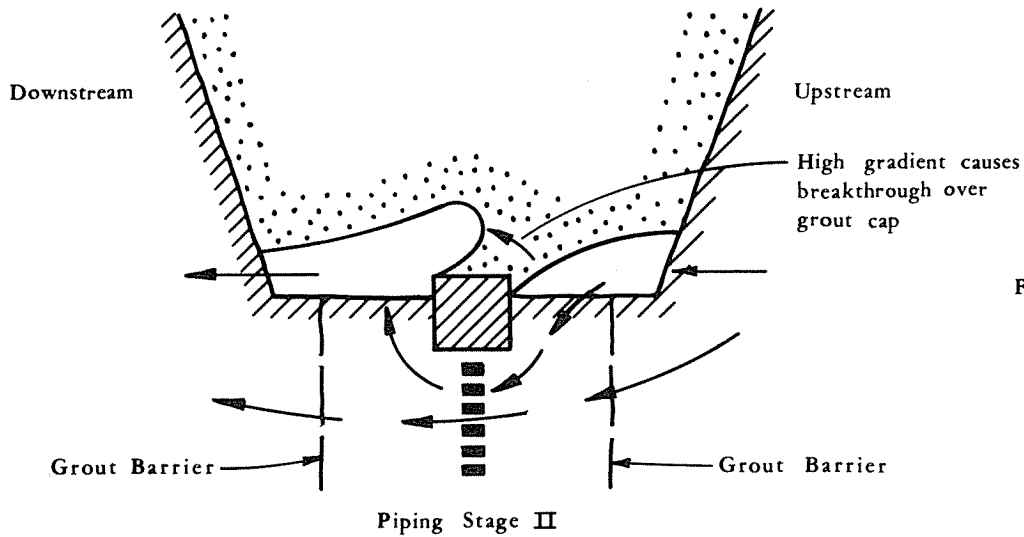


Fig. 12-5

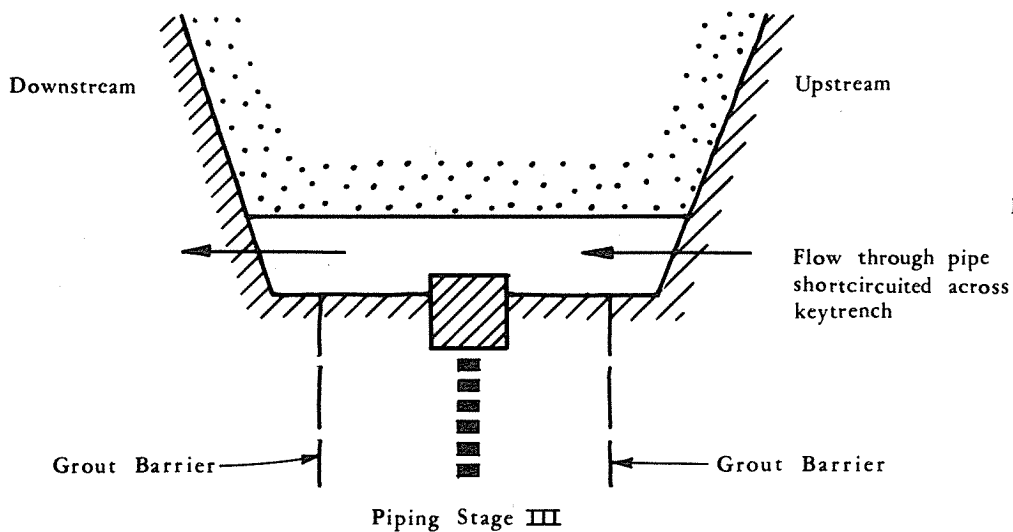


Fig. 12-6

EROSION DIAGRAMS

FIGS. 12-4. THROUGH 12-6.

U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Cracking is associated with zones of low compressive stresses in the fill. Such zones are related to differences in compressibility of materials in adjacent portions of the dam, or between the embankment and its foundation or abutments. The weight of the embankment overlying a compressible material is partly transferred to adjacent less compressible materials as a result of shearing stresses developed in the embankment. This process of stress transfer is commonly referred to as arching.

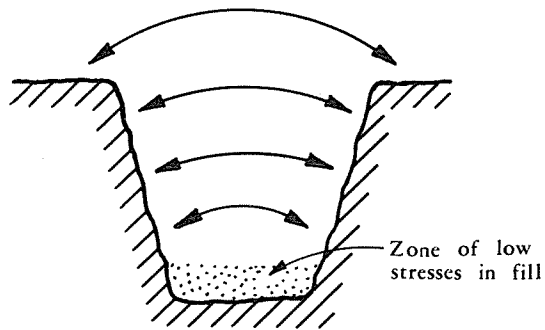
In a qualitative sense, arching is illustrated in Fig. 12-7 which depicts an idealized cross section through the key trench of Teton Dam. The rigidity of the rock walls, combined with their steep slopes, causes a high degree of arching or stress transfer to the abutments, and a corresponding zone of low stresses in the fill at the bottom of the trench. Because of its importance, this arching transverse to the key trench is designated as first-order arching. In a longitudinal direction, however, the profile of the bottom of the key trench contains major steps or irregularities; in the depressions the compressible materials cause load to be transferred to the adjacent rock surfaces. Arching of this type, Fig. 12-8, is designated as second-order. Finally, local smaller steps and overhangs occur that lead to a third order of arching (Figs. 12-2 and 12-3) that may not only be significant in itself but is especially likely to be associated with poor compaction. All three orders of arching may occur simultaneously and the reductions of stress are additive. That the conditions in the key trench near Sta. 14+00 were conducive to all three orders is evident from a study of the cross sections, the longitudinal sections, and numerous photographs. The occurrence of a high degree of arching near Sta. 14+00 combined with the presence in this area of joints capable of delivering and carrying away large quantities of water make cracking a highly probable potential cause of breaching of the key-trench fill.

The qualitative discussion of arching represented by Figs. 12-7, 12-8, and 12-3 can be supplemented by quantitative studies carried out by the finite-element method. Although this method of analysis has been applied to the calculation of stresses in earth dams for some years, it is still under development. At the present time, the techniques are practicable for two-dimensional analyses. The most refined techniques require the determination of nine different soil parameters, determined from triaxial tests. The parameters do not take into account the influence of time, although allowances can be made for the effect of yielding, and the computational procedures permit allowing for the progressive construction of the embankment. Thus, the numerical results of stress and displacement calculations, although themselves quantitative, are best used in a qualitative or semi-quantitative manner.

Soil parameters for analyses at critical locations were selected by members of the Panel on the basis of five triaxial tests, some performed by the USBR during design of the dam and some performed by the USBR and Northern Testing Laboratories according to procedures specified by the Panel, and on the basis of experience in testing other materials. A range of values was chosen for the principal parameters to take some account of variations in properties from those represented by the samples tested. Details of the materials tested and of the tests are described in Chapter 3.

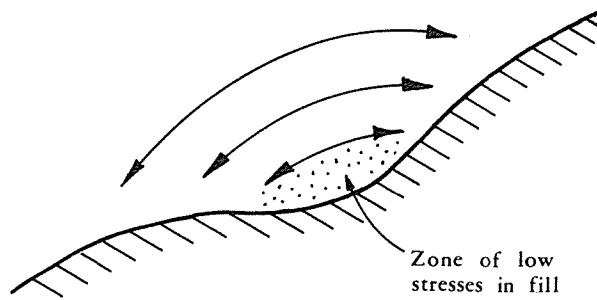
Two-dimensional analyses were performed for cross sections at Stas. 12+70, 13+70, and 15+00 on the right abutment and at Sta. 27+00 on the left abutment. The sections on the right abutment were chosen by the Panel as being representative of the portion of the key trench where the seepage barrier was initially breached. Similar sections at Stas. 26+00 and 27+00 were analyzed to permit comparing certain predictions based on the finite-element analysis with field observations carried out in the portion of the embankment remaining after the failure, as discussed in the next subsection.

The values of vertical normal stress throughout the embankment and key-trench fill are shown as fractions of overburden pressure in Fig. 12-9 for Sta. 15+00. For comparison, the figure also shows



First order arching in fill over and between walls of key trench

Fig. 12-7



Second order arching produced by longitudinal topography of rock surface in key trench

Fig. 12-8

EROSION DIAGRAMS

FIGS. 12-7. THROUGH 12-8.

U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

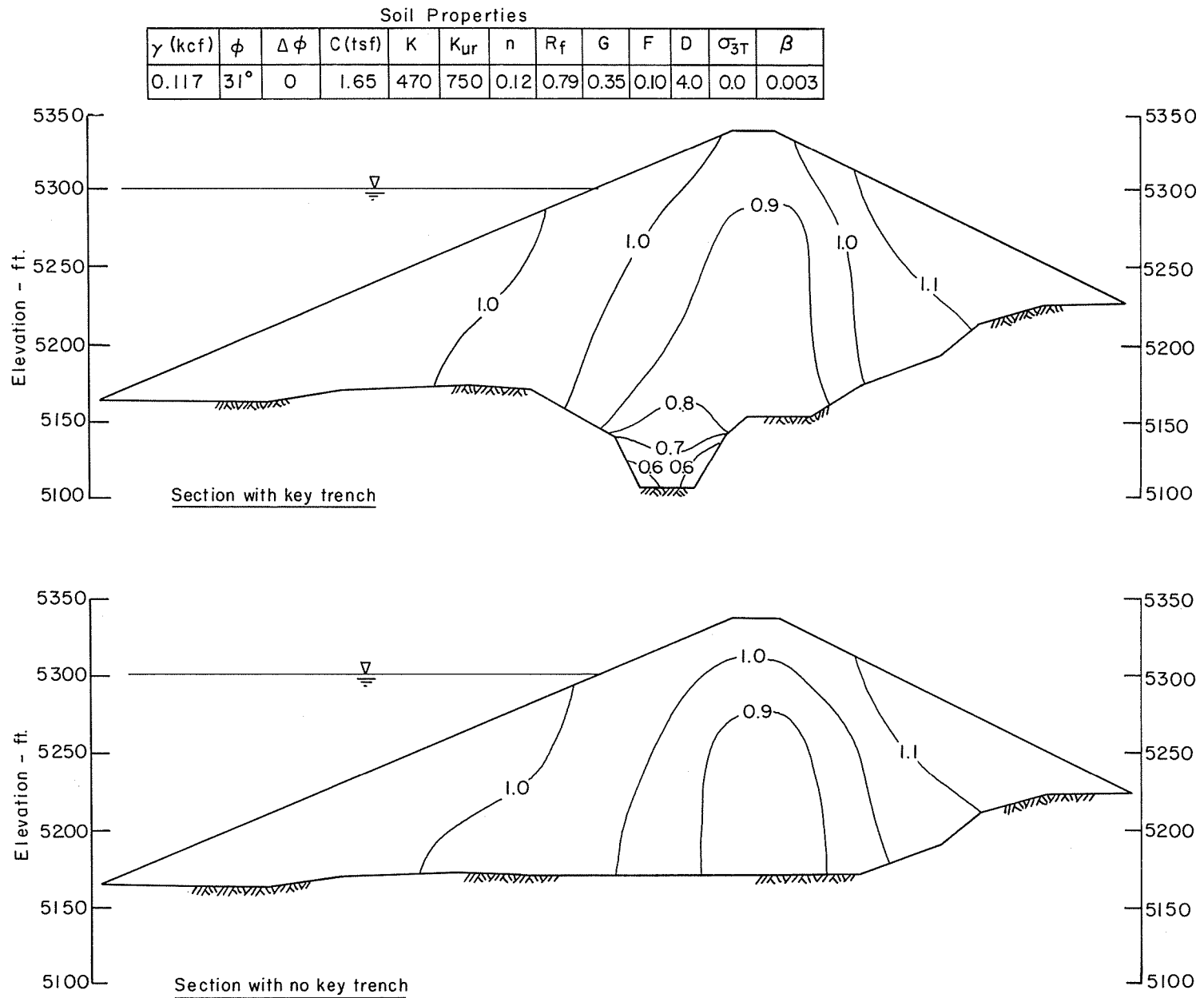


FIG. 12-9

CONTOURS SHOWING RATIO OF VERTICAL STRESS IN EMBANKMENT TO OVERBURDEN PRESSURE FOR CONDITIONS WITH AND WITHOUT KEY TRENCH AT STA. 15+00 - BEFORE WETTING OF SOIL IN KEY TRENCH

the stresses that would exist if there were no key trench; that is, if there were no redistribution of stresses due to arching. The results demonstrate the marked reduction of stress due to first-order arching as a result of the presence of the key trench. Because the results are two-dimensional, they give no information about the second-order arching associated with the longitudinal configuration of the key trench (Fig. 12-8), or about the third-order arching illustrated in Figs. 12-2 and 12-3.

Hence, the finite-element studies confirm and to a degree quantify the importance of the arching associated with the key trenches adopted for the abutments of Teton Dam.

The analyses were carried out under the direction of members of the Panel. The details of the analyses and an assessment of their pertinence are contained in Appendix D.

Considerations of Hydraulic Fracturing.

Within the limitations of the validity of the assumptions, the finite-element method of analysis also permits calculation of whatever other components of stress are considered to be of interest. It is thus possible, in principle, to compute the minimum total compressive stress at any point within the dam and to compare the sum of this stress and the tensile strength of the fill material with the porewater pressure. If the sum of the normal stress and tensile strength is less than the porewater pressure, the possibility exists that cracks will develop by a mechanism known as hydraulic fracturing.

Hydraulic fracturing has often been induced in the impervious zones of earth dams by creating sufficient head in water-filled drill holes in the dams. Measured values of water pressure to cause fracturing, indicated by sudden loss of water from the drill hole, have compared favorably with the sum of reasonable values of tensile strength of the material and the total minor principal stress calculated by the finite-element procedure.

Because of the agreement between calculated and observed values of water pressure to cause hydraulic fracturing around drill holes, field tests in drill holes have been used to check the reasonableness of soil parameters determined by laboratory tests for finite-element analyses, or to determine the most appropriate value of one of the more significant parameters. Such tests were carried out at the request of the Panel near the left abutment at Stas. 26+00 and 27+00 where the key trench sections closely resemble those at Stas. 15+00 and 13+70 in the right abutment, in order to verify the general applicability of the parameters selected from the soil tests and to aid in selecting the applicable value of Poisson's ratio. The details of the procedure and of the interpretation are given in Appendix D.

The possibility of hydraulic fracturing in dams as a result of water pressure from a reservoir applied against an impervious zone has also been investigated by the finite-element procedure. This is accomplished by comparing the sum of the minimum total compressive stress and the tensile strength at a point in the zone with the water pressure at that point. In computing the stresses in the soil, the effects of consolidation, swelling and creep are not fully considered. However, this limitation can be overcome to some extent by considering a range of soil parameters and by utilizing observed field performance as an additional guide in selecting the most suitable values.

In determining the water pressures, consideration should be given to the head losses associated with whatever seepage may occur. If a completely impervious grout curtain should be achieved, for example, full hydrostatic pressures corresponding to the reservoir level would be exerted against the cutoff and the possibility of hydraulic fracturing would be maximized. At the other extreme, if the efficiency of the grout curtain is very low, the water pressures exerted against the cutoff would be greatly reduced and, for a reasonably symmetrical section, would approach half the values

corresponding to the reservoir level. The potential for hydraulic fracturing would be correspondingly reduced.

Thus, depending on personal evaluations of the efficacy of a grout curtain and of the applicability of the stress analyses, there are likely to be differences of opinion concerning the possibility of hydraulic fracturing in any given case. Nevertheless, the results of such calculations when applied to the right abutment of Teton Dam between Stas. 12+70 and 15+00 lead to interesting conclusions. The abbreviated discussion in this section is treated more fully in Appendix D.

The results for Sta. 13+70 illustrate the findings. They show that the minor principal stress lies in the plane of the cross section and, at the upstream face of the key trench, is inclined downward at about 30° in the downstream direction, as illustrated in Fig. 12-10. Thus, the first cracks would form in a direction parallel to the axis of the key trench rather than across it. Computed values of the minor principal stress in the plane of the section at Sta. 13+70 are compared with the full hydrostatic pressures for reservoir level at 5300 ft in Fig. 12-11. The comparison shows that hydraulic fracturing would occur in a zone along the upstream face and the lower portion of the key-trench fill. Thus, water would be distributed longitudinally to any nearby sections where transverse cracking might occur.

The possibility of transverse cracking is shown in Fig. 12-12, in which the computed values of normal stress on the transverse section at Sta. 13+70 are compared with hydrostatic pressures for reservoir level at 5300 ft. Hydraulic fracturing is indicated across the entire bottom of the key trench up to a height of approximately 15 ft. Thus, the analysis indicates that cracking due to hydraulic fracturing could have been responsible for initial breaching of the seepage barrier, if full hydrostatic pressures developed on the upstream face of the key trench. Similar results, with more extensive zones of fracturing, are found for the sections at Sta. 15+00. On the other hand, no fracturing is indicated for the section at Sta. 12+70.

With the reservoir level at El. 5255, as it was on May 20, 1976, the calculations indicate that the hydrostatic pressures in the upstream jointed rock would have been sufficient to cause hydraulic fracturing only in the bottom 10 ft of the key trench at Sta. 15+00, but not at Stas. 13+70 and 12+70. This condition is shown by the longitudinal section, Fig. 12-13, drawn through the centerline of the key trench. The shaded area indicates a very small zone near Sta. 15+00 where on May 20 the water could move through hydraulically induced fractures. The extent of the zone on May 25 with reservoir elevation at 5275, and on June 5 with reservoir elevation at 5300, are also shown. Downslope of about Sta. 16+00, hydraulic fracturing would not have occurred because the key trench became either very shallow or nonexistent.

Fig. 12-13 summarizes the extent of the zone of hydraulic fracturing as estimated from the results of the analytical studies. It indicates that a substantial zone of vulnerability could have developed no earlier than two weeks before failure actually occurred, and that the location of the zone coincides closely with the zone in which piping finally developed. These coincidences lend support to the hypothesis that hydraulic fracturing of the soil in the key trench is a highly probable mechanism for the initial breaching of the seepage barrier.

On the other hand, in the calculations made to determine the zones susceptible to hydraulic fracturing, it has been assumed that full hydrostatic pressure acts on the upstream face of the impervious fill. In reality, because the grout curtain had windows and flow occurred through it, the actual pressures against the upstream face and beneath the key trench upstream of the grout curtain would be less than full reservoir pressure. Hence such comparisons indicate greater potential for

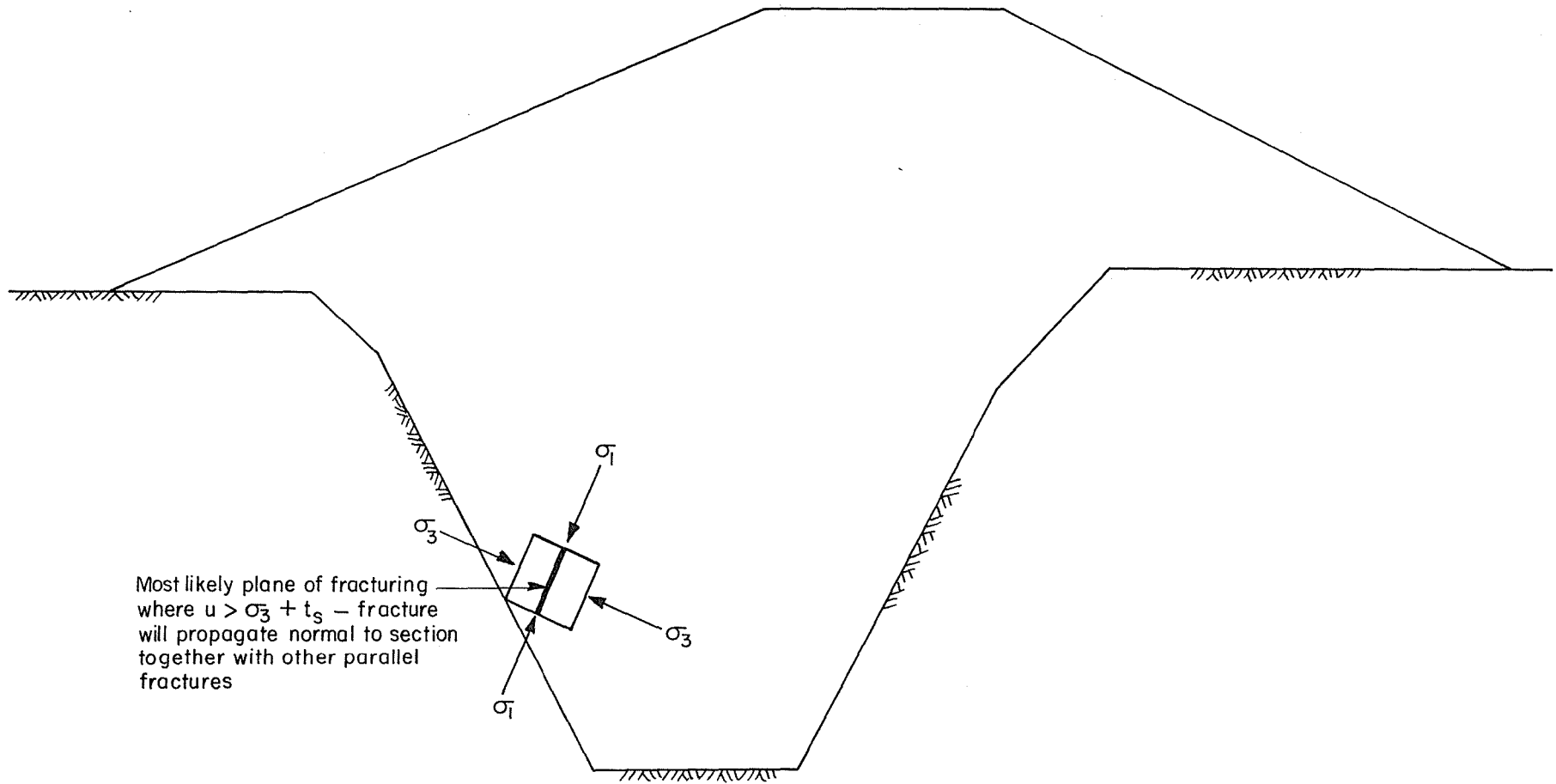


FIG. 12-10

DEVELOPMENT OF LONGITUDINAL FRACTURE PATTERN

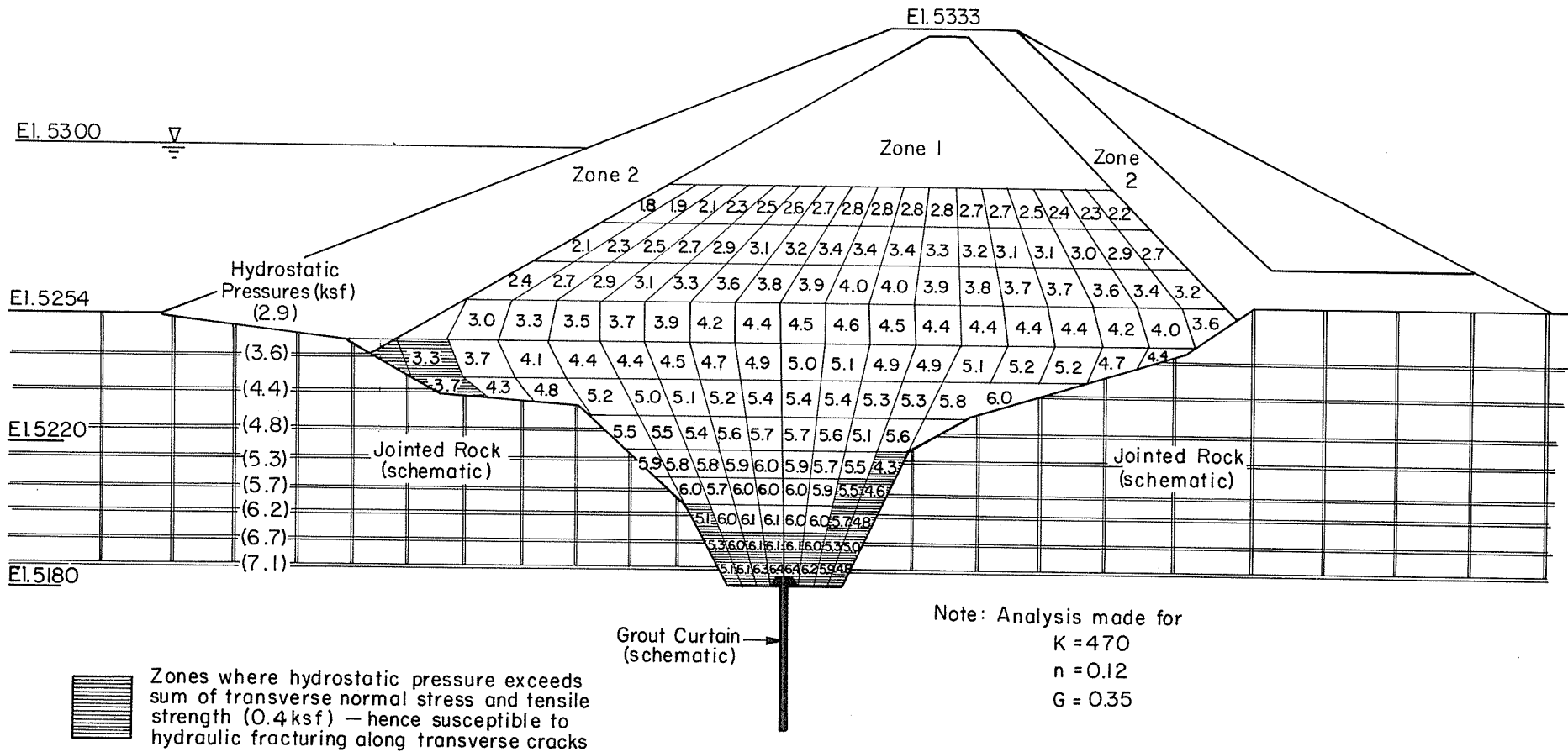


FIG.12-II COMPUTED VALUES OF MINOR PRINCIPAL STRESS IN PLANE OF SECTION STA. 13+70

12-16

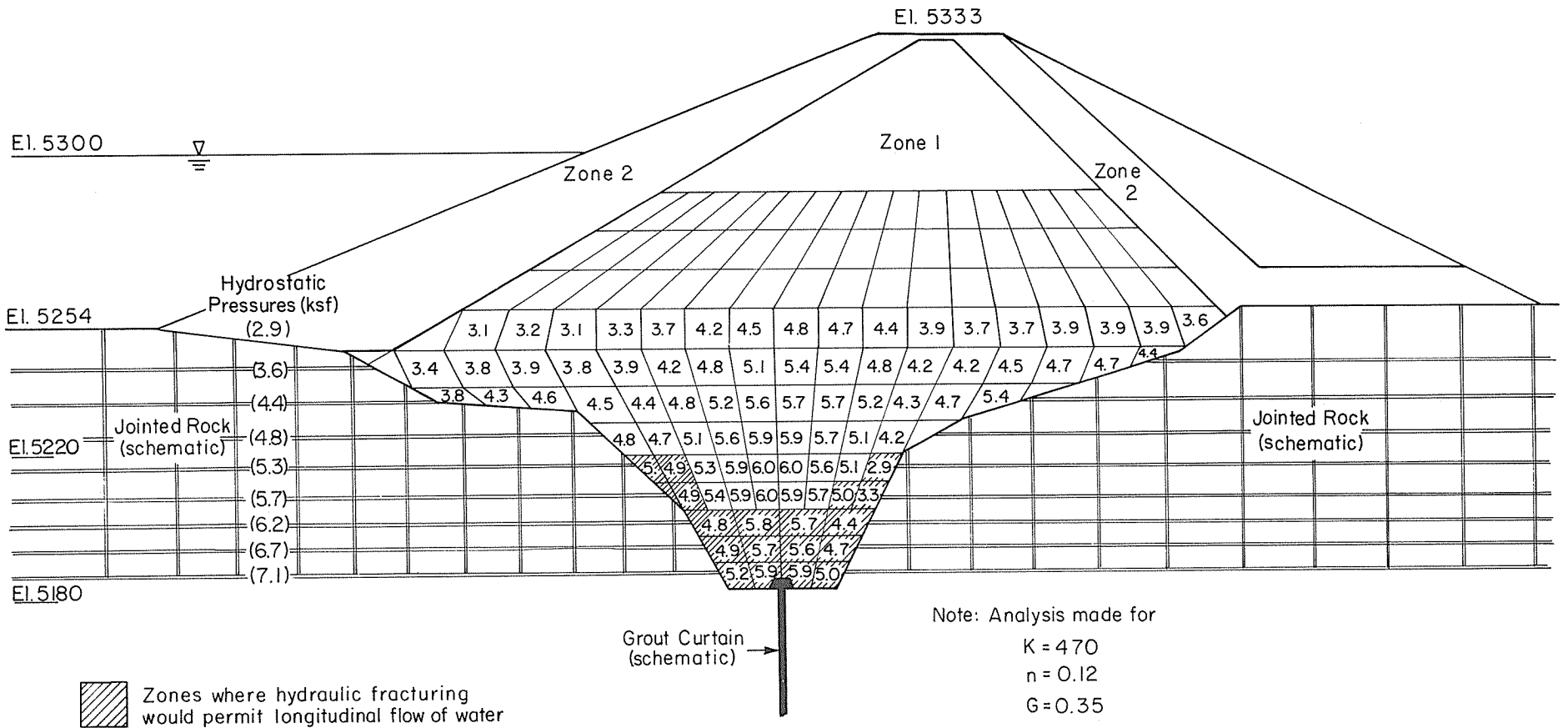


FIG.12-12 COMPUTED VALUES OF NORMAL STRESS ON TRANSVERSE SECTION IN ksf STA.13+70

12-17

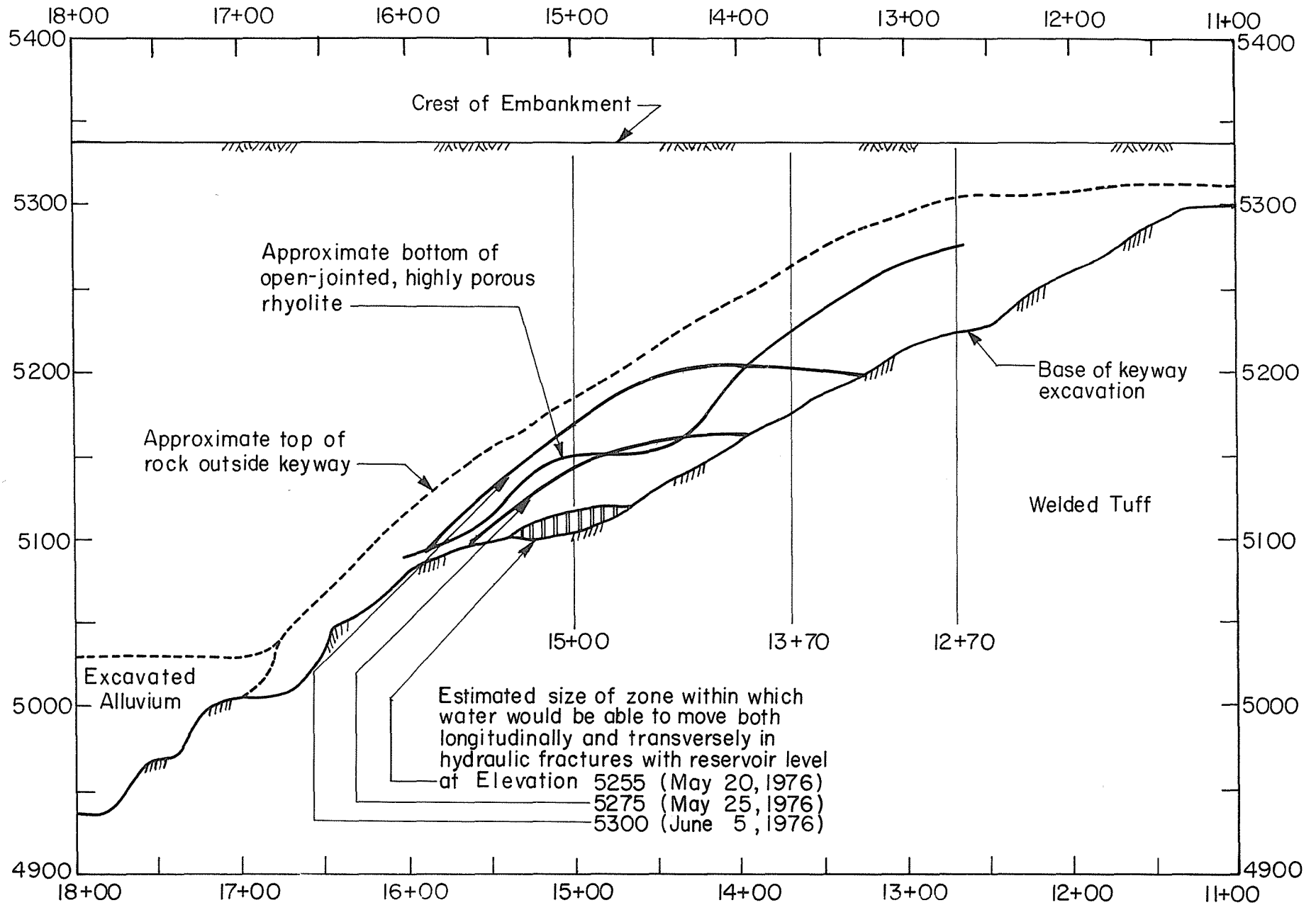


FIG. 12-13 LONGITUDINAL SECTION THROUGH CENTER LINE CREST AND GROUT CAP

hydraulic fracturing than actually existed. Even so, allowing for some reduction in water pressure due to seepage, sufficient disparity exists between water pressures and lateral soil stresses at sections such as Stas. 13+70 and 15+00 to enable fracturing to occur. This would be accentuated as the key-trench fill became saturated and arching effects became more pronounced.

It is perhaps paradoxical that if, on the one hand, the grout curtain were not effective, failure would result directly from the underseepage whereas, on the other hand, if the grout curtain were fully effective, failure would tend to develop as a result of hydraulic fracturing.

SUMMARY

Upstream of the seepage barrier there was ample opportunity for reservoir water to reach the barrier in quantity through the joint system in the rock. The physical conditions were fully satisfied for water flowing under high pressure to attack the lower part of the key-trench fill along open joints, some of which were found to transmit water freely through the grout curtain, particularly through the upper part near the grout cap. The attack was fully capable of quickly developing an erosion tunnel breaching the key trench. Arching at local irregularities, loose zones of fill at reentrants, and local cracking may have contributed to the success of the attack and determined the precise location. Hydraulic fracturing, according to analytical studies, may also have been responsible for the initial breaching of the key-trench fill. Conditions were favorable for escape of the water and eroded solids into the joints of the rock downstream, for discharging the water against and along the interface of the right abutment of the dam and the embankment, and for development of the erosion features that ultimately breached the entire dam.

The precise combination of geologic details, geometry of key trench, variation in compaction, or stress conditions in fill and porewater that caused the first breach of the key-trench fill is of course unknown and, moreover, is not relevant. The failure was caused not because some unforeseeable fatal combination existed, but because (1) the many combinations of unfavorable circumstances inherent in the situation were not visualized, and because (2) adequate defenses against these circumstances were not included in the design.

APPENDIX A

USBR LIST OF TETON DAM FAILURE EXHIBITS FURNISHED TO INDEPENDENT PANEL

TETON DAM FAILURE EXHIBITS

No.

1. Panel Information Packet

- 1.1 Brochure on Lower Teton Division, Idaho, dated 1974
- 1.2 Comparison - Specifications vs. Final Quantities (DC-6766)
Teton Dam, Pilot Grouting (Table)
- 1.3 Construction Materials Test Data
- 1.4 Design Considerations
- 1.5 Drawing of Teton Dam Left Abutment Cut-off Trench, Station $\pm 33+20$ to
Station 34+00, dated 8/17/72
- 1.6 Drawing - Teton Dam - Location of Explorations for Borrow Areas "A,"
"B," and "C"
- 1.7 Earthwork Construction Data, dated June 1975
- 1.8 Earthwork Control Analysis (2 printouts - Zone Number Dam 1, Run No. 14,
and Zone Number Dam 3, Run No. 7)
- 1.9 Final Environmental Statement (including pertinent letters)
- 1.10 Geological Survey letter regarding Teton Dam (Memo from Commissioner Stamm
transmitting letter dated June 11, 1976 from V. E. McKelvey of
the Survey to Senator Henry M. Jackson)
- 1.11 Geological Survey Questions and Answers Regarding Teton Dam (Wire
message dated June 18, 1976 - Questions and Answers dated June 15,
1976)
- 1.12 Key Events and Key Personnel - Teton Dam, Design and Construction,
Denver Office, from 1/1/69 to present (June 28, 1976)
- 1.13 L-10 - Final Report on Foundation Pilot Grouting
- 1.14 List of Teton Dam Material in Central Files, Library, etc., at
E&R Center
- 1.15 List of Teton Original Drawings on file in PN 700 as of 6/15/76
- 1.16 Listing of correspondence and reports on Teton Dam Project on file in the
Regional Geology Office
- 1.17 Listing of Key Personnel - Teton Project Office - 1967 to present
- 1.18 Morrison-Knudsen wire message dated June 11, 1976, regarding their part
in building Teton Dam
- 1.19 News Release - Teton Dam Failure - Department of the Interior, dated
6-9-76
- 1.20 News Release - Panel Named to Review Cause of Teton Dam Failure -
Department of Interior, dated 6-10-76
- 1.21 Preliminary Geologic Map of the NW 1/4 Driggs 1° by 2° Quadrangle,
Southeastern Idaho (USGS)
- 1.22 Progress Chart - DC-6910 (showing maximum section, with dates of
construction)
- 1.23 Records available at Teton Project Office (Faxogram from Project
Construction Engineer, Newdale, Idaho to Regional Director, Boise)
- 1.24 Resume of Facts and Findings - Teton Dam, Idaho

TETON DAM FAILURE EXHIBITS - Continued

No.

- 1.25 Seismic monitoring program - Teton Dam and Reservoir (including memorandum from Acting Chief Geologist Robert C. Davis to D. J. Duck, dated July 20, 1973 and Preliminary Report on Geologic Investigations, Eastern Snake River Plain and Adjoining Mountains, a draft report by Steven S. Oriel, Harold J. Prostka, David Schleicher, and Robert J. Hackman, USGS, June 1973)
- 1.26 Testimony, Vol II, Vol V, Civil Case No. 1-71-88, Trout Unlimited et al vs. Rogers C.B. Morton et al
- 1.27 Water Surface Elevations - March, April, May, and June 1976
- 1.28 Wire Message from R. R. Robison to Commissioner of Reclamation, Director of Design and Construction, and Regional Director, Boise, subject, "Failure of Teton Dam, Teton Project, Idaho," dated June 6, 1976
2. Teton Dam Book
3. Plans and Specifications Packet
 - 3.1 Plans and Specifications - DC-6910 - with supplemental notices (Four volumes - 10 supplemental notices)
 - 3.2 Abstract of Bids
 - 3.3 Record of Subsurface Investigations
 - 3.4 Specifications No. DC-6766 - Teton Dam, Pilot Grouting - with one supplemental notice
4. General Plan Sketch
5. Maximum Section Sketch
6. Profile Sketch
7. Prints of slides - Location of Damsite - Construction through Failure
8. Photographs of Failure
9. 8mm Film of Dam Failure
10. 16mm Film of Dam Failure
11. Record of Filling of Teton Reservoir (2-page memorandum with the following)
 - 11.1 Memorandum of March 3, 1976 from Project Construction Engineer to Director of Design and Construction, subject, "Monitoring Ground Water Conditions - Teton Project, Idaho"
 - 11.2 Memorandum of March 23, 1976 from Director of Design and Construction to Project Construction Engineer, subject, "Reservoir Operating Criteria - Teton Dam - Teton Basin Project, Idaho,"
 - 11.3 Faxogram from Project Construction Engineer to Director of Design and Construction dated May 14, 1976, subject, "Status of Construction of Teton Dam and Filling of Reservoir - Teton Project, Idaho"

TETON DAM FAILURE EXHIBITS - Continued

No.

- 11.4 Daily Records of Reservoir Filling (Same as Exhibit 1.27)
- 11.5 Record of observation well from October 1975 to June 1976 (6 sheets)

12. Geology Handout

- 12.1 Introduction
- 12.2 Part I
- 12.3 Part II
- 12.4 Index
- 12.5 Letter of June 14, 1976 to Director of Design and Construction from Regional Director with attachments, as follows:
 - 12.5.1 Maps of the reservoir seepage loss study, including isopachs, water table contour for 2-2-76 and 6-1-76, and cross sections
 - 12.5.2 "500 series" geologic drill logs DH-501 through -507
 - 12.5.3 Water level data from observation wells
 - 12.5.4 Hydrographs of Teton Reservoir and observation wells
- 12.6 Seismicity reports including four sent to the Bureau by U.S. Geological Survey in letters dated:
 - 12.6.1 April 26, 1976
 - 12.6.2 February 19, 1976
 - 12.6.3 September 19, 1975
 - 12.6.4 September 4, 1975
 - 12.6.5 Memorandum on the geologic and seismic factors of Island Park and Jackson Lake Dams dated March 30, 1973
- 12.7 Two reports
 - 12.7.1 "Preliminary Report on Geologic Investigations, Eastern Snake River Plain and Adjoining Mountains" by the USGS, sent by cover letter dated July 20, 1973
 - 12.7.2 "Groundwater Investigations of the Rexburg Bench," by the Bureau of Reclamation, February 1972
- 12.8 Laboratory test data of foundation rock core specimens covered by memorandums dated:
 - 12.8.1 November 24, 1970
 - 12.8.2 December 1, 1970
 - 12.8.3 December 2, 1970
- 12.9 Final Construction Geology Report for the Spillway (Draft)
- 12.10 Drawings
 - 12.10.1 Teton Dam - Plan View of Fissures Exposed in Haul Road Cut - Drawing No. 549-100-176 - July 1976

TETON DAM FAILURE EXHIBITS - Continued

No.

- 12.10.2 Profiles of Right Abutment 200 and 250 Feet Upstream of Dam Axis, Un-numbered - post June 5, 1976
- 12.10.3 Construction Geology of Spillway - Drawings No. 549-100-124 to -132 - December 1975
- 12.10.4 Generalized Geologic Section A-A' Drawing No. 549-100-152 - March 1976
- 12.10.5 Geologic Map of Cutoff Trench, Stations 2+60 to 34+20 - Drawings No. 549-100-158 to -168 - June 1976
- 12.10.6 Geologic Section Along Upstream Grout Curtain, Stations -5+10 to 49+00 - Drawings No. 549-100-169 to -172 - June 1976
- 12.10.7 River Outlet Works Tunnel Geology, Stations 7+72.5 to 28+97.0 - Drawings No. 549-147-100 to -115 - April 1973
- 12.10.8 River Outlet Works Tunnel Gate Shaft Geology - Drawings No. 549-147-117 to -118 - April 1973
- 12.10.9 River Outlet Works Tunnel Gate Chamber Geology - Drawing No. 549-147-119 - April 1973
- 12.10.10 River Outlet Works Tunnel Intake Shaft Geology - Drawing No. 549-147-120 - April 1973
- 12.10.11 Geology and Explorations in Right Abutment Keyway Trench - Drawing No. 549-147-133 - April 1974
- 12.10.12 Geologic Sections Across Fissures in Right Abutment Keyway Trench - Drawing No. 549-147-134 - April 1974
- 12.10.13 Auxiliary Outlet Works Geology, Stations 6+63 to 34+11.33 - Drawings No. 549-147-400 to -419 - October 1974
- 12.10.14 Auxiliary Outlet Works Shaft and Adit Geology - Drawing No. 549-147-420 - October 1974
- 12.10.15 Auxiliary Outlet Works Access Shaft Geology, el 5080 to el 5290 - Drawing No. 549-147-121 - October 1974
- 12.10.16 Location of Exploration and Surface Geology - Drawings No. GEOL-76-020 and -021 - June 1976
- 12.10.17 Geologic Section Along Downstream Grout Curtain - Right Abutment Drawing No. GEOL-76-022 - June 1976

13. Prints of Slides (Geology)

14. Seismicity

- 14.1 Epicenters with Modified Mercalli Epicentral Intensity V or Greater through 1970
- 14.2 Maximum Epicentral Intensity (Modified Mercalli) per 10,000 sq. km. through 1970
- 14.3 Horizontal Acceleration in Rock with 10% Probability of Being Exceeded in 50 Years (2 sheets, one redrawn from the other)
- 14.4 Figure 2.--Location of seismic stations near Teton Dam
- 14.5 Figure 6.--Portion of seismogram showing ground motion induced by flooding waters

TETON DAM FAILURE EXHIBITS - Continued

- No.
15. Regional Environmental Geology of Southeastern Idaho, by Steven S. Oriel
(Unedited remarks prepared for presentation to Review Group June 15, 1976)
 16. Composite Drawing of Grouting Profile (Same as 12.10.6)
 17. Photographs of Key Trenches (Grouting)
 18. Grout Profile of Right Abutment (This is included in Exhibit 32.)
 19. Handouts on Embankment
 - 19.1 Teton Dam Earthwork Control Data - "Part C - Earthwork Construction Data" from L29 Reports - May 1972 to November 1975
 - 19.2 Teton Dam - Earthwork Information from Weekly Progress Reports - June 1973 to December 1975
 - 19.3 Sequence of Earthfill Placement from L29 Reports - June 1972 to October 1975
 - 19.4 Maximum Sections and Earthwork Control Statistics of Earth-fill Dams Built by the Bureau of Reclamation - June 1973
 - 19.5 Measurement Points (with observation dates) (seven sheets)
 - 19.6 Right Abutment Cross-Sections Before and After June 5, 1976 - Stations 100 through 400 Upstream of the Dam Axis (seven sheets)
 - 19.7 Memorandum dated June 4, 1976 from Project Construction Engineer to Director of Design and Construction, subject, "Filling of Teton Reservoir, Teton, Project, Idaho, with drawing showing location of springs.
 - 19.8 Teton Flood Data
 - 19.9 Memorandum dated June 4, 1976 to Project Construction Engineer from Director of Design and Construction, subject, "Status of Construction of Teton Dam and Filling of Reservoir - Specifications No. DC-6900 - Morrison-Knudsen-Kiewit, Contractor - Teton Dam, Power and Pumping Plant - Teton Basin Project, Idaho"
 20. Photographs of Key Trenches (Embankment)
 21. Letter from R. Keith Higginson, State of Idaho Department of Water Resources, to Wallace L. Chadwick, dated June 21, 1976, requesting additional information for the Panel
 - 21.1 Draft of Reply, dated 6/24/76
 - 21.2 Corrected Reply, dated 7/8/76
 22. Chart - Bureau of Reclamation Organizations at Engineering and Research Center - March 1976
 23. Eye Witness Accounts - Interrogatories by Division of Investigation Special Agents, Office of Audit and Investigation, Office of the Secretary, on Behalf of the Teton Dam Project Review Committee, dated June 25, 1976

TETON DAM FAILURE EXHIBITS - Continued

- No.
- 23.1 Analysis of Eye Witnesses to Teton Dam Failure, June 5, 1976, dated July 2, 1976 plus three more accounts
 24. Denver Laboratory Test Data entitled "Sample Index Sheets"
 25. Observation Well Maps (Readings through June 20, 1976)
 26. Slurry and Grout Used to Fill Cracks & Fissures in Abutment (Six pages)
 27. Drawings
 - 27.1 549-D-5 Location Map
 - 27.2 549-D-6 Vicinity Map
 - 27.3 549-D-8 General Plan and Sections
 - 27.4 549-D-9 Embankment Details
 28. Set of Six Grout Summary Sheets - Main Dam - Final Quantities (taken from October 25, 1975 L-10 Report)
 29. Preliminary Report on Failure of Teton Dam, by Harold G. Arthur
 30. Pressure Grouting Foundation on Teton Dam, by Peter P. Aberle
 31. Questions and Answers Concerning the Failure of Teton Dam - prepared by the Bureau of Reclamation
 32. Foundation Grouting Profile and Plan Drawings - Drawings No. 549-147-150 through -195 (with index)
 33. Preconstruction Geologic Report, Teton Damsite, April 1971
 34. Photographs of Teton Dam construction and prefailure (from project files)
 35. Volume of material washed away by failure of Teton Dam - dated 6-18-76

TETON DAM FAILURE EXHIBITS
(Added Subsequent to July 8, 1976)

No.

- 36. Sequence of Failure Photographs (taken by Gibbons and Reed employee)
- 37. Chronology of Failure (from Interim Report of Interior Teton Dam Failure Review Group)
- 38. Aerial Photographs (Only one set available. Furnished to Mr. Jansen for panel use, 7/27/76)
- 39. Teton Dam Earthwork Control Data Book
- 40. Teton Dam Earthwork Control Statistics, Zones 1 and 3
- 41. Map showing Observation Wells located near Teton Dam

10/21/76

(Exhibits added subsequent to July 30, 1976)

- 42. Transcript of Hearings before Conservation, Energy, and Natural Resources Subcommittee of the Committee on Government Operations, House of Representatives, Congress of the United States, August 5, 1976
- 43. Transcript of Hearings before Conservation, Energy, and Natural Resources Subcommittee of the Committee on Government Operations, House of Representatives, Congress of the United States, August 6, 1976
- 44. Transcript of Hearings before Conservation, Energy, and Natural Resources Subcommittee of the Committee on Government Operations, House of Representatives, Congress of the United States, August 31, 1976
- 45. Prepared Statement of Robert R. Curry Presented to Conservation, Energy, and Natural Resources Subcommittee Hearing, August 5, 1976
- 46. Prepared Statement of Marshall K. Corbett Presented to Conservation, Energy, and Natural Resources Subcommittee Hearing, August 5, 1976
- 47. Prepared Statement of H. Anthony Ruckel Presented to Conservation, Energy, and Natural Resources Subcommittee Hearing, August 5, 1976
- 48. Prepared Statement of Robert W. James Presented to Conservation, Energy, and Natural Resources Subcommittee Hearing, August 5, 1976
- 49. Prepared Statement of R. R. Robison Presented to Conservation, Energy, and Natural Resources Subcommittee Hearing, August 6, 1976
- 50. Prepared Statement of H. G. Arthur Presented to Conservation, Energy, and Natural Resources Subcommittee Hearing, August 6, 1976
- 51. Prepared Statement of Gilbert G. Stamm Presented to Conservation, Energy, and Natural Resources Subcommittee Hearing, August 31, 1976

TETON DAM FAILURE EXHIBITS - Continued

- No.
52. Teton Dam Disaster - Thirtieth Report by the Committee on Government Operations to the 94th Congress, Based on a Study Made by its Conservation, Energy, and Natural Resources Subcommittee, September 23, 1976 (House Report No. 94-1667)
 53. Summary of Bureau of Reclamation Comments on Testimony Presented to Conservation, Energy, and Natural Resources Subcommittee of the House Committee on Government Operations
 54. Seismicity of the Teton Dam Area, June 16, 1974-June 9, 1976, by R. Navarro, G. Wuolet, J. West, K. King, and D. Perkins (Open File Report 76-555)
 55. Drawing No. 549-125-268, Geologic Cross Section Along Spillway Site, Revised April 1976
 56. Transcript of Meeting of Teton Dam Failure Review Group, Idaho Falls, Idaho, September 15, 1976

APPENDIX B

PANEL CORRESPONDENCE



United States Department of the Interior

OFFICE OF THE SECRETARY
WASHINGTON, D.C. 20240

TO SECRETARY

In Reply Refer To:
LBR 501/930.4

JUN 11 1976 JUN 11 1976

Mr. Wallace L. Chadwick
Suite 904
523 West 6th Street
Los Angeles, California 90014

FOR SIGNATURE

Dear Mr. Chadwick:

This will confirm discussions with you concerning your appointment to a non-Federal panel for the independent review of the causes of the Teton Dam failure.

We are establishing this panel as a joint undertaking with Governor Cecil D. Andrus of Idaho. The Governor and I would appreciate your serving as chairman. The following members will serve with you:

Mr. Ralph B. Peck
1101 Warm Sands Drive, SE.
Albuquerque, New Mexico 87123
tel. 505-293-2484

Mr. Arthur Casagrande
Pierce Hall
Harvard University
Cambridge, Massachusetts 02138
Home address: 16 Rockmont Road
Belmont, Massachusetts 02178
tel. 617-495-2843

Mr. Thomas M. Leps
177 Watkins Avenue
Atherton, California 94025
tel. 415-325-9032

Mr. H. Bolton Seed
Professor of Civil Engineering
University of California
441 Davis Hall
Berkeley, California 94672
tel. 415-642-1262

Mr. R. Keith Higginson
Director, Idaho Department
of Natural Resources
373 West Franklin Street
Boise, Idaho 83702
tel. 208-384-2215

Mr. E. Montford Fucik
President and Chairman
Harza Engineering Company
150 South Wacker Drive
Chicago, Illinois 60606
tel. 312-855-7000

Mr. Munson W. Dowd
Chief Engineer
Metropolitan Water District of
Southern California
1111 Sunset Boulevard
Los Angeles, California 90012
tel. 213-626-4282

Your appointment papers will be forwarded separately. In the meantime, you may make arrangements to meet with the panel members and plan your review. The Department of the Interior will finance all expenses associated with your independent investigation. Upon completion of your review, the panel should report its findings and recommendations simultaneously to Governor Andrus and the Secretary.

We thank you for undertaking this important public service.

Sincerely yours,

(Sgd) Thomas S. Kleppe

Secretary of the Interior

cc: Messrs. Peck, Leps, Casagrande, Seed, Higginson, Fucik, and
Dowd
Governor Cecil D. Andrus

bcc:
Secretary's Files
Secretary's Reading Files--RECLAMATION (2)
Assistant Secretary - LW
Under Secretary
Regional Director, Boise, Idaho
Director of Design and Construction, ESR Center, Denver, Colorado

W.O. Code 100, 105
LER:JA Healy:vs:6/10/76:x4626

ON MAG CARD II



United States Department of the Interior

OFFICE OF THE SECRETARY
WASHINGTON, D.C. 20240

RECEIVED

JUN 31 1976

W. L. CHADWICK

JUN 11 1976

Dr. Howard A. Coombs
3856 46th Avenue, NE.
Seattle, Washington 98105

Dear Dr. Coombs:

This will confirm discussions with you concerning your appointment to a non-Federal panel for the independent review of the causes of the Teton Dam failure.

We are establishing this panel as a joint undertaking with Governor Cecil D. Andrus of Idaho. The Governor and I appreciate your willingness to serve. The following are the other panel members:

Mr. Wallace L. Chadwick, Chairman
Suite 904
523 West 6th Street
Los Angeles, California 90014
tel. 213-623-6954

Mr. Arthur Casagrande
Pierce Hall
Harvard University
Cambridge, Massachusetts 02138
Home address: 16 Rockmont Road
Belmont, Massachusetts 02178
tel. 617-495-2843

Mr. Thomas M. Leps
177 Watkins Avenue
Atherton, California 94025
tel. 415-325-9032

Mr. H. Bolton Seed
Professor of Civil Engineering
University of California
441 Davis Hall
Berkeley, California 94672
tel. 415-642-1262



WLD

Mr. R. Keith Higginson
 Director, Idaho Department
 of Natural Resources
 373 West Franklin Street
 Boise, Idaho 83702
 tel. 208-384-2215

Mr. E. Montford Fucik
 President and Chairman
 Harza Engineering Company
 150 South Wacker Drive
 Chicago, Illinois 60606
 tel. 312-855-7000

Mr. Munson W. Dowd
 Chief Engineer
 Metropolitan Water District of
 Southern California
 1111 Sunset Boulevard
 Los Angeles, California 90012
 tel. 213-626-4282

*Dr. Peck not included
 in history
 ES.*

Your appointment papers will be forwarded separately. In the meantime, Mr. Chadwick will be in touch with you and the other panel members to plan your review. The Department of the Interior will finance all expenses associated with the panel's independent investigation. Upon completion of the review, the panel should report its findings and recommendations simultaneously to Governor Andrus and the Secretary.

We thank you for undertaking this important public service.

Sincerely yours,

(sgd) Tom Kleppe

Secretary of the Interior

Enclosure

cc: / Mr. Chadwick, Chairman
 Governor Cecil Andrus
 Messrs. Casagrande, Leps, Seed, Higginson, Fucik, and Dowd
 (NOTE: Dr. Coombs' tel. no. is 206-522-9242)

LOS ANGELES, CALIFORNIA
JUNE 11, 1976

MR. HAROLD G. ARTHUR, DIRECTOR
DESIGN & CONSTRUCTION
USBR, BUILDING 67
DENVER FEDERAL CENTER
DENVER, COLORADO 80225

UNABLE TO REACH YOU BY TELEPHONE. HAVE BEEN ADVISED BY DON GIAMPOLI THAT I ~~SHOULD~~ CONVENE PANEL WHICH HAS BEEN APPOINTED TO REVIEW TETON DAM PROBLEM AND THAT YOU WILL BE CONTACT FOR THAT PANEL WITH USBR. TRYING TO ASSEMBLE GROUP FOR WEEK, JUNE 28 AT PLACE AND TIME TO BE DETERMINED. SUGGEST THAT PANEL WILL BE GREATLY AIDED BY HAVING A DATA BOOK AVAILABLE IF POSSIBLE BEFORE MEETING. THAT BOOK TO PRESENT FOLLOWING INFORMATION (1) SITE GEOLOGY IN PLAN AND SECTIONS WITH ANY TEST RESULTS ON FOUNDATION MATERIALS (2) SITE EXPLORATION WITH DETAIL OF DRILL LOGS, EXPLORATION TRENCHES, BORROW MATERIALS AND TESTS, (3) GROUT RECORDS IN DETAIL SHOWING NON-AVERAGE TAKES BY LOCATION AND DEPTHS, PATTERNS USED AND RECORD OF ANY INTER-CONNECTIONS, (4) FOUNDATION PREPARATION SHOWING BEFORE AND AFTER CONDITIONS (5) DESIGN MEMORANDA FOR EMBANKMENT, SPILLWAY, DIVERSION STRUCTURES AND OUTLETS, (6) BASIC DRAWINGS AND TECHNICAL SPECIFICATIONS, (7) ANY OUTSIDE REPORT RE SITE OR DESIGNS, (8) CONSTRUCTION HISTORY, BORROW PITS, HAULING, PLACEMENT, PROGRESS, INSPECTION, IN PLACE TESTS, (9) ANY SEEPAGE MEASUREMENTS OR OBSERVATIONS, (10) EYEWITNESS ACCOUNTS, PROGRESS OF FAILURE.

WILL APPRECIATE YOUR CALL RE THESE SUGGESTIONS, AREA CODE 213-623-6954, ADDRESS 904 PACIFIC MUTUAL BUILDING, 523 WEST SIXTH STREET, LOS ANGELES, CALIFORNIA 90014.

W. L. CHADWICK

LOS ANGELES, CALIFORNIA

JUNE 14, 1976

HAROLD ARTHUR, DIR. OF DESIGN & CONSTRUCTION
USBR, BUILDING 67 - DENVER, FEDERAL CENTER
DENVER, COLORADO 80225

REFERENCE MY TELEGRAM JUNE 11 RE DATA FOR TETON DAM PANEL, PLEASE
ADD FOLLOWING:

1. HYDROLOGY
2. SEISMICITY
3. DRAIN DESIGNS AND DRAINAGE OBSERVATIONS
4. ANY CHANGES IN SPILLWAY OR AUXILIARY OUTLET STRUCTURES
5. ANY CHANGES IN PRECISE LEVEL OR HORIZONTAL CONTROL SURVEY POINTS
6. CHANGES TOPOGRAPHY UP AND DOWN STREAM
7. PHOTOS OF FOUNDATION AS APPROVED ^{AT} ~~AND~~ START OF EMBANKMENT,
PARTICULARLY IN CUTOFF TRENCH
8. RECORD OF ANY SEEPS OR SPRINGS IN CUTOFF AND CORE CONTACT AREA
AND
9. RECORD COFFERDAM SEEPAGE AND PUMPAGE FROM FOUNDATION AREA.

THANKS.

W. L. CHADWICK

Dictated to me 6/14/76 from Dorval/Montreal Airport
8:40 A.M. - Called in to W. U. 9:00 A.M.

COPY TO DONALD A. GIAMPOLI

366



STATE OF IDAHO
DEPARTMENT OF WATER RESOURCES

Cecil D. Andrus
Governor

R. Keith Higginson
Director

Statehouse
Boise, Idaho 83720
(208) 384-2215

June 21, 1976

OFFICIAL FILE COPY	JUN 23 1976	
INFORMATION COPY TO:	1310 Hwy	
	ROUTE TO	DATE
	1 200	5/2/76
	210	

Mr. Wallace L. Chadwick
904 Pacific Mutual Building
523 West Sixth Street
Los Angeles, CA 90014

Dear Mr. Chadwick:

In addition to the information you have requested, I would like the Bureau to furnish the panel with the following:

- (1) Operating criteria for outlet works during period of initial filling.
- (2) Statement of condition and operability of outlet works on date of failure.
- (3) History of reservoir filling from date of closure of diversion to failure giving reservoir contents, rate of rise of water level and water surface elevations.

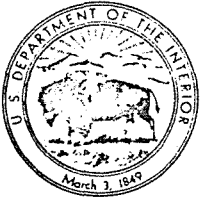
I would also like some information concerning the decision-making process and authority within the Bureau of Reclamation during the construction of a dam such as Teton.

Very truly yours,

R. KEITH HIGGINSON
Director

RKH/slg

CC: Mr. Harold Arthur



United States Department of the Interior

OFFICE OF THE SECRETARY
WASHINGTON, D.C. 20240

JUN 23 1976

Dear Mr. Chadwick:

You may already be aware of an Interior Department Teton Dam Failure Review Group. In addition to establishing with Governor Andrus your independent Blue Ribbon Panel, I felt it was appropriate to establish this "in-house" group to determine the cause of failure and recommend measures to prevent recurrences of such failures.

I have appointed Dennis N. Sachs, Deputy Assistant Secretary for Land and Water Resources, as Chairman of the Review Group. I have drawn other members of the Review Group from the Bureau of Reclamation, Geological Survey, Tennessee Valley Authority, Corps of Engineers, and the Soil Conservation Service.

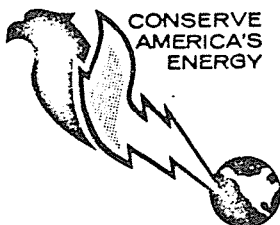
I envision your Blue Ribbon Panel and the Interior Review Group conducting their investigations essentially independent of each other. Nonetheless, there may be certain matters related to the investigations about which the two groups should consult. I have, therefore, directed Mr. Sachs to meet with you at your earliest convenience to discuss whatever coordination may be appropriate.

Thank you for your continued cooperation.

Sincerely yours,

Secretary of the Interior

Mr. Wallace L. Chadwick
Suite 904
523 West 6th Street
Los Angeles, California 90014

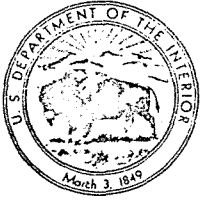


Save Energy and You Serve America!

U.S. DEPARTMENT OF THE INTERIOR

JUN 23 1976

W. L. CHADWICK



United States Department of the Interior

OFFICE OF THE SECRETARY
WASHINGTON, D.C. 20240

RECEIVED

JUL 1 1976

W. L. CHADWICK

JUN 30 1976

Chad
Dear Mr. Chadwick:

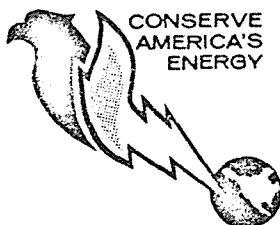
This will confirm our discussion of June 23, 1976, concerning the charge to the independent panel for its review of the Teton Dam failure.

The panel should determine the cause of the failure of Teton Dam. In so doing, the panel should examine, among other matters relative to the cause of the failure, the following:

- (1) The geology of the site.
- (2) Seismicity of the site.
- (3) Preconstruction investigation.
- (4) Embankment construction materials.
- (5) Embankment designs.
- (6) Embankment construction and construction control.
- (7) Foundation design.
- (8) Foundation construction and construction control.
- (9) Reservoir filling.
- (10) Measures taken to monitor the safety of the dam.
- (11) Reaction of Reclamation personnel to the emergency.
- (12) Status of outlet works construction and ability to pass Teton river water.
- (13) Such other matters that the panel may determine appropriate to its charge.

If the facts warrant, the panel should also make findings as appropriate with regard to measures that can be taken to avert recurrence of failure.

The findings of the panel are needed as soon as possible. To that end, the entire resources of the Department of the Interior are available to the panel. Also, the panel will be authorized to secure the services of other organizations as may be required to reach its findings.

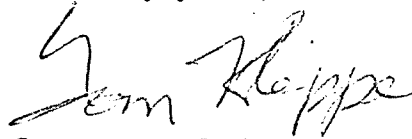


Save Energy and You Serve America!

The panel is requested to provide me and Governor Andrus with a preliminary report by August 1, 1976, and with status reports by the first of each month following until the final report is made.

The Governor concurs with the charge and other arrangements indicated in this letter. We thank you for undertaking this important service.

Sincerely yours,

A handwritten signature in cursive script that reads "Sam Klippe". The signature is written in dark ink and is positioned above the typed name.

Secretary of the Interior

Mr. Wallace L. Chadwick
Suite 904
523 West Sixth Street
Los Angeles, California 90014

July 2, 1976

Honorable Thomas J. Kleppe, Secretary
United States Department of the Interior

Honorable Cecil D. Andrus, Governor
State of Idaho

Gentlemen:

The undersigned non-Federal Panel for Independent Review of the Causes of the Teton Dam Failure has proceeded with your charge. In doing so it has organized for continuing its investigation, including appointment of Mr. Robert B. Jansen as the Panel's Executive Director. Procedural details are being developed for implementation.

The Panel met in Denver for one and one-half days for the purpose of obtaining from the Bureau of Reclamation and the United States Geological Survey information and data of relevance to the failure. Response was free and unrestrained. As a result, the Panel has received a large mass of pertinent technical data, information and reports to review and analyze. The Panel also spent two days, except for travel time, at the dam site. During travel to the site from Idaho Falls the extent and nature of the downstream damage were observed. At the site, inspections of general, construction, and geological conditions were observed during numerous helicopter over-flights. Additional and closer inspections were made of the left and right abutments, while walking and climbing along them. Inspections were also made from a powered boat.

A major portion of one day at the site was devoted to examination of data and photographs and obtaining personal statements concerning many construction details, including the manner in which the key and cutoff trenches were treated prior to placement of overlying embankment.

Only tentative hypotheses of causes of failure can be considered at this time, because of the need to study all of the various factors which may support each particular hypothesis or negate it. At this time, however, it seems apparent that the failure resulted from piping. This is a process by which embankment material is eroded internally and transported by water flowing through some channel. Piping may be initiated by several detailed causes and, unfortunately, most of the direct evidence appears to have been destroyed by the violence of the failure itself. The Panel is planning to examine all obtainable evidence in detail and has prepared a program of field explorations to pinpoint, if possible, which of the following potential causes is responsible:

1. Massive seepage through the grout curtain, impinging forcibly against the contact between the downstream part of the dam and the rock abutment.
2. Piping through the core at the core-to-rock contact at the right abutment.
3. Piping through the core at levels above the base of the keyway core-to-rock contact.
4. Piping through a transverse tension crack in the core in the right abutment area.
5. Massive seepage around the end of the grout curtain, directed by the foundation joint system against the contact between the downstream part of the dam and the rock abutment.

The Panel will continue its investigation independently, and in accord with the best professional practice. This necessarily requires weighing of many data and many factors by full analysis and free exchanges within the Panel. Your need for an early preliminary report is understood. The Panel will meet again during the week of August 2 to review the data, observations and information developed in the interim, anticipating a preliminary report to you early in August.

Included in the Panel's on-going inquiry will be a public fact-finding meeting in Idaho Falls prior to its next session, soliciting any relevant information as to the cause of the Teton Dam failure.

It is intended to establish a temporary office of the Panel in Idaho Falls at an early date.

The Panel is presenting today to the Director, Design & Construction of the Bureau of Reclamation, a list of exploratory work which will be necessary for its additional information. In doing so the Panel wishes to release the Bureau of Reclamation from any further restraint on all site-changing physical work which the Bureau considers necessary to reduce hazards to safety.

The support you have given to the Panel is greatly appreciated, as also is the excellent cooperation of all members of the United States Bureau of Reclamation and Geological Survey of whom the Panel has inquired.

Respectfully submitted,

Wallace L. Chadwick, Chairman
Independent Panel for Review of
Teton Dam Failure

July 2, 1976

Mr. Harold G. Arthur
Director of Design and Construction
Bureau of Reclamation
Denver Federal Center
Denver, Colorado 80225

Re: Teton Dam Investigation

Dear Mr. Arthur:

The following activities represent the Panel's highest priority and are recommended for immediate implementation. It should be recognized that additional activities will be proposed in the coming months.

1. The remnant of the right-abutment keyway fill to the left of the spillway should be excavated to permit inspection of conditions below Elevation 5301. Down to Elevation 5301 the remnant can be removed in any manner that will not disturb the material below. Below Elevation 5301 the remnant can be removed in any stages and by any means, provided that a width of undisturbed material remains with a minimum horizontal thickness of five feet on each side and a minimum vertical distance of ten feet above the bottom of the original trench. The material within the five-foot envelope on each side should be removed by hand, where directed by the Panel's representative, as required to permit appropriate sampling to allow description of conditions of soil, rock, and any joint treatment disclosed by the excavation, to allow observation of any indications of piping or other defects. The bottom ten feet should be removed in two lifts. These lifts should be preceded by excavating trenches at places selected by the representative of the Panel to a depth of five feet with appropriate sampling and observation.

2. Any debris remaining on the face of the central part of the abutment, especially where the grout cap remains intact, should be carefully cleaned to permit detailed inspection.

3. The area of the lower spring (50 cfs.) should be exposed. Any original material still in place should be left undisturbed. The details of jointing of the rock in this area should be carefully examined.

4. All steps necessary to assure safety at the remaining left section of the dam can be carried out promptly.

5. In order to provide some quantitative evaluation of permeability in the rocks in the right abutment, detailed studies should be made on enlarged photographs of representative areas of each joint type near the keyway.

Total footage of open joints per unit of area (e.g., one square yard) should be determined by direct measurements on enlargements of the photos, using a reliable scale with which a grid system is drawn on the enlargement.

The details of this survey, including best lighting (either direct sun during the forenoon or on a cloudy day) should be developed in a pilot program.

6. An item of prime importance is the nature of the joint system in the right abutment on either side of the keyway. Particularly important is the identification of major, throughgoing joints on the downstream side of the keyway that might provide access of water to the embankment.

Primary and secondary joint systems should be plotted on a new topographic map. Symbols may be used to indicate wide and continuous joints in contrast to the numerous, smaller joints. Any evidence of springs or water-courses along or through the joints should be indicated on the joint map.

7. On the basis of the evidence presented to it by the U.S. Geological Survey, the Panel does not consider that the failure was in any way related to seismic activity in the vicinity of the site. There is no record of significant seismic activity at the site either on the day of the failure or in the year preceeding the failure. No additional investigations of seismicity, other than those currently in progress by the U.S.G.S., are recommended.

Sincerely yours,

Wallace L. Chadwick, Chairman
Independent Panel for Review of
Teton Dam Failure



United States Department of the Interior

BUREAU OF RECLAMATION
OFFICE OF DESIGN AND CONSTRUCTION
ENGINEERING AND RESEARCH CENTER
P.O. BOX 25007
BUILDING 67, DENVER FEDERAL CENTER
DENVER, COLORADO 80225

IN REPLY
REFER TO: 222
510.

Memorandum

JUL 8 1976

To: Mr. Wallace L. Chadwick, Chairman, Independent Panel for
Review of Teton Dam Failure

From: Director of Design and Construction

Subject: USBR Reply to R. Keith Higginson Letter

A further review of our correspondence reveals that an error was made in response to Question No. 2 of Mr. Higginson's June 21, 1976 request to Mr. Wallace L. Chadwick for further information on Teton Dam.

The initial response to Question No. 2 had indicated that the electrical power was not available for operating the river outlet works gates; however, power was available for operating the gates on and following May 17, 1976.

It was not possible to immediately operate the river outlet works gates on June 5, 1976, due to the contractor's sandblasting and painting operations for the downstream liner of the river outlet works.



JUL 29 1976

WALLACE L. CHADWICK
904 PACIFIC MUTUAL BUILDING
523 WEST SIXTH STREET
LOS ANGELES, CALIFORNIA 90014

July 24, 1976

Mr. Harold G. Arthur, Director of Design and
Construction, United States Bureau of Reclamation
Building 67, Denver Federal Center
Denver, Colorado 80225

Dear Mr. Arthur:

Reference is made to Secretary Kleppe's letter
of June 30, 1976 supplementing his and Governor Andrus'
original charge to the Independent Panel to Review the
Cause of Teton Dam Failure, particularly to items 9,
10 and 12.

It will be a great help to the Panel if you can
furnish the following additional information, or if pre-
viously supplied, give references to facilitate ready find-
ing:

1. Were 1976 runoff forecasts made during March, April,
and May, for use in estimating the Teton reservoir
filling rate, and comparing the expected rate with
the actual.
2. Was an operating rule curve developed for use in pro-
gramming reservoir filling and releases, particularly
any releases required to control filling rate.
3. What schedule was used for progressing erection of the
river outlet gates and controls, the auxiliary outlet
gates and controls, and the spillway gates and controls.
On what dates were such facilities completed and ready
for use. Copies of any original schedules and pro-
gressive changes will be appreciated.

Mr. Harold S. Arthur

July 24, 1976

Page 2

4. When was each gate hoist commissioned?
5. When and how frequently were walkover surveillance inspections made as the reservoir filled. Copies of daily logs or diary entries of each surveillance inspection will be appreciated. In the absence of logs or other records, a written statement of the inspector or inspectors will be helpful.

Thank you for your cooperation.

Very truly yours,



WLC:ecs

cc: Panel Members /
R. B. Jansen ✓

UNITED STATES DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Wallace L. Chadwick, Chairman
Arthur Casagrande
Howard A. Coombs
Munson W. Dowd
E. Montford Fucik
R. Keith Higginson
Thomas M. Leps
Ralph B. Peck
H. Bolton Seed
Robert B. Jansen, Executive Director

August 5, 1976

Honorable Thomas S. Kleppe, Secretary
United States Department of the Interior
Interior Building
Washington, D.C. 20240

Honorable Cecil D. Andrus, Governor
State of Idaho
Capitol Building
Boise, Idaho 83720

Gentlemen:

The Independent Panel to Review Cause of Teton Dam Failure has continued its work under your charge. The Panel conducted technical working sessions in Idaho Falls August 3 through 5, 1976, with all members present. Included in these sessions was a visit to the damsite on August 3, 1976, to review the progress which has been made to date in the exploration being performed on the right abutment by the Bureau of Reclamation for observation by the staff of the Panel.

The following is a report of the progress which has been made by the Panel since its report to you dated July 2, 1976.

Organization

A small but capable staff has been assembled, based both at the site and in Idaho Falls, under the direction of Mr. Robert B. Jansen. The cooperation of Governor Andrus, and of Governor Brown of California in making Mr. Jansen available for this important responsibility, is much appreciated. Mr. Clifford J. Cortright has been actively at work as staff engineer since July 3. Likewise, Laurence B. James is at work as staff geologist. The entire staff has unique experience and expertise with which to serve the Panel.

Through the assistance of Secretary Kleppe and the Bonneville Power Administration, a secretary-office manager and two technicians have been provided for temporary aid to the Panel. In addition, Mr. Higginson has

provided a geologist from his staff to assist the Panel. All this assistance, as well as the support and help received from numerous people in the Secretary's office, is much appreciated by each member of the Panel. Also, the Panel appreciates the full cooperation it has received from the Bureau of Reclamation.

Public Meeting

On July 21, 1976, the public was invited to bring into either of two public meetings, conducted in Idaho Falls, information regarding pertinent first-hand observations prior to the failure of the dam on June 5, 1976. Response was somewhat disappointing because only four individuals testified. A transcript was taken.

Site Work

On July 16, 1976, the Bureau of Reclamation awarded Contract No. DC-7232 to Gibbons & Reed to carry out, among other things, work requested by the Panel in its letter of July 2, 1976 to Mr. H. G. Arthur. Work under that contract was started at the site on July 23. On August 3, at the time of the Panel's visit, the right remnant of Teton Dam had been removed to about Elevation 5301 and the first exploratory trenches had been cut, permitting the first in-situ observations. Initial progress has been good. Work was also in progress directed toward uncovering the downstream portal of the auxiliary outlet tunnel. The proposed detailed mapping of right abutment bedrock joint systems has progressed well.

Accomplishments of the Panel

Since its last meeting, the Panel members have reviewed the extensive documentation received from various sources and the Bureau of Reclamation. A fine chronological photographic record has been compiled showing the progressive development of seepage on June 5. Unfortunately, to this date, no photographs are available of the early development of this seepage. Search will be continued for such photographs.

A chronological statement has been compiled of the sequence of observations by various individuals during the period from June 3 to late in the day of June 5, 1976.

Following a detailed discussion of the five hypotheses which were enumerated in the Panel's report to you dated July 2, 1976, the Panel developed a schedule for specific laboratory and field tests and for analyses which will be of assistance in reaching conclusions. A copy of that schedule is attached.

August 5, 1976

A preliminary finite element analysis has been made, for the Panel, to indicate possible stress conditions across the embankment at Station 14+00. The results of this analysis were used as a basis for planning the further analyses included in the attached schedule.

Hydrographic studies have been made through the offices of Mr. Higginson seeking to relate (1) the 1976 runoff expectancy to historic flows, (2) the Teton Dam project's expected rate of reservoir filling, and (3) the actual rate of filling. Such studies will be continued and related to historic reservoir operation.

The Panel has scheduled its next technical working session for October 4, 5, and 6 in Idaho Falls.

The Panel appreciates your continuing interest and support.

Very truly yours,

Chairman

Encl.

SCHEDULE FOR LABORATORY AND FIELD TESTS AND ANALYSIS

APPENDED TO PANEL REPORT OF AUGUST 5, 1976

A. Purpose

In its report of July 2, 1976, the Panel listed five potential causes of the piping failure of Teton Dam, and on the same date, in a letter to the Director, Design and Construction of the Bureau of Reclamation, listed items of highest priority recommended for action by the Bureau to provide data for choosing among the potential causes. In its deliberations during its meeting of August 3-5, the Panel concluded that the field evidence virtually excludes massive seepage around the end of the grout curtain as a likely cause. Accordingly, the following detailed program was developed to aid in discriminating among the other four hypothetical causes, namely whether the massive seepage or piping took place (1) through the grout curtain, (2) through the core at the core-to-rock contact, (3) through the core above the base of the keyway core-to-rock contact, or (4) through a crack in the core. The program is in part a particularization of the work recommended on July 2, and in part a supplement to that work.

B. Investigation of Bottom of Key Trench and Grout Curtain

The purpose of the program is twofold: first, to determine if any cracks encountered in the rock in the bottom of the key trench, either up- or downstream, are open enough to permit flows of water through them; and second, to test the watertightness of the grout curtain under the grout cap and under the spillway. The section of the key trench to be tested extends from Station 12+50 to 14+50.

To test the water-carrying characteristics of cracks in the bottom of the key trench, it is proposed to pond water over selected cracks and observe the drop in the level of ponds. Each pond can be formed by placing a dike of stiff mortar on the low side of the crack, high enough to produce a depth of water of about 6 inches over the crack. Visual observation of the loss of water will permit a rough idea of whether the crack is relatively open or tight. At open cracks, an approximate measurement should be made of the outflow per linear foot of crack per minute. It is suggested that the wider cracks be tested first, and then the narrower ones.

Tests should be made both upstream and downstream of the grout cap. It is envisioned that between 10 and 20 representative cracks should be tested in the proposed section. The cracks tested should be distributed throughout the length of the section. If most of the cracks leak substantially, additional tests might be made to verify the conclusion that most cracks would transmit water easily.

To test the watertightness of the grout curtain, it is proposed to drill through the grout cap and the spillway crest into the rock below, and to water-test these holes. The holes should preferably be of AX size and cores should be obtained from each hole to permit observation of any grout that may fill cracks in the rock. The holes through the grout cap should be drilled to a depth of 10 feet below the bottom of the grout cap, water tested, drilled 10 feet more and tested again. If pressure is used, it should not exceed 10 psi at the collar. The rate of flow in each stage of the hole should be recorded. If the second stage of any hole shows large leakage, a third 10-foot stage should be drilled and tested.

It is suggested that tests be carried out on the centerline of the grout curtain approximately at Stations 12+65, 13+05, and 13+40. At each station, three holes should be drilled, one vertical, one inclined 22-1/2° from the vertical toward the abutment, and one inclined 45° into the abutment. At each location, three holes should be drilled, in each stage, before starting the water testing.

It is also suggested that holes be drilled at about the center of each of the three spillway bays. Three holes should be drilled at each location, one vertical, one at an angle of 30° toward the river, and one at an angle of 30° away from the river. The holes through the spillway crest should be drilled and water-tested in three stages of 25 feet each, so that the grout curtain will be tested to the depth of the adjacent key trenches.

If large water takes are observed at any location, additional holes should be drilled on each side to determine the extent of the open zone.

C. Investigation of Key-Trench Fill

As the key trench fill on the right abutment is excavated in accordance with the Panel's recommendation of July 2, detailed studies should be made of the variations in the degree of compaction of the fill material by penetration tests, and samples should be taken for investigation of erosion resistance, stress-strain characteristics, and such other purposes as may become desirable as the investigation proceeds. The specific studies are as follows:

1. Field Investigations and Routine Laboratory Tests

a. Observations and Sampling in Trenches

Immediately upon completion of excavation of an approximately 30-foot long section of exploratory trench, the following observations and sampling should be performed:

With a shovel or spade, make a fresh exposure by removing a vertical slice at least one inch thick, at locations spaced approximately 7 to 8 feet. In this fresh exposure make a rapid survey of variations in consistency along a vertical line, using a screwdriver or other convenient hand tool; also examine variations in types of materials; then perform penetration tests with the Proctor Needle on several representative layers, to define the entire range of strengths, with special attention to the weakest layers or lenses. For the penetration tests on the weakest materials, it will probably be necessary to use the largest diameter "point". Prepare a log of all observations and penetration tests, including thickness of representative layers.

To facilitate recording the logs, it will be desirable to develop a simple classification system which should be based on the BR test data of the Zone 1 fill and on initial experience in surveying the trenches.

b. Sampling

(1) Hand-cut block samples. Samples, usually about 8 inches square and about 12 inches high, should be taken of representative materials, but with particular emphasis on the weakest materials. Usually three such samples should be taken at each location, side by side, of material that is essentially similar.

Each sample should be wrapped in Saran wrap, or similar plastic film, and then covered with at least a 1/4-inch thick layer of microcrystalline wax by dipping several times into the wax melted to the correct temperature. (Do not overheat the wax, which would change its properties.) Use a grade of wax as used in soils laboratories for such purposes. Then place a clearly written identifying label on one side of the sample and again wrap in one layer of plastic film, taking care to place the film smoothly over the label to ensure that it can be read easily.

(2) A Bag Sample should be taken at each location where block samples are taken and placed in a plastic bag which is closed tight. Usually about 10 lb. will be sufficient.

(3) Storage of Samples should be in a shed with appropriate shelves to provide space for samples taken from an estimated 100 locations and equipped with a humidifier (to maintain humidity at greater than 80% relative humidity) and heated in winter to a temperature above 40°F.

c. Observation of Features That May be Related to Potential or Actual Piping.

Special attention must be paid to careful observation of fissures, holes, and any signs indicating that the originally placed fill was disturbed. Such features should be identified, sketched, described and photographed. Particular care should be exercised in identifying such

features immediately adjacent to the downstream rock face and the bottom of the key trench. If such features are discovered, it will be necessary to proceed with the greatest of caution in further excavation to protect vital evidence of erosion. At such junctures, the field staff will have to make ad hoc decisions how to proceed. Mr. Jansen should be notified immediately. When particularly meaningful discoveries are made, Mr. Jansen will confer by telephone with available geotechnical panel members.

d. Laboratory Tests

Preferably in a field laboratory, the following tests should be performed on representative samples:

- (1) Natural water content.
- (2) Grain size analyses.
- (3) Liquid and plastic limit tests. (Report actual test results; not the computed plasticity index in lieu of the measured plastic limit.)
- (4) Unconfined compression tests.

e. Miscellaneous Comments

The depth of the exploratory trenches should not exceed 6 feet to facilitate operations.

During removal of fill immediately adjacent to the rock slopes of the key trench, all loose rock should be removed to ensure safety of the men who will work later at lower levels.

2. Evaluation of Erosion Potential of Zone 1 Material

In view of the fact that the failure of Teton Dam has already been attributed to internal erosion of the Zone 1 material, it is important to establish the vulnerability to erosion of this particular material in comparison with that of other soils customarily used as core materials. This is particularly true since visual inspection and classification-test data of Zone 1 materials would appear to indicate that these soils would be highly susceptible to erosion.

To establish the erosion potential of this soil, it is recommended that selected samples be sent to two laboratories for independent evaluations as follows:

- a. A series of 10 samples should be sent to the Waterways Experiment Station at Vicksburg, Mississippi, for performance of the pinhole test as now standardized by that laboratory. Grain-size distribution curves and liquid and plastic limit values should be determined for

each of the test samples and the results used to establish the relative erodibility of Teton Dam Zone 1 materials.

b. A series of 10 samples should be sent to a second laboratory specializing in measuring the erosion potential of soils (e.g., the Soil Mechanics Laboratory of the University of California at Davis) where the erodibility can be evaluated and compared with data for other soils by means of two or more appropriate types of tests. As before, grain-size distribution curves and liquid and plastic limit values should be determined for each test sample.

In all cases, the erosion tests should be performed on the undisturbed block samples cut from the right abutment key trench. The selected samples should be representative of the range of materials and densities found in the trench, with particular emphasis on materials that appear to be most erodible, as established in the field survey. To the extent practicable, the two independent laboratories should be sent similar suites of samples.

3. Determination of Stress-Strain Characteristics for Use in Finite-Element Analyses

To determine the possibility of hydraulic fracturing or of crack formation in the Zone 1 material, it is desirable to evaluate the stress distribution within Zone 1. This can best be achieved by finite-element analyses incorporating realistic representations of the stress-strain characteristics of the compacted loessial soil used to fill the key trenches and to form the main core of the embankment.

The stress-strain properties should be determined by several series of drained triaxial compression tests on representative samples cut from the Zone 1 section of the dam. At least 3 series of tests should be performed, each series including one test at each of four confining pressures, approximately 15, 40, 70, and 100 psi. Samples should be 1.4 inches in diameter and approximately 3-1/2 inches high and should not be saturated before testing. Stress-strain relationships should be recorded up to the point of failure.

At least one series of the drained tests should be conducted by stress-control techniques to investigate the creep characteristics under loads sustained for several days.

An additional two series of tests should be performed on samples tested as discussed above, but with the specimens saturated prior to testing.

Representative grain-size distribution curves and liquid and plastic limit values should be determined for the samples in each series.

D. Embankment Stress Analysis

It is requested that additional finite element stress analyses be made of the embankment fill. This work would constitute an expansion of a pilot analysis submitted to the Panel on August 3, and would incorporate the following specific requirements:

1. Three cross sections of the original right abutment embankment between Stations 12 and 15, and one axial section of the right abutment embankment (Stations 12+00 to 20+00) should be analyzed. The three transverse stations utilized, and the details of analytical formulation, are to be selected after review of the shape of detailed as-built cross sections.

2. The displayed results should include vertical stress, minor principal stress and strain.

3. The stresses should be those developed by layered construction, as opposed to the "gravity-turn-on" option.

4. In addition, stresses should be calculated to reflect the effect on the embankment of a reservoir rise to Elevation 5300.

5. Two complete sets of stresses should be computed for each section:

a. One adopting a core stiffness factor K of 470, as measured by the USBR on a composite, reconstituted triaxial sample under rapid shearing; and

b. One utilizing a K of 200, a value judged to be a probable lower limit for the Zone 1 fill.

The foregoing finite element analyses should be undertaken at once, under the guidance of Mr. Leps and Dr. Seed, with a target delivery date of perhaps October 15. Concurrently, a suite of triaxial shear tests on representative samples should go forward, as covered in the previous section, to provide appropriate verification of the K-parameter range assumed in requirement 5. above.

E. Modifications in Program

Field conditions may require modification of some of the details of the recommended program. Moreover, as the findings accumulate, the results may suggest changes, additions, or deletions. The field staff is encouraged to make changes that appear appropriate and to inform the Panel promptly. If major changes seem desirable, the staff should communicate with the Panel.

UNITED STATES DEPARTMENT OF THE INTERIOR STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Wallace L. Chadwick, Chairman
Arthur Casagrande
Howard A. Coombs
Munson W. Dowd
E. Montford Fucik
R. Keith Higginson
Thomas M. Leps
Ralph B. Peck
H. Bolton Seed
Robert B. Jansen, Executive Director

August 18, 1976

Mr. H. G. Arthur, Director
Design and Construction
U.S. Bureau of Reclamation
Building 67, Denver Federal Center
Denver, Colorado 80225

Mr. William H. McMurren
President & Chief Executive Officer
Morrison-Knudsen Co., Inc.
P. O. Box 7808
Boise, Idaho 83729

Gentlemen:

Reference is made to this Panel's charge from the Secretary of the Interior and the Governor of Idaho to review the cause of Teton Dam failure. It will be of important assistance to the Panel in this review if the construction techniques used, particularly on the right abutment, are as thoroughly understood as may be possible in the absence of personal observations. As an aid to such an understanding, the following questions have been prepared. Your full and candid answers to these questions will be a significant aid to the work of the Panel and will be much appreciated.

Please describe:

a. The manner in which axial grout distribution and closure were assured when the up and downstream grout travel was relatively unlimited. Details of any doubts over the effectiveness of this axial distribution in any particular location along the three grout curtains between Station 18+00 and Station 2+00 will be helpful. Likewise, details of difficulties in obtaining assurance of axial closure at any stations or grout holes along this same stretch of curtain will be helpful.

b. The manner in which the key trench between Station 18+00 and Station 2+00 was prepared to receive the first embankment material. Compare the way in which this trench was prepared with "broom clean". If there were differences in clean-up between particular stations, because of weather, or any other cause, please describe such differences in detail.

Letter to Messrs. H. G. Arthur and William H. McMurren

c. The manner in which any fissures or open joints in the key trench walls and floor were sealed between Station 18+00 and Station 2+00; that is, the manner in which, and the places where, slush grouting, dental concrete, gunite, or shotcrete may have been used, also the extent to which such sealing was general. Were any joints left unsealed and, if so, where? If known, please indicate the particular stations, if any.

d. The method of material selection, preparation, placement and compaction, in the key trench, of the "specially compacted earthfill" shown in the cross section marked "Foundation Key Trench" on USBR Drawing 549-D-9. If special difficulties were encountered in selection, preparation, placement or compaction at any points along the length from Station 18+00 to Station 2+00, please describe each.

e. The method of material selection, preparation, placement, and compaction in the key trench between Station 18+00 and Station 2+00 of the core material. If special difficulties were encountered in selection, preparation, placement or compaction at any points along the length from Station 18+00 to Station 2+00, please describe each.

f. The manner in which the contact area under the core of the dam outside of the key trench was prepared to receive the first core material. If special difficulties were encountered at any location along the length of dam between Station 18+00 and Station 2+00, please describe.

g. The manner in which core material was selected, prepared, placed, and compacted outside of the key trench, between Station 18+00 and Station 2+00. If special difficulties were encountered, please describe in detail by specific location.

h. Similarities and significant differences in the appearance of the walls and floor of the key trenches in the right and left abutments.

The information sought through this questionnaire is of special importance to the Panel in its review and early receipt of your answers will be much appreciated. However, it is realized that the task of preparation is a large one. For this reason, if it would be advantageous to you and permit earlier answer, the task may be broken into two phases, with priority given to Phase I covering the length of foundation from Station 16+00 to the spillway centerline, and Phase II covering from Station 18+00 to Station 16+00 and from the spillway centerline to Station 2+00. Partial replies are encouraged, that is transmittals for individual questions will be helpful.

Page 3

August 18, 1976

Letter to Messrs. H. G. Arthur and William H. McMurren

Please accept our appreciation in advance for your cooperation in supplying this important supplementing information.

Very truly yours,

for Robert D. Jensen
Wallace L. Chewick

UNITED STATES DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Wallace L. Chadwick, Chairman
Arthur Casagrande
Howard A. Coombs
Munson W. Dowd
E. Montford Fucik
R. Keith Higginson
Thomas M. Leps
Ralph B. Peck
H. Bolton Seed
Robert B. Jansen, Executive Director

August 20, 1976

MEMO TO R. B. Jansen:

I have completed review of the Inspector Daily Reports with special attention to foundation preparation and embankment placement at the right abutment during the 1975 season, which is the period of interest to us. I have attached highlighted copies of those Inspector Daily Reports which bear some information that is helpful to us. Steven Johnson's reports are most informative. R. Jones' are not bad; but Doug Janic's and Jerry Smith's only recite equipment employed and are of no value for our purpose.

The Special Inspection Report file covers special subjects, almost 90% of which are the Hobbs Riprap Quarry, a few reports on the powerhouse, and the record of the abutment dental concrete. We checked the record, as given in the Special Inspection Report, against the summary that we had previously shown on the drawing that we presented to the Panel at their last meeting. The record agrees with the summary.

In summary, from review of these reports, I find no glaring violations of the specifications, but I don't consider them a real reliable source of information of that type. The photographic record, of course, is much better and speaks for itself.


Clifford J. Cortright
Staff Engineer

Encls.

UNITED STATES DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Wallace L. Chadwick, Chairman
Arthur Casagrande
Howard A. Coombs
Munson W. Dowd
E. Montford Fucik
R. Keith Higginson
Thomas M. Leys
Ralph B. Peck
H. Bolton Seed
Robert B. Jansen, Executive Director

August 24, 1976

Mr. Robert R. Robison
Project Construction Engineer
Teton Project Office
P.O. Box 88
Newdale, Idaho 83436

Dear Mr. Robison:

On August 19, 1976, representatives of the Teton Dam Failure Review Group, the Teton Project Office, and the United States Department of the Interior - State of Idaho Independent Panel to Review Cause of Teton Dam Failure met in the Idaho Falls office of the Panel to discuss coordination of their activities.

With regard to the Schedule for Laboratory and Field Tests and Analyses appended to the Panel Report of August 5, 1976, several modifications and clarifications were made and mutually accepted.

The drilling and water testing of the grout curtain will be performed by crews and equipment from the Boise Regional Office of the USBR. The holes, water testing, and core will be logged by the Regional geologists and also independently by the Panel's on-site representatives.

The holes will be of NX size.

The depths of the final stages of both the vertical and inclined holes in the three spillway bays will be sufficient to penetrate the rock beyond the 80-foot depth of the foundation consolidation grouting beneath the spillway control structure.

Soils samples to be later identified for testing by Northern Testing Laboratories will be delivered with special handling by Bureau personnel to Billings, Montana. Samples to be tested by the Earth Sciences Branch

August 24, 1976

and samples to be stored will be delivered under special handling by Bureau personnel to Denver, Colorado. Space will be made available in advance in the humidity room there to receive and store the samples.

The Teton Project Office Laboratory will perform the following tests:

- (1) Natural Water Content
- (2) Grain Size Analyses
- (3) Liquid and Plastic Limit Tests

The Earth Sciences Branch will perform unconfined compression tests and drained triaxial compression tests. Detailed special instructions for these tests will be supplied by Panel representatives later.

Panel representatives will arrange for shipment of samples to be tested at the Corps of Engineers' Waterways Experiment Station, Vicksburg, Mississippi, and the University of California at Davis, California. Purchase Orders for testing at those laboratories other than those of the USBR have been arranged by Department of the Interior purchasing agents. Samples for testing will be selected and detailed instructions for testing will be issued to those laboratories by Panel representatives.

Sincerely,

Robert B. Jansen
Executive Director

cc:
Dennis Sachs
Sam D. Guy

UNITED STATES DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Wallace L. Chadwick, Chairman
Arthur Casagrande
Howard A. Coombs
Munson W. Dowd
E. Montford Fucik
R. Keith Higginson
Thomas M. Leps
Ralph B. Peck
H. Bolton Seed
Robert B. Jansen, Executive Director

September 8, 1976

TO: Robert B. Jansen

SUBJECT: Review of Report, "Right Abutment Excavation," Teton Basin
Project, by Teton Project Office, Newdale, Idaho,
November, 1975

I have reviewed subject report available at the Project Office.

The report was prepared in analyzing the contractor's claim for contract adjustment arising from alleged delays caused by the Government not giving timely direction for the removal of rock overhangs during the stripping of the right abutment.

I find nothing in the text which affords a clue to the cause of failure. The timing and manner of overhang removal performed within the Zone 1 foundation area in no way appears related to the failure.

Two photos, P549-147-2557NA, 10/10/72, and P549-147-2974NA, 8/8/73, do show a bench or profile irregularity in the keyway invert excavation estimated to be near Station 12+70 and Elevation 5220 which may have had some influence on the failure.

Clifford J. Cortright
Clifford J. Cortright

UNITED STATES DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Wallace L. Chadwick, Chairman
Arthur Casagrande
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Munson W. Dowd
E. Montford Fucik
R. Keith Higginson
Thomas M. Leps
Ralph B. Peck
H. Bolton Seed
Robert B. Jansen, Executive Director

October 6, 1976

Honorable Thomas S. Kleppe, Secretary
United States Department of the Interior
Interior Building
Washington, D.C. 20240

Honorable Cecil D. Andrus, Governor
State of Idaho
Capitol Building
Boise, Idaho 83720

Gentlemen:

The Independent Panel to Review Cause of Teton Dam Failure has continued its work under your charge. The Panel conducted technical working sessions in Idaho Falls on October 4 through 6, 1976, with all members attending. On October 4, inspections were made of investigative excavations at the damsite and of the auxiliary outlet works, which has been dewatered recently.

The following is a report on progress by the Panel since its report to you of August 5, 1976.

Organization

Through the cooperation of the National Park Service, two technicians have been assigned to the Panel for temporary assistance in preparation of illustrations for the Panel's report. Also, a draftsman will be provided by the Bureau of Indian Affairs on October 12.

Site Work

Satisfactory progress is being made on work requested by the Panel's letter of July 2, 1976 to Mr. H. G. Arthur, under USBR Contract No. DC-7232 with Gibbons and Reed Co. At the time of the Panel's inspection of the damsite on October 4, 1976, the general level of excavation, by five foot stages, of the embankment remnant on the right abutment was at

October 6, 1976

Letter to Honorable Thomas S. Kleppe and Honorable Cecil D. Andrus

Elevation 5210 and inspection trenches had been excavated at each stage to Elevation 5205. Trenches are being surveyed, logged, and photographed. Penetration tests are being made and soil samples are being taken for laboratory testing. A total of 92 nine-inch cube samples have been taken, distributed throughout the length and depth of the excavation. Arrangements have been made for specific tests at the Northern Testing Laboratory in Billings, the University of California at Davis, the Waterways Experiment Station of the Corps of Engineers in Vicksburg, and the USBR laboratories at Teton Dam and Denver. All of this work is pursuant to the schedule appended to the Panel's report of August 5, 1976. Shipments of soil specimens from the dam were made to the Billings and Denver laboratories during the week of September 13 and to the Davis and Vicksburg laboratories during the week of September 27.

The Panel received for its consideration during the technical work sessions of October 4-6 the results of two finite element analyses of transverse sections of the embankment conducted by Dynamic Analysis Corporation. These are being studied by the Panel. Copies of these analytical results have been supplied to the Interior panel for its use. Two other analyses are in progress.

As exploratory excavation on the right abutment has progressed to lower elevations in recent weeks, signs of distress have appeared in the dam embankment in the form of cracks and general distortion. This evidence is being carefully studied by the Panel in an attempt to ascertain whether it relates to the cause of failure, or is a post-failure condition resulting from collapse of the adjoining dam mass.

Quite satisfactory progress has been made in the channel excavation to lower the river. This work had advanced so that the Panel was able to enter the dewatered auxiliary outlet works tunnel on October 4. Inspection was made of the full length of the facility and it was found to be in sound condition, with no visible evidence of distress that could be related to the failure of the dam.

Drilling into the foundation of the spillway was begun by the USBR early in September. Nine holes have now been completed. Water pressure testing so far has indicated the grouted rock under this structure to be reasonably impermeable, within generally accepted standards.

Drilling is underway into the foundation in the vicinity of fissures near Dam Station 4+00, described in the USBR construction reports. This is in addition to drilling described in the schedule of August 5, 1976. One of the drill holes at that location, which has now progressed to a depth of about 300 feet, will extend into deep underlying sediments where samples can be taken for compression testing. The Panel also will have tests made on core samples taken from these sediments during the Bureau's preconstruction drilling.

October 6, 1976

Letter to Honorable Thomas S. Kleppe and Honorable Cecil D. Andrus

The Panel continues its review and analysis of data and the drafting of material intended for use in the final report.

In addition to participation in the technical working sessions of the Panel, individual members have periodically consulted with the staff in the Idaho Falls office and have made inspections of work at the damsite.

Arrangements have been made for construction of a model of the dam to facilitate visualization of various features that are regarded as pertinent in analysis of the failure.

The Panel appreciates the attention given by the office of the Secretary of the Interior to finalizing its definitive contract.

The continuing support which you and your staffs have extended to the Panel is deeply appreciated, as is the cooperative response of the Bureau of Reclamation to the Panel's requests.

The next technical working sessions of the Panel are scheduled for November 1-3, 1976, in Idaho Falls.

Respectfully submitted,

Wallace L. Chadwick, Chairman
Independent Panel to Review Cause
of Teton Dam Failure

CONTRACTORS
ENGINEERS
DEVELOPERS



MORRISON-KNUDSEN COMPANY, INC.

EXECUTIVE OFFICE
ONE MORRISON-KNUDSEN PLAZA / P.O. BOX 7808 / BOISE, IDAHO 83729 / U.S.A.
TELEX: 368439
PHONE: (208) 345-5000

E. M. ARMSTRONG
EXECUTIVE VICE PRESIDENT

October 7, 1976

Mr. Wallace L. Chadwick
United States Department of the Interior
State of Idaho
Independent Panel to Review Cause of
Teton Dam Failure
539 9th Street
Idaho Falls, Idaho 83401

RECEIVED

OCT 20 1976

W. L. CHADWICK

Re: Teton Dam

Dear Mr. Chadwick:

In its letter of August 18, 1976, the Panel has asked certain questions with regard to construction techniques used in the construction of Teton Dam with special attention to the right abutment. The Contractor, a joint venture composed of Morrison-Knudsen Company, Inc. and Peter Kiewit Sons' Co., hereby submits the following answers to those questions:

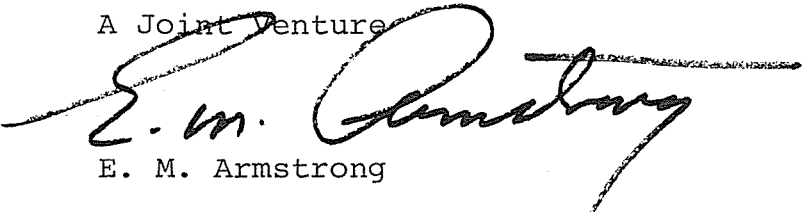
- a) The best information available to the Contractor with regard to this question is that contained in a letter submitted to the Contractor by its grouting subcontractor, McCabe Bros. Drilling, Inc., dated August 25, 1976 and appended hereto as an attachment.
- b) The key trench between 2 + 00 and 18 + 00 was prepared by using air and water. The cleanup was more extensive between Station 3 + 00 and 4 + 35 due to open joints and fissures.
- c) All fissures or open joints were backfilled with dental concrete or slush grout at the direction of the Bureau of Reclamation. To the knowledge of the Contractor, no joints were left unsealed.

Letter to Mr. Wallace L. Chadwick, dated October 7, 1976
Page two

- d) This material came out of the borrow area designated by the Bureau of Reclamation. In general, material of higher plasticity and optimum moisture was selected from the pit. Preparation of the material was by pre-irrigation. Placement was accomplished in 3" lifts and compacted by using air operated tampers and plate tampers and wheel rolling with heavy equipment. To the Contractor's knowledge, there were no difficulties encountered in any of these areas.
- e) Material selection was accomplished by the same method described in d), above. The pit was prepared in the following manner: A cut depth was determined by topographic notes to establish the desired drainage pattern. The pit was then divided into material blocks. Three-inch holes were then augered to the depth of cut required on a 200 foot grid and proctor optimum moistures were determined for drill cuttings and noted for the respective section of the pit. Moisture was added to the pit by sprinkling the required amount of water for the design cut on the area at least 3 weeks prior to excavation. Constant monitoring was possible by utilizing a Speedy Moisture Teller. Placement and compaction were in accordance with Bureau of Reclamation specifications.
- f) This area was blown clean with air and water. The rock was badly fractured and cleanup was a little more difficult on the right abutment than it was on the left abutment. There were areas on the right abutment which required the treatment described in c) above
- g) This material was handled as described in d) and e) above.
- h) The rock on the right abutment was more fractured than that on the left abutment and there were more fissures in the key trench in the right abutment than in the left abutment key trench.

Very truly yours,

MORRISON-KNUDSEN-KIEWIT
A Joint Venture


E. M. Armstrong

EMA:jl



DIAMOND CORE DRILLING — ROTARY DRILLING
PRESSURE GROUTING — FOUNDATION TESTING

BOX 1892, IDAHO FALLS, IDAHO 83401
PHONE 522-5437

August 25, 1976

Morrison-Knudsen Company, Inc.
P. O. Box 127
Newdale, Idaho 83436

RECEIVED

AUG 31 1976

Morrison-Knudsen - Kiewit
Contract No. 2594

Attention: Mr. Duane E. Buckert

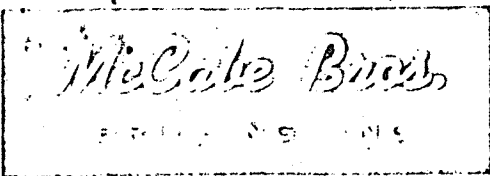
Re: Letter from 'Independent Panel to Review Cause of Teton Dam Failure' to Morrison Knudsen Co., Inc. for information on Construction Techniques.

Dear Sir:

Referring to Question: The Manner in which Axial grout Distribution and Closures were assured when the up and downstream grout travel was relatively unlimited.

There were three grout lines; a downstream, a center and a upstream. The downstream grout line was from Station 2 + 20 to 16 + 00, the upstream grout line was from Station 2 + 28 to 15 + 94 and the center grout line was from Station 2 + 23 to Station 18 + 00 and on. Most of the grout nipples were 2" diameter. The Area holes were Located over fairly large cracks and the nipples were set to intercept the cracks at different depths, some being set vertically over cracks with concrete poured around them. The downstream holes were vertical with 20 ft. centers. The upstream holes were at a 30° angle with 20 ft. centers, except one vertical at Station 5 + 28 and one fan hole at 2 + 28 37°. The center line holes were at a 30° angle with 10 ft. centers.

There were no closure holes on the downstream line. There are three fan holes at Station 2 + 20; one at 15°, one at 30°, and one at 45°. The upstream line has the following closures: 3 holes on 5' centers, 6 holes on 6' centers, 1 hole on 7' center, 2 holes on 9' centers, 1 hole on 10' center, 3 holes on 11' centers, 2 holes on 12' centers and 2 holes on 13' centers. As directed by the contracting officer, the holes from Station 9 + 22 to 10 + 00 in the upstream line were deleted.



Page 2

The primary holes were staggered from each other on the three grout lines. The Area holes that were set close to cracks were grouted first. We drilled the holes until we lost 50% or more of our drill water. Then commenced grouting at the bottom stage of the hole. Most of the area holes were intermittently grouted if the take was 500 cu. ft., with a waiting period of three hours. Eventually that stage of the hole would come up to the desired pressure required by the inspector. At that time we set the packer up to the next stage and progressed out of the hole through the different stages and finished grouting by hooking the nipple and grouting to the specified pressure. If the above hole was required to go deeper, we then drilled to the specified depth or until we had a water loss of 50% or more and then set the packer at the directed settings and grouted the different stages at required pressures until we staged up to the previous stage grouted, thereby completing the entire hole. Closures were added to area holes.

There are primary holes every 80 ft., secondary holes every 80 ft. and closure holes every 40ft. on the downstream and upstream grout lines. The primary holes were drilled and grouted first, the secondary holes were drilled and grouted second and the closure holes were drilled and grouted last as directed. All grouting of holes was accomplished in the same manner described above for the Area holes.

The centerline holes have a primary every 80 ft., a secondary every 80 ft. and an intermediate hole every 40 ft. with closure holes every 20 ft. The primary holes were drilled and grouted first, the secondary holes were drilled and grouted second, the intermediate holes were drilled and grouted third and the closure holes were drilled and grouted last as directed. Of course the centerline has a more complex pattern than the downstream and upstream grout lines and is designed to serve as a closure line for the downstream and upstream grout lines, with many closure holes being added.

Good packer settings were accomplished with the very minimum of difficulty.

A large percent of the holes where the water loss was negligible, we were able to drill to the complete depth of the hole, in this case we grouted from the bottom stage up, until the hole was completely grouted.

about 98 or 99% of the stages in all holes were water tested, with the exception of the top 20 ft. in many holes. The migration of water from the water tests and the grout travel into other drilled holes was very minimal. All holes were completely backfilled with grout after all stages were completed. All grout leaks to surface areas were calked immediately and continuously until leakage stopped.

Referring to Question: Details of any doubts over the effectiveness of this Axial distribution in any particular location along the three grout curtains between Station 18 +00 and Station 2 + 00.



Page 3

The only doubt that we have been concerned with was the high percent of Calcium Chloride being used. The highest percent used as directed by the inspectors was 10% for a short time, later this was lowered to 8%.

As an example, if the hole was 100 ft. deep and we were grouting with about 3% or more Calcium Chloride, intermittently grouting the bottom stage and finally the bottom stage came up to pressure, then we stage grouted the hole up to the surface. At this time we were directed to deepen the hole to 130 ft. We then drilled the hole down to 90 ft. and had a total water loss. Then we set the packer above the water loss in the bottom stage of the hole and started grouting until the stage came up to pressure; sometimes this stage required intermittent grouting. This condition has happened many times when using Calcium Chloride. The question we have asked ourselves about the above problem is: Is this same condition happening in the grouting of large or small cracks and fissures, causing a honey-comb effect? Thereby causing many more closure holes to be drilled and grouted on the centerline than otherwise would be necessary.

Referring to Question: Details of difficulties in obtaining assurance of Axial closure at any station or grout hole along the same stretch of curtain.

We had no difficulty in grouting up to the desired pressure in all stages of all closure holes. If one closure hole took more than the minimum amount of grout in any one stage, we were then ordered to drill and grout additional closure holes.

Referring to Question: Similarities and significant differences in appearance of the walls and floor of the Key trench in the right and left abutments.

The left abutment had a few caverns in the walls, the floor of the key trench appeared to be good solid rock. Directly up the grout line on the right abutment, 75 to 125 ft. above the tower on the steepest slope, we had some grout leaks around some large boulders. As we were calking these leaks, numerous bats were flying out of the cracks that we were attempting to calk.

The right abutment from about Station 18 + 00 to Station 11 + 50 appeared to be very badly fractured with small cracks in the walls and the floor of the trench. From Station 11 + 50 to 10 + 00 it appeared to be good solid rock. From Station 10 + 00 to 7 + 50 it appeared to be good solid rock on the walls and floor of the key trench, with some small visible fissures. From 7 + 50 to 2 + 00 the walls and floor appeared to be of good sound rock with a very little small fracturing with the exception of numerous large faults visible in the walls and floor of the key trench.

McCabe Bros.

DRILLING INC.

DIAMOND CORE DRILLING — ROTARY DRILLING
PRESSURE GROUTING — FOUNDATION TESTING

BOX 1292. IDAHO FALLS. IDAHO 83401

PHONE 522-5437

Page 4

The lake bed sediments that underlay the riolite formation along the extreme length and width of the dam at depth were drilled and grouted to the desired pressures for a short distance on the outer end of the left abutment.

All work described above was completed as directed by the contracting officer, The Bureau of Reclamation.

Very truly yours,

McCABE BROS. DRILLING, INC.

Edwin L. McCabe

Edwin L. McCabe
President

ELM:RJN



United States Department of the Interior

BUREAU OF RECLAMATION
OFFICE OF DESIGN AND CONSTRUCTION

ENGINEERING AND RESEARCH CENTER
P.O. BOX 25007
BUILDING 67, DENVER FEDERAL CENTER
DENVER, COLORADO 80225

IN REPLY
REFER TO: 210
510.

OCT 22 1976

OCT 19 1976

Mr. Wallace L. Chadwick
Chairman, Independent Panel to
Review Cause of Teton Dam Failure
Post Office Box 1643
Idaho Falls, ID 83401

Dear Mr. Chadwick:

Draft answers to the questions regarding the Teton Dam failure posed to Mr. McMurren of Morrison-Knudsen Company and to me in your letter of August 18, 1976 were handed to you on October 4. We are enclosing our final responses to these questions which we ask that you use instead.

The material you received earlier was subsequently reviewed in this office by our grouting expert, Mr. Lloyd R. Gebhart, and others. Certain portions were discussed with project personnel, several minor changes were made in the text, and the photographic references were corrected.

We understand that the Morrison-Knudsen Company is giving you their answers to these questions in a separate transmittal. The answers we have given are, therefore, attributable only to Bureau of Reclamation project and Denver Office records and observations. We believe they accurately describe the situation at the damsite as it existed during construction and the construction techniques used.

Sincerely yours,

H. G. Arthur
Director
Design and Construction

Enclosures

Copy to: Morrison-Knudsen Company, Inc.
Post Office Box 7808
Boise, Idaho 83729
Attention: Mr. W. K. Smith
(with copy of enclosures)



Please describe:

A. The manner in which axial grout distribution and closure were assured when the up and downstream grout travel was relatively unlimited. Details of any doubts over the effectiveness of this axial distribution in any particular location along the three grout curtains between Station 18+00 and Station 2+00 will be helpful. Likewise, details of difficulties in obtaining assurance of axial closure at any stations or grout holes along this same stretch of curtain will be helpful.

GROUTING REQUIREMENTS

(A) Grouting requirements between Station 2+00 and Station 18+00 consisted of a triple curtain between Station 2+00 and 16+00 and a single curtain between 16+00 and 18+00. Blanket holes were located in areas where joints and fissures were exposed in the curtain area and also a blanket grouting program was performed under the spillway weir section. The minimum depth requirements for the curtain holes were 260-60-160-60-260 feet for 80-foot patterns on the centerline curtain on 10-foot centers. In the spillway area, the maximum depths were increased to 310 feet.

Specifications drawings required that both the upstream curtain and downstream curtain be drilled on 20-foot centers with no provisions for spaced closure. However, these curtains were split spaced and closed to depth. Specifications drawings also required that both the upstream and downstream curtain consist of vertical holes. After excavation of the key trenches was completed, it was determined that angle holes on one of the two outer curtains would readily intercept more joints and, therefore, the upstream curtain holes were drilled on angles 30 degrees from vertical. Specifications required AX (1-7/8-inch) diameter size holes be drilled and that holes be down staged if water losses larger than 50 percent occurred. When partial water losses occurred, the percentage amount was determined by the onsite inspector and grouting of these partial water loss stages consisting of 50 percent or larger was strictly adhered to.

In the vicinity of the spillway section, an exception was made in regard to the centerline curtain insofar that between Station 10+00 and Station 11+37, the centerline curtain was eliminated and incorporated with the upstream curtain. This was done because the alignment of the centerline curtain was in the same alignment as the AOW gate chamber shaft and because a positive curtain was better protection for the shaft when located upstream of the shaft. This was accomplished for two reasons. First, double coverage could be given to the adit and shaft by enveloping the curtain between the shaft and the reservoir, and secondly, curtain holes could be extended to their full design depth rather than having to be shortened to prevent intersection with the shaft and adit concrete. Curtain holes enveloping the AOW tunnel had to be shortened to prevent intersecting the tunnel concrete. However, radial holes from within the tunnel in the curtain area were deepened to overlap the curtain holes by 30 feet.

GROUTING ORGANIZATION

U.S. Bureau of Reclamation

The grouting organization for the Bureau of Reclamation consisted of one Supervisory Civil Engineer and primarily, three Construction Inspectors. The Supervisor had approximately 12 years of inspection and supervisory experience in the field of grouting. Three primary Construction Inspectors had grouting experience varying from 2 to 5 years prior to arriving on Teton Dam.

Each of the three primary inspectors was responsible for one shift on a three-shift basis and supervised additional inspectors when grouting operations were separated and additional inspectors were required. When grouting operations were separated, the primary inspector was able to contact subordinate inspectors through the contractor's communications system to discuss any problems.

Contractor (McCabe Brothers Drilling Company)

The contractor usually had a work force which varied from 18 to 27 men. The Company is owned by three brothers and each brother was a shift foreman. A mechanic was on duty on day shift to make necessary repairs. Other workmen consisted of pump operators and drillers.

CONTROL OF GROUTING OPERATIONS

Order of Grouting

When the contractor determined the area that he wanted to grout in, grout holes on the upstream and downstream curtain were located by Bureau inspectors. These locations were previously determined from profile drawings from which the proper spacings were determined. Blanket holes were located in the field to fit the rock foundation conditions except those required beneath the spillway weir which were located on a pattern basis. Location of grout holes for the centerline curtain was also determined from a profile drawing prepared prior to concrete placement in the grout cap. Pipe nipples were embedded in the concrete as the concrete was placed. Angles for the pipe were accurately determined with a machinist's protractor and the pipe nipples were set above the concrete-rock contact at all times so that this contact would be drilled and grouted if a bond did not occur.

When grouting was initiated within a specific area, the blanket holes were drilled and grouted prior to any grouting performance or curtain holes. The contractor usually worked in an area 400 to 500 feet long. Therefore, initial curtain grouting consisted of drilling on five to six pattern holes. As drilling and grouting progressed on the original pattern holes to depth, it was sometimes necessary to initiate drilling and grouting on

intermediate and final closure holes simultaneously to facilitate the contractor's operations. However, a lag of 40 feet in vertical distance was always adhered to with respect to adjacent related holes.

The upstream curtain was grouted in similar fashion to the downstream curtain; however, no holes were drilled on the upstream curtain until those patterns on the downstream curtain in vicinity of the holes on the upstream curtain to be drilled were completed.

Grouting on the centerline curtain was initiated after all other grouting in the vicinity was completed. As previously mentioned, the centerline curtain was grouted on 10-foot centers with the 10-foot center holes split to 5-foot centers or less if a grout take of 20 cubic feet or more per stage occurred. This criterion was adhered to, with two exceptions. At Station 10+25 stage 0-20 feet, a grout take of 28 cubic feet was not split as most of the grout injected leaked to the surface within a few feet of the holes. Also, a grout take of 1,003 cubic feet at Station 11+37 stage 220 to 245 was only split on one side. However, this take is near the gate chamber adit, and the area was super-saturated with grout holes from within the adit and access shaft.

Five-foot-closure holes were drilled and grouted at Stations 11+09, 8+19, 6+34, 6+46, 6+22, and 15+28 to check areas of doubt. However, these holes were not required as the adjacent 10-foot-closure holes previously grouted were tight.

DAILY DIRECTION BY THE BUREAU SUPERVISION

As grouting was initiated in each area, a drilling and grouting instruction sheet was made by the Bureau supervisor. On this sheet were listed holes that were available for drilling and grouting by the contractor as determined by the Bureau supervisor. This sheet was made on a daily basis and was updated taking into consideration the work that had been previously completed, and the work that was expected to be completed during that particular day. Special instructions and safety notes were also added to these sheets from time to time. On rare occasions, it would be necessary for the field inspector to make additions to the sheet if field operations made it necessary. This daily sheet was made for the purpose of keeping unity by having a single organized program within the Bureau inspection forces and it was also available to the contractor so he could plan his operations accordingly. Examples of these sheets are attached at the suggestion of Cliff Cortright, Panel Representative.

LOG BOOK KEPT BY INSPECTORS

From the plan and profile drawings kept in the Bureau Office, log books were made which contained a profile of grout holes as located in the field. In these log books, the onsite grouting inspector kept a running record of all drilling and grouting that was performed. The log books were passed from inspector to inspector (shift to shift). This record contained the history of each hole and was available to the inspector at all times at the pump site for the purposes of back-checking for related grout takes, water losses, water test information, surface leaks, and performance dates of adjacent holes. Copies of pages from several log books are attached to show the types of information contained.

DAILY WRITTEN REPORTS

In addition to the log books, each field inspector was required to write a daily report which gave a brief description of the holes drilled and grouted as to location, depth, water tests, grout takes, equipment problems, conversations with the contractor's representative, and instructions to the contractor. Also, a drill sheet was made for each hole drilled and a grout sheet for each hole grouted. The drill sheet was passed on from shift to shift until that particular hole was completed or the hole was stopped for grouting at which time it was turned in to the Bureau supervisor at the end of the graveyard shift. The drill sheet contained drilling information such as rock hardness, color of water return, time of drilling, and water losses. The grout sheet was also passed on from shift to shift until that particular hole was completed or the hole was ready to be redrilled to a deeper depth at which time it was turned in to the Bureau supervisor at the end of the graveyard shift. The grouting sheet gives a complete history of a grout hole. This history may be very complex; however, all information relating to the hole is recorded in minute detail in relation to time. The grout sheet primarily contains information relating to water tests, packer settings, initial grout mixes, final grout mixes, pressures, surface leaks, amount of grout take per hour, total grout take, holding pressures, back pressures, suction, etc. A copy of a drill and a grout report (see attached examples) are appended to the daily written report.

After the daily reports in conjunction with the drill and grout reports were turned over to the Bureau supervisor by the field inspector at 8:30 a.m. each morning, they were reviewed and checked for accuracy. The results from the drill and grout reports were then immediately plotted on a plan and profile drawing. These results were plotted each day on the same drawing and thoroughly studied by the supervisor to correlate grout takes from hole to hole and curtain to curtain.

Profile drawings from each curtain were usually overlain for a positive check so that no gaps in the overall curtain area would occur. From each days' information as it was plotted, depth of holes could be changed and additional holes added as required. The daily drilling and grouting

instruction sheet discussed above was determined from the plan and profile drawing.

RECORDS BY THE CONTRACTOR

The records made by McCabe Brothers Drilling Company are extensive and were kept diligently by the employees of the contractor. Drill logs were made by each driller of each hole drilled on each shift. Grout pump operators kept a running record of all grout injected which contained the time, number of batches, cubic feet per batch, and the grout mix. A record of all water tests was also made which stated the hole number stage and amount of take.

Drillers recorded drill bit serial numbers used each shift with corresponding drilling depths.

A profile of each curtain in the vicinity of the work area was kept current daily and given to each foreman. This profile was reviewed with the inspector and correlated with the Bureau Daily Drilling and Grouting Sheet. A time log on each grout pump was kept by pump operators.

GROUT MIXES, CALCIUM CHLORIDE, SAND, PRESSURES, WATER TESTS

Grout mixes were designed to fulfill the scope of the specifications and design criteria. It was desirable that grout travel be limited to within 100 feet of the curtain area and that the upstream and downstream curtains be constructed as barrier curtains for the centerline curtain which was the final closure curtain. When large grout takes on the upstream and downstream curtain were encountered, grout mixes were readily thickened. Calcium chloride was used to increase hydration and decrease the initial set time and was rarely used when a hole was being pumped under pressure.

When a hole was relatively wide open and the grout mix used was an 0.8:1 W-C ratio (by volume), the hole would accept grout at the rate of 250 cubic feet of cement per hour (maximum pump rate) and the pressure on the hole gage would be zero and the hole would have extreme suction. This indicated that the grout was traveling away from the hole area and, in order to restrict travel, the hole was pumped intermittently (500 cubic feet with delays ranging from 3-8 hours) by using calcium chloride. When pressures began to register on the hole gage during a pumping sequence, the calcium chloride was discontinued and pumping would then usually continue to refusal. Precautions were taken to prevent slugging a hole prematurely.

Sand was used when evidence showed that a large void had been encountered and that the sanded mix would be readily accepted. For instance, the blanket holes in the spillway area accepted large amounts of grout; however,

sand was used in only one hole as no large voids were encountered during drilling of these holes. Although other holes accepted grout quite readily, no voids of consequence were detected by the drillers when water losses occurred and, therefore, a sanded grout mix was not used in this area.

Calcium chloride was added to accelerate hydration of the grout mix and control travel within the curtain area. Laboratory and field experiments were performed to determine the optimum amount of calcium chloride to be used to achieve setting after the grout reached the area to be grouted. Numerous variables, such as mix water temperature, sand temperature, cement temperature, air temperature, rate of take of the hole, and distance of hole from the mix plant, affected the set-up time and the injection time. A hole that was wide open would usually accept grout at the rate of 250 cubic feet of sand and cement or cement per hour. The lapsed time between mixing and injecting the grout at this rate varied between 6 and 8 minutes. An initial set-time of 12 to 16 minutes was therefore desirable, so that the grout could adequately reach its destination before prematurely setting.

Due to these temperature variances of the grout ingredients, it was impossible to develop a usable criteria to accurately predetermine amount of calcium chloride required to attain the desired set time. A more feasible set of criteria was used based on grout temperatures at the grout pump. From 2 to 3 percent by weight of cement of calcium chloride was added when the mix water temperature ranged between 75 and 80 degrees F. and up to $6\frac{1}{2}$ percent of calcium chloride was required when the mix water temperature was in the 35 to 40 degrees F. range. Eight percent calcium chloride was used for a short interval when near freezing water was used by the contractor, however, set times were uncontrollable and the percentage was ultimately lowered. Grout would reach the critical temperature of 90 degrees F. when using the warmer mix water and near 70 degrees F. when using the colder mix water. Grout temperatures were monitored constantly at the pump by the pump operator and the inspector so that the proper amount of calcium chloride required could be constantly adjusted. Water was added to the grout mix at the pump on rare occasions when the grout began its initial set in the tub before it could be injected. It was of utmost importance that, when calcium chloride was being used in a grout mix, the temperature of the grout mix be kept as high as possible without prematurely setting in the tub before it could be injected. Adding lesser amounts of calcium chloride only prolonged the set time and increased grout travel distances which was undesirable in holes which were determined to be wide open. The use of calcium chloride on the centerline curtain holes was very limited.

Pressures used during grouting and water testing consisted of 10 psi at the hole collar and were increased by 0.75 psi per foot of depth of the packer setting normal to the rock surface at the hole collar. Pumping pressures were kept at the design pressure at all times unless surface leaks occurred or when the hole acceptance rate was greater than the capacity of the pump.

For grouting of the downstream and upstream curtains and blanket holes, a maximum of 5:1 water-cement ratio by volume was used. An 8:1 maximum ratio was used on the centerline (final closure) curtain. All packer settings were water tested prior to grouting and the starting grout mix was determined by the amount of water accepted in the 5-minute water test period. On the upstream and downstream curtain and blanket holes, the following criteria were used:

<u>Water accepted in 5 minutes</u>	<u>Starting Grout Mixture</u>
30 c.f. or more	3:1 w/c ratio
20 - 30 c.f.	4:1 w/c ratio
20 c.f. or less	5:1 w/c ratio

For the centerline curtain, these criteria were modified to:

<u>Water accepted in 5 minutes</u>	<u>Starting Grout Mixture</u>
30 c.f. or more	5:1 w/c ratio
20 - 30 c.f.	6:1 w/c ratio
20 c.f. or less	8:1 w/c ratio

Grout mixes were changed when it was felt that a thicker mix would be readily accepted by the hole. When to change mixes was a judgment decision made by the onsite inspector and was based on rate of take, drilling characteristics, pumping pressure, and intuition or so called "feel of the hole" by the inspector. Only basic criteria were specified as mix changes could be based on hole behavior and this was quite variable even within different stages within the same hole.

When large grout takes were encountered in any portion of any hole at lower than normal pressures, the grout mix was progressively thickened. Sand or calcium chloride was used only after it was definitely determined that a hole would accept thick mixes. Once it was determined that a hole was wide open, intermittent grouting was performed by injecting 500 cubic feet of cement or cement and sand and then washing the hole with just enough water to clear the hole. Grouting was resumed after a 4-hour interval. Two percent of bentonite by weight of the cement in a batch was added to all mixes containing sand to facilitate keeping the sand in suspension during pumping.

EQUIPMENT

Grout pumps used by the contractor consisted of Gardner-Denver 6"x3"x6" and 5"x2½"x5" air operated duplex piston type in conjunction with a 25-cubic-foot agitator tube and circulating system grout lines. Pumps were usually located within 50 feet of the hole being pumped which facilitated the pumping of thick mixes. Pumps were identified by

number and operating logs were kept. Pumps were cleaned after each pumping interval and were dismantled every 110 hours at which time piston swabs were replaced and liners checked. This maintenance schedule was strictly adhered to by the contractor and throughout the duration of the grouting program a pump breakdown occurred only once while a hole was being pumped.

Pressure gages consisted of Ashcroft 0-100 lbs. for low pressure and 0-300 lbs. for higher pressure. The gages were internally filled with glycerin for dampening purposes from pump surges which made them extremely long lasting. The gages were activated by an oil filled diaphragm in contact with the grout mixture.

Communications between persons at the grout pumps at the grout hole and the mixing plant were achieved by the use of waterproof mine telephones. These telephones were also equipped with signal lights for use in ordering batches. Three separate light systems, one for each mixer, were incorporated to facilitate operations between a grout pump and its designated mixer at the plant. This system was used to call personnel to the telephone who may have been at some distance from the telephone. Telephones were located at the office, repair shop, mixing plant, and at each grout pump located at the grout hole. The main telephone line had numerous outlets and the telephones were equipped with extensions so they could be readily moved.

REPORTS

Monthly Reports (L-10's) were submitted for construction and design review. These reports contained plan and profile drawings of all the work performed during the month and a summary of holes grouted. The hole summary sheet contained all information pertinent to each hole such as stages, pressures, mixes, water tests, surface leaks, and holding pressures. A summary sheet is attached. A general narrative was also included which stated the amount of drilling accomplished, the total amount of cement and sand injected, and also the number of water tests performed.

SUMMARY

The upstream-downstream curtains were not intended to be closed beyond 20-foot centers. The purpose of these two curtains was to act as barriers for the centerline curtain which was the intended main-line of final closure. Final closure of the centerline curtain was rather easily attained. The number of 5-foot closure holes was negligible and 2½-foot closures were required only twice (Stations 2+60 and 3+10). To eliminate doubts during the time of grouting, holes were extended or extra holes added. Full confidence in the effectiveness of the grout curtain as a barrier was obtained by the meticulous drilling and grouting operations and method of closures.

In regard to the attached letter submitted by McCabe Brothers to Morrison-Knudsen Company, dated August 25, 1976, we generally agree with all statements except for Paragraphs No. 1 and No. 2 on page 3. High percentages of calcium chloride were seldom used and the situation of water losses occurring higher in the hole after grouting than when the water loss had originally occurred did not exist. Water losses did however occur at times at the same location or immediately below the original water loss, which is a normal occurrence.

Spillway + D/S Curtain

3-20-75

Drilling

①	12+00 - 10' D/S	0-260
②	11+20 5' D/S	0-310' after 1119 5' U/S is comp
③	9+40 10' D/S	0-260'
④	10+44 12' U/S	0-80 after 10+44 2 1/2' D/S is comp also 10159 5' U/S is comp to 80'
⑤	10+74 19 1/2' D/S	33-80 after grouting WL
⑥	10+66 19 1/2' D/S	0-80 after 10+74 19 1/2' D/S is complete to 80 feet.
	10+59 5' U/S	38-80 after grouting WL.

Grouting

Int	10+31 15' U/S	0-10 WL
Int	10+44 12 1/2' D/S	0-80
Int	10+89 5' U/S	0-80
Dist H+	10+74 19 1/2' D/S	33 WL
D	11+19 5' U/S	0-80

of ~~11+19~~ 10+74 19 1/2' D/S proceeds to
take at the same rate as before - thicken up
quicker and use coll on 1:1 & 8:1 mixes to
bring Temp to 90° or thereabouts.

Spillway - D/S Centran

3-22-15

① Drilling

Cont	11+20	5' V/S	0-310
	12+00	10' D/S	0-260
	9+40	10' D/S	0-260
	10+44	12' V/S	0-80 after 10+59 5' V/S is complete to 80'
	10+74	19 1/2' D/S	33-80 after grouting WL
	10+59	5' V/S	38-80 after grouting WL
	10+66	19 1/2' D/S	0-80 after 10+74 19 1/2' D/S is complete to 80 feet

Grouting

Cont	10+31	15' V/S	0-10
Cont	10+59	5' V/S	38' WL
Cont	10+74	19 1/2' D/S	33' WL
	11+20	5' D/S	0-310 after drilling

Note: Due to the close proximity of the tunnel use 100 psi map on D/S Centran holes between station 11+20 and 9+60

Also let's correlate our labor reports on shift. We're listing some people twice.

1410 X 41
13796

14-106 Sample of Log Book Sheet
HOLE COMPLETE
9/13/74

DAY
7-13-74

1 SK
9/11/74

20'
H₂O 3.5 CF
DAY 9-12-74
D.M.

40'
H₂O 1.7 CF
9-12-74 DAY
D.M.

60'
H₂O 1.6 CF
DAY 9-12-74
D.M.

80'
H₂O 3.0 CF
DAY 9-12-74
D.M.

100'
H₂O 3 CF
DAY
9-12-74
D.M.

130'
H₂O 0 CF
DAY
9-12-74
D.M.

160'

DRILLING COMPLETE
9/11/74 SWING

HOLE COMPLETE
DAY 9-13-74 D.M.



H₂O 0.6 CF
9/8/74

20
H₂O 0.3 CF
9/15/74

40
H₂O 0.8 CF
9/13/74

60'

DRILLING COMP
DAY 7-13-74 D.M.

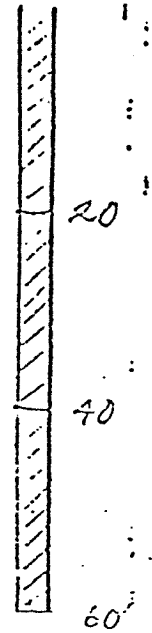
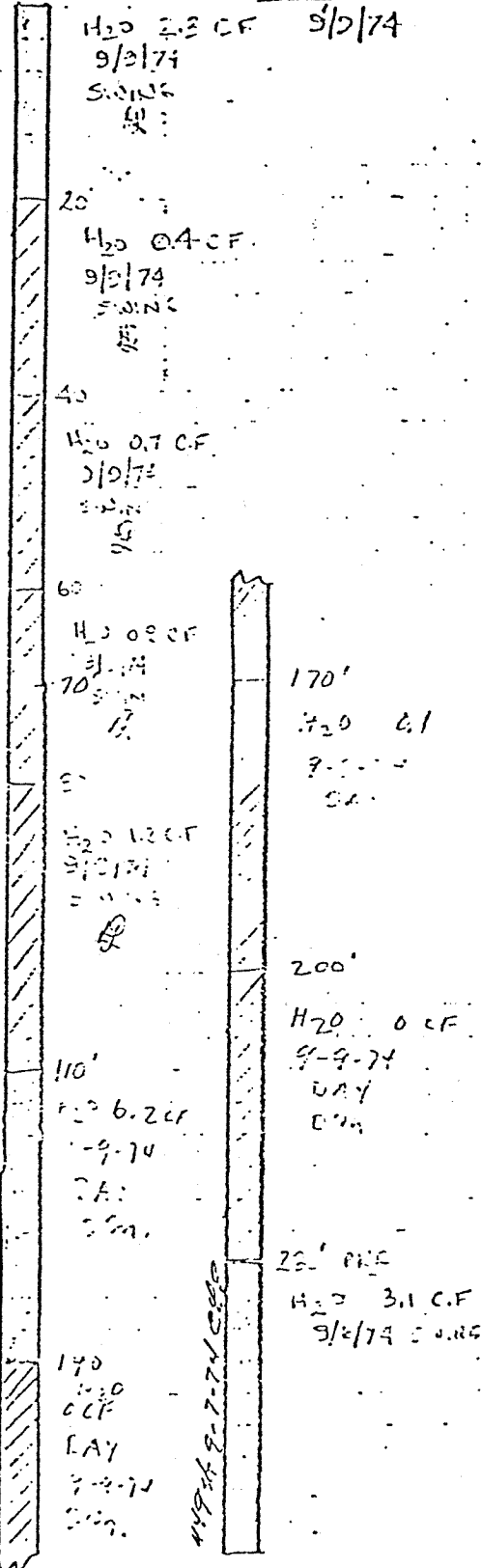
Page 38 from
Inspector's Log Book
Holes 13796 and 14106
on Centerville Contain

P
2 1/2" x 4 1/2"
14+15

2 1/2" x 4 1/2"
14+25

39

HOLE
COMPLETE



Drilling complete

9/12/74

Hole Complete

9/12/74

BACKFILLED 9/13/74

Page 39 from
Inspector Log book
Holes 14+15 and 14+25
on Centerville Extension

DRILLING COMPLETE
DAY 9-6-74

BACKFILLED 9/13/74

GROUTING INSPECTOR'S REPORT

Sample Grout Sheet
SHEET 1 OF 1

PROJECT Teton Dam SHIFT 10:00
 FEATURE Teton Dam (DAM, TUNNELS, SPILLWAY, ETC.) INSPECTOR A. Stocker
 SPECS NO. DC-6910 LINE (A, B, C ETC.) RELIEVED BY Charles K. Schwab

HOLE NO. <u>6+70 10' 1/5</u>	
DEPTH DRILLED	<u>60</u>
PROPOSED FINAL DEPTH	<u>60</u>
PREVIOUSLY GROUTED	<u>0</u>

CEMENT SUMMARY (C.F.)			
	PLACED	WASTED	
		GOVT.	CONTR.
WITH PACKER	<u>134</u>		<u>2 1/2</u>
WITHOUT PACKER			<u>12-4 1/2</u>
TOTAL			

(GURUP O.K. 11:00 P.M.)

STAGE DEPTH.	TIME	CEMENT			W/C	PUMPING PRESSURE	REMARKS
		TOTAL	CHANGE	SACKS PER HR.			
40-60	11:00 P	Water test			4:50	26	start test
	11:05				11	11	0.2 c.f. in 5 min 25# HP 15# BP
20-40	11:13	Water test			4:50	13	start test
	11:18				11	11	0.1 c.f. in 5 min. 13# HP 10# BP
0-20	11:27	Water test			4:50	10	start test
	11:32				11	11	5.8 c.f. in 5 min.
0-20	12:20	start grout			5:1	10	started grouting
	12:30	4	4	24	5:1	10	
	1:30	14	10	10	5:1	10	
	2:30	24	10	10	5:1	10	
	3:00	30	6	12	5:1	10	
	4:00	38	8	8	11	11	
	5:00	48	10	10	5:1	10	
	6:00	56	8	8	11	11	
	7:00	64	8	8	11	11	
	8:00	70	6	6	5:1	10	E.O.S. Grout O.K.
0-20	8:24	12-5	74	240			
	9:00	97	15	8=	5:1	10	
	10:00	98	8	8	11	11	Told Capt to change to 4:1
	11:00	99	3	8=	11	11	change to 4:1
	11:50 AM	104	4	8	4:1	11	
	12:00	100	6	6	11	11	
	1:50 PM	106	6	6	11	11	
	2:00 PM	112	6	6	4:1	10	
	2:00 PM	116	4	4	11	11	
	4:00 PM	118	2	2	4:1	11	END OF SHIFT.
	5:00	120	2	2+	11	11	1/2 c.f. in 5 min
	6:00	122	2	2+	11	11	
	7:00	124	2	2+	11	11	
	8:00	126	2	2+	11	11	1/2 c.f. in 5 min.
	9:00	128	2	2+	11	11	
	10:00	130	2	2+	11	11	
	11:00	132	2	2+	11	11	1.25 c.f. in 5 min
	12:00	134	2	2	11	11	1/2 c.f. in 5 min

This report should show a complete record of the treatment given each hole listed on HP.
 Remarks column: Record leaks, difficulties, back pressure, recommendations, etc.
 Reasons for 'Waste' should be explained in detail. W/C to be measured by volume.

GROUTING INSPECTOR'S REPORT

SHEET OF

PROJECT Fe-ton DAM Dec. 5 SHIFT Swing 19 7/2
 FEATURE spillway Blanket grout INSPECTOR A. Harris
 (DAM, TUNNELS, SPILLWAY, ETC.)
 SPECS NO. DC 6910 LINE 12' D/S RELIEVED BY R. P. Michel
 (A, B, C ETC.)

HOLE NO. <u>11-04 12' D/S</u>	
DEPTH DRILLED	<u>80</u>
PROPOSED FINAL DEPTH	<u>80</u>
PREVIOUSLY GROUTED	<u>0</u>

CEMENT SUMMARY (C.F.)			
	PLACED	WASTED	
		GOVT.	CONTR.
WITH PACKER			
WITHOUT PACKER			
TOTAL			

STAGE DEPTH	TIME	CEMENT			W/C	PUMPING PRESSURE	REMARKS
		TOTAL	CHANGE	SACKS PER HR.			
<u>60-80</u>	<u>7:30</u> <u>7:35</u>					<u>45</u>	<u>0.6 CF in 5 min</u>
<u>40-60</u>	<u>7:40</u> <u>7:45</u>					<u>30</u>	<u>5.5 CF in 5 min</u>
<u>40-60</u>	<u>1:45</u> <u>2:15</u> <u>3:00</u> <u>3:20</u>				<u>5-1</u>	<u>30</u>	<u>start grouting</u>
		<u>2</u>	<u>2</u>	<u>4</u>	<u>5-1</u>	<u>30</u>	
		<u>4</u>	<u>2</u>	<u>2+</u>	<u>5-1</u>	<u>30</u>	
		<u>4</u>	<u>0</u>	<u>—</u>	<u>5-1</u>	<u>30</u>	<u>refused grout</u>
<u>20-40</u>	<u>3:52</u> <u>3:57</u>				<u>4-3</u>	<u>15</u>	<u>0.8 C.F. in 5 min.</u>
<u>0-20</u>	<u>4:06</u> <u>4:11</u>				<u>4-3</u>	<u>10</u>	<u>10.6 C.F. in 5 min.</u>
<u>0-20</u>	<u>4:20</u> <u>5:00</u> <u>6:00</u> <u>7:00</u> <u>8:00</u>				<u>5-1</u>	<u>10</u>	<u>start grouting</u>
		<u>4</u>	<u>4</u>	<u>6</u>	<u>5-1</u>	<u>10</u>	
		<u>10</u>	<u>6</u>	<u>6</u>	<u>5-1</u>	<u>10</u>	
		<u>16</u>	<u>6</u>	<u>6</u>	<u>5-1</u>	<u>10</u>	
		<u>22</u>	<u>6</u>	<u>6</u>	<u>5-1</u>	<u>10</u>	<u>E.O.S.</u>
<u>0-30'</u>	<u>8:00</u>				<u>5-1</u>	<u>10</u>	<u>CONTINUED GROUTING</u>
	<u>9:00</u>	<u>24</u>	<u>2</u>	<u>2</u>	<u>5-1</u>	<u>10</u>	<u>WELL ABOVE 5' GROUT</u> <u>FEELING UP</u>
							<u>HOLE CLOSED</u>

This report should show a complete record of the treatment given each hole listed.
 Remarks column: Record leaks, difficulties, back pressure, recommendations, etc.
 Reasons for 'Waste' should be explained in detail. W/C to be measured by volume.

INSPECTOR'S DRILL RECORD (FOR GROUT OR DRAINAGE HOLES)

7-1744 (2-71)
Bureau of Reclamation

PROJECT TETON BASIN SPECS. NO. DC 6910
 FEATURE TETON DAM TYPE HOLES BLANKET
(A, B, DRAINAGE, ETC.)
 DETAIL SPILLWAY BIT SIZE 4X
(RT. ABUT., DIVERSION TUNNEL, SPWY., ETC.)

HOLE	DRILL TIME		DEPTH (FEET)		PERTINENT INFORMATION*	SHIFT & DATE	INSP. INITIALS
	START	STOP	FROM	TO			
					2" X 41" PIPE		
11+04	4:40 AM		0		STARTED DRILLING	6Y 12-10-74	Dm
18+4/5					DRILLER: G. NIELSEN		
		5:00 AM		5'	20% WATER LOSS		
		5:00 AM		5'	CONTINUED DRILLING		
		7:20 AM		42'	40% WATER LOSS	6Y 12-10-74	Dm
	4:40 ^{PM}		42'			SKIPS 12-10-74	S.F.
		6:30		60'	100% water loss at 59'	11	11
					stopped to grout		

*NOTE DEPTH OF CONCRETE IF ANY, FORMATION CHANGES, WATER LOSS OR GAIN, CAVE, ETC.

RECORD OF DRILLING AND GROUTING OPERATIONS

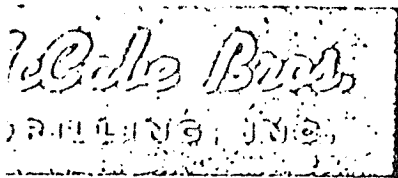
Project Teton Dam
Feature Rt. Abut. Key Truss (Dam, Spillway, Tunnel, etc.)
Drawing 549-147-193 Line Upstream Curtain (A, B, C, etc.)

Date May, 1975
Sheet 10 of 36

DRILLING

GROUTING

STA. OF HOLE	DEPTH OF HOLE			DATE DRILLED	REMARKS	DATE GROUTED	CEMENT PLACED	PRESSURE		W/C		MAX. SKS/HR	REMARKS
	IN CONC.	IN ROCK	STAGE DEPTH					MAX.	MIN.	MAX.	MIN.		
10+44			0-3		Pipe								
U/S	17	3-20	5-10			5-11	0	10	10	W	W	—	H ₂ O Test - Took 1.3 c.f. in 5 min.
	20	20-40	"			"	0	13	13	W	W	—	H ₂ O Test - Took 1.4 c.f. in 5 min.
	20	40-60	"			"	0	26	26	W	W	—	H ₂ O Test - Took 1.6 c.f. in 5 min.
	20	60-80	"			"	0	39	39	W	W	—	H ₂ O Test - Took 1.4 c.f. in 5 min.
	20	80-100	"			"	0	52	52	W	W	—	H ₂ O Test - Took 1.9 c.f. in 5 min.
	30	100-130	"			"	0	65	65	W	W	—	H ₂ O Test - Took 1.7 c.f. in 5 min.
	30	130-160	"			"	5	84	84	8:1	8:1	4	H ₂ O Test - Took 2.5 c.f. in 5 min.
	30	160-190	"			"	0	100	100	W	W	—	H ₂ O Test - Took 0.7 c.f. in 5 min.
	30	190-220	"			"	0	100	100	W	W	—	H ₂ O Test - Took 0.8 c.f. in 5 min.
	30	220-250	"			"	1	100	100	8:1	8:1	1	H ₂ O Test - Took 0.3 c.f. in 5 min.
	30	250-280	"			"	2	100	100	8:1	8:1	2	H ₂ O Test - Took 3.3 c.f. in 5 min.
	30	280-310	"			5-10	11	100	100	8:1	8:1	12	H ₂ O Test - Took 6.7 c.f. in 5 min. Hole Complete
10+91			0-3		Pipe								
U/S	17	3-20	5-10			5-11	2	10	10	8:1	8:1	2	No H ₂ O Test
	20	20-40	"			"	1	13	13	8:1	8:1	2	H ₂ O Test - Took 2.8 c.f. in 5 min.
	20	40-60	"			"	0	26	26	W	W	—	H ₂ O Test - Took 1.8 c.f. in 5 min.
	20	60-80	5-11			"	0	39	39	W	W	—	H ₂ O Test - Took 0.5 c.f. in 5 min.
	20	80-100	"			"	0	52	52	W	W	—	H ₂ O Test - Took 0.6 c.f. in 5 min.
	30	100-130	"			"	0	65	65	W	W	—	H ₂ O Test - Took 0.7 c.f. in 5 min.
	30	130-160	"			"	3	84	84	8:1	8:1	2+	H ₂ O Test - Took 2.9 c.f. in 5 min.
	30	160-190	"			"	0	100	100	W	W	—	H ₂ O Test - Took 1.8 c.f. in 5 min.
	30	190-220	"		Grout seems @ 210' & 220'	"	0	100	100	W	W	—	H ₂ O Test - Took 1.9 c.f. in 5 min.
	30	220-250	"			"	0	100	100	W	W	—	H ₂ O Test - Took 1.0 c.f. in 5 min.
	30	250-280	"			"	0	100	100	W	W	—	H ₂ O Test - Took 0.8 c.f. in 5 min.
	30	280-310	"			"	0	100	100	W	W	—	H ₂ O Test - Took 0.5 c.f. in 5 min. Hole Complete



DIAMOND CORE DRILLING
PRESSURE GROUTING

ROTARY DRILLING
FOUNDATION TESTING

BOX 1892. IDAHO FALLS. IDAHO 83401
PHONE 522-5437

August 25, 1976

RECEIVED

AUG 26 1976

Morrison-Knudsen Company, Inc.
P. O. Box 127
Newdale, Idaho 83436

Morrison-Knudsen - Kiewit
Contract No. 2594

Attention: Mr. Duane E. Buckert

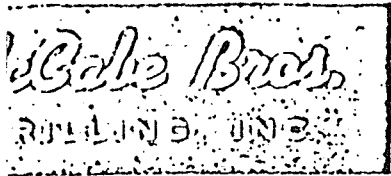
Re: Letter from 'Independent Panel to
Review Cause of Teton Dam Failure'
to Morrison-Knudsen Co. Inc. for
Information on Construction Techniques.

Gentlemen:

Referring to Question: The Manner in which Axial grout distribution and Closures were assured when the up and downstream grout travel was relatively unlimited.

There were three grout lines an upstream, a center and a downstream. The downstream grout line was from Station 2 + 20 to 16 + 00; The upstream grout line was from Station 2 + 28 to 15 + 94 and the center grout line was from Station 2 + 23 to Station 18 + 00 and on. Most of the grout nipples were 2" diameter. The Area holes were located over fairly large cracks and the nipples were set to intercept the cracks at different depths, some being set vertically over cracks with concrete poured around them. The downstream holes were vertical with 20 ft. centers. The upstream holes were at a 30° angle with 20 ft. centers, except one vertical at station 5 + 28 and one fan hole at 2 + 28 37°. The centerline holes were at a 30° angle with 10 ft. centers.

There are no closure holes on the downstream line. There are three fan holes at station 2 + 20; one at 15°, one at 30° and one at 45°. The upstream line has the following closures: 3 holes on 5' centers, 6 holes on 6' centers, 1 hole on 7' center, 2 holes on 9' center, 1 hole on 10' center, 3 holes on 11' centers, 2 holes on 12' centers, and 2 holes on 13' centers. As directed by the contracting officer, the holes from station 9 + 22 to 10 + 00 in the upstream line were deleted.



DIAMOND CORE DRILLING — ROTARY DRILLING
PRESSURE GROUTING — FOUNDATION TESTING

BOX 1892. IDAHO FALLS. IDAHO 83401
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The primary holes were staggered from each other on the three grout lines. The Area holes that were set close to cracks were grouted first. We drilled the holes until we lost 50% or more of our drill water. Then commenced grouting at the bottom stage of the hole, most of the area holes were intermittently grouted if the take was 500 cu. ft., with a waiting period of three hours. Eventually that stage of the hole would come up to the desired pressure required by the inspector. At that time we set the packer up to the next stage and progressed out of the hole through the different stages and finished grouting by hooking the nipple and grouting to the specified pressure. If the above hole was required to go deeper, we then drilled to the specified depth or until we had a water loss of 50% or more and then set the packer at the directed settings and grouted the different stages at required pressures until we staged up to the previous stage grouted, thereby completing the entire hole. Closures were added to area holes.

There are primary holes every 80 ft., secondary holes every 80 ft. and closure holes every 40 ft. on the downstream and upstream grout lines. The primary holes were drilled and grouted first, the secondary holes were drilled and grouted second and the closure holes were drilled and grouted last as directed. All grouting of holes was accomplished in the same manner described above for the area holes.

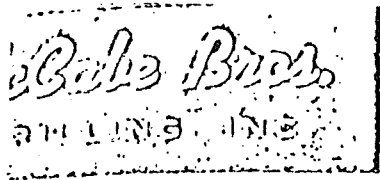
The centerline holes have a primary every 80 ft., a secondary every 80 ft. and an intermediate hole every 40 ft. with closure holes every 20 ft. The primary holes were drilled and grouted first, the secondary holes were drilled and grouted second, the intermediate holes were drilled and grouted third and the closure holes were drilled and grouted last as directed. Of course the centerline has a more complex pattern than the downstream and upstream grout lines and is designed to serve as a closure line for the downstream and upstream grout lines, with many closure holes being added.

Good packer settings were accomplished with the very minimum of difficulty.

A large percent of the holes where the water loss was negligible, we were able to drill to the complete depth of the hole, in this case we grouted from the bottom stage up, until the hole was completely grouted.

About 98 or 99% of the stages in all holes were water tested, with the exception of the top 20 ft. in many holes. The migration of water from the water tests and the grout travel into other drilled holes was very minimal. All holes were completely backfilled with grout after all stages were completed. All grout leaks to surface areas were calked immediately and continuously until leakage stopped.

Referring to Question: Details of any doubts over the effectiveness of this Axial distribution in any particular location



Page 3

The only doubt that we have been concerned with was the high percent of Calcium Chloride being used. The highest percent used as directed by the inspectors was 10% for a short time, later this was lowered to 8%.

As an example, if the hole was 100ft. deep and we were grouting with about 3% or more Calcium Chloride, intermittently grouting the bottom stage and finally the bottom stage came up to pressure, then we stage grouted the hole up to the surface. At this time we were directed to deepen the hole to 130 ft. We then drilled the hole down to 90 ft. and had a total water loss. Then we set the packer above the water loss in the bottom stage of the hole and started grouting until the stage came up to pressure - sometimes this stage required intermittent grouting. This condition has happened many times when using Calcium Chloride. The question we have asked ourselves about the above problem is - Is this same condition happening in the grouting of large or small cracks and fissures, causing a honey-comb effect. Thereby causing many more holes to be drilled and grouted on the centerline than otherwise would be necessary.

Referring to Question: Details of difficulties in obtaining assurance of axial closure at any station or grout hole along the same stretch of curtain.

We had no difficulty in grouting up to the desired pressure in all stages of all closure holes. If one closure hole took more than the minimum amount of grout in any one stage, we were then ordered to drill and grout additional closure holes.

Referring to Question: Similarities and significant differences in the appearance of the walls and floor of the Key trench in the right and left abutments.

Right
The left abutment had a few caverns in the walls and the floor of key trench appeared to be good solid rock. Directly up the grout line, 75 to 125 ft. above the tower on the steepest slop, we had some grout leaks around some large boulders and as we were calking these leaks, numerous bats were flying out of the cracks that we were attempting to calk.

The right abutment from about Station 18 +00 to Station 11 + 50 appeared to be very badly fractured with small cracks in the walls and the floor of the trench. From Station 11 +50 to 10 +00 it appeared to be good solid rock. From Station 10 +00 to 7 +50 it appeared to be good solid rock on the walls and floor of the key trench, with some small visable fissures. From 7 + 50 to 2 + 00 the walls and floor appeared to be of good sound rock with a very little small fracturing with the exception of numerous large faults visable in the walls and floor of the key trench.



DIAMOND CORE DRILLING — ROTARY DRILLING
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The lake bed sediments that underlay the riolite formation along the extreme length and width of the dam at depth were drilled and grouted to the desired pressures for a short distance on the outer end of the left abutment.

All work described above was completed as directed by the contracting officer, Bureau of Reclamation.

Very truly your,

McCABE BROS. DRILLING, INC.

Edwin L. McCabe

Edwin L. McCabe
President

ELM:rjn

Please describe:

B. The manner in which the key trench between Station 18+00 and Station 2+00 was prepared to receive the first embankment material. Compare the way in which this trench was prepared with "broom clean." If there were differences in clean-up between particular stations, because of weather, or any other cause, please describe such differences in detail.

(B) The key trench between Station 2+00 and 18+00 was all cleaned in basically the same manner. Laborers using hand shovels and bars would first remove any loose rock or earth materials from the rock foundation. An air jet was then used to clean any remaining finer material down to a clean rock condition. Any grout which had been spilled in key trench areas was loosened by paving breaker and cleaned by air jet. Cleanup of key trenches and all abutment areas generally progressed to 2 to 10 feet above the elevation of the Zone 1 fill. Material accumulated during cleanup was removed by a rubber-tired backhoe.

Prior to placement of each lift of specially compacted Zone 1 material, the abutment rock which had been cleaned by shovels and air jets was always sprayed with water to assure a proper bond with the fill material.

No particular areas in the foundation key trench received a different type of treatment from the rest of the key trench. The air jet and water treatment method of cleaning the abutment rock was considered superior to broom clean because the use of air jets and water resulted in a more thorough cleaning of cracks and irregularities in the rock surface than with the broom method.

Please describe:

C. The manner in which any fissures or open joints in the key trench walls and floor were sealed between Station 18+00 and Station 2+00; that is, the manner in which, and the places where, slush grouting, dental concrete, gunite, or shotcrete may have been used, also the extent to which such sealing was general. Were any joints left unsealed and, if so, where? If known, please indicate the particular stations, if any.

- (C) The excavation for the right abutment keyway trench disclosed two unusually large fissures that cross the floor and extend into the walls of the keyway near the toe of the walls. On the floor of the keyway, the fissures were filled with rubble; but at both locations, the contractor excavated a trench about 3 to 4 feet wide and about 5 feet deep. Both fissures apparently were developed along joints that strike about N80 W and are vertical to steeply inclined. The largest fissure crossed the keyway from station 4+44 of the upstream face to station 3+45 on the downstream face. The strike of a smaller fissure was about N75 W and crossed the keyway trench from station 5+33 of the upstream face to station 5+11 of the downstream face.

The largest and most extensive open zone extended into the upstream wall from the toe of the keyway wall near station 4+44. The opening at the toe was about 5 feet wide and 3 feet high. There was a rubble-filled floor about 4 feet below the lip of the opening. A few feet in from the wall the fissure was about 7 feet wide, but a very large block of welded tuff detached from the roof and/or the north wall rested in the middle. Beyond the large block about 20 feet in from the opening the fissure narrowed to about 2-1/2 feet wide. The rubble floor sloped gently away from the opening and the vertical clearance was about 10 feet. About 35 feet in, the rubble floor sloped rather steeply and the roof tilted sharply upward. About 50 feet in from the opening, the vertical clearance was about 40 feet and the fissure curved out of sight at the top. About 75 feet back, the fissure curved slightly southward out of view. The smaller fissure was mostly rubble-filled and was open only at the upstream face. The opening was about 1 foot square at the face and the fissure appeared to be rubble-filled about 5 feet back from the face.

The continuation of this fissure intersected the downstream wall of the keyway near station 4+21. The opening was about 4 feet wide and 4 feet high. A rubble-filled floor lay about 4 feet below the lip of the opening. The large opening extended only about 5 feet back from the face and then a foot wide fissure at the north edge continued about 10 feet back and about 10 feet upward before going out of view.

The other large open zone extended into the upstream wall from the toe of the wall near station 3+66. The opening at the toe of the wall was about 1-1/2 feet wide and 1-1/2 feet high. From the opening, the fissure extended about 10 feet down to a rubble floor and about 15 feet back before going out of view. The continuation of this fissure intersected the downstream wall of the keyway at about station 3+45. There was no open fissure at the downstream wall but

there was a 3-1/2-foot-wide zone of very broken rock with open spaces up to 0.8 foot wide. About 2-1/2 feet north, there was an open joint about 10 feet long and 0.2 feet wide that dipped about 78 degrees south.

At both the upstream and downstream locations of the fissure zones, broken rock extended to about midway up the keyway walls. Above the broken zones there appeared to be filled fissures about 0.5 foot wide that extended vertically to the top of the keyway cut.

Two 9-5/8-inch-diameter holes were bored to intersect the open fissure that extended into the upstream and downstream walls of the keyway trench. One hole was located 72 feet upstream from dam axis station 4+64 and the other was located 75 feet downstream from dam axis station 4+02. The upstream extension of the fissure was encountered at a depth of 68 feet and the downstream extension was encountered at a depth of 58 feet. The holes were cased with 8-5/8-inch-diameter steel casing. High-slump concrete was poured through these casings into the fissures. Ninety-five cubic yards of concrete was placed in the upstream hole and 233 cubic yards was placed in the downstream hole in April 1974.

Three 3-inch-diameter vertical drill holes were bored 77 feet downstream from dam axis station 3+30 to explore for a possible open fissure indicated by earlier horizontal drill holes bored from the floor of the keyway trench. The vertical holes encountered some voids and some soft, broken, or loose rock; however, these voids did not appear to be of sufficient volume to warrant drilling large diameter holes for backfilling with concrete.

In May 1974, an additional 18 cubic yards of high-slump concrete was placed in the 8-5/8-inch-diameter-cased hole which intercepts the open fissure 75 feet downstream from station 4+02. A total of 251 cubic yards of high-slump concrete was placed in this hole. Drawings No. 549-147-133 and -134 (Exhibits 12.10.11 and 12.10.12) show the location of the holes, the estimated outline of the fissures, and the concrete that was placed into the fissures.

Other open joints or holes were observed on the floor of the keyway near centerline at stations 5+03, 5+68, and 6+18 and about 5 feet left of centerline between stations 6+03 and 6+08. The holes were rubble filled at shallow depths and their lateral extent, if any, was covered by rubble. Heavy calcareous deposits were associated with all of the open zones except for a 0.2-foot-wide open joint between stations 6+03 and 6+08.

The joints between station 5+03 and 6+08 were filled with grout during the grouting operation.

Dental concrete was placed in an open jointed area on the spillway floor at approximate station 9+00 where the 1-1/2:1 slope of the key trench meets the spillway floor.

The surface grouting on the abutments began because of the numerous joints in the rocks. This grouting was started on July 29, 1974 and was completed about August 6, 1975.

The joints in the rock between elevation 5055 and 5205 were grouted to refusal by mixing grout in the mix trucks and placing it in the crack or joint by making a funnel out of the zone 1 material around the cracks and dumping it in out of the trucks. The smaller cracks, approximately 1/2 inch to 2 inches wide, were grouted with a 0.7 to 1 mix by volume. For gravity filling the larger cracks, approximately 3 to 4 inches wide, a sand-cement grout was used. These cracks were marked and filled by inspectors for zone 1 special compaction placing.

Occasionally, the batch plant could not place grout in these cracks daily. Therefore, the zone 1 special compaction was held up until the cracks could be grouted. At times, the fill would get ahead of the special compaction a foot or 2, but this was not a problem because the batch plant operated on two shifts and grout could be placed during the graveyard shift when the fill was shut down.

The foundation keyway and abutment rock above elevation 5205 had fewer open joints than below this elevation. Generally the rock in the keyway was more massive and the joints and cracks very small; hence, the slurry grouting above elevation 5205 became impracticable. It was noted also that the fewer large joints above elevation 5205 were usually filled with rubble or silt which also added to the difficulty of treating these joints.

Please refer to the detailed geologic maps of the abutment and key trench areas for a description of the joints and fissures. The panoramic photos of key trenches and zone 1 foundation rock will also reveal the more massive nature of the rock in the key trenches.

No fissures or large joints were knowingly left untreated. A tabulated list of the locations where slurry grout was used is attached.

The fissures crossing the key trench at stations 3+55 and 4+34 were excavated similar to the grout cap trench and filled with concrete.

SLURRY GROUT USED TO FILL CRACKS AND
FISSURES IN RIGHT ABUTMENT

(c)	Date	Station	Offset	Volume in Cu. Yds.
	8-14-74	16+50		1.00
	8-19-74	16+30	340' - 355'	1.00
	8-22-74	16+10	300' - 350'us	2.00
	9-3-74	15+95	75' ds	1.00
	9-3-74	15+95	250' us	1.00
	9-5-74	16+00	15' ds	4.00
	9-5-74	16+20	74' ds	4.00
	9-5-74	16+20	176' us	4.00
	9-5-74	15+90	64' ds	2.00
	9-5-74	15+90	150' us	2.00
	9-6-74	16+00	35' ds	1.00
	9-6-74	16+10	45' ds	1.00
	9-6-74	16+20	74' ds	1.00
	9-10-74	16+00	190' ds	6.00
	9-10-74	16+00	15' - 90'ds	6.00
	9-10-74	16+00	300' us	6.00
	9-13-74	16+00		4.00
	9-17-74	16+00	60' ds	2.00
	9-24-74	15+70	50' & 75' ds	20.00
	9-25-74	15+70	50' & 75' ds	48.00
	10-3-74	15+80	125' ds	9.00
	10-3-74	15+50	48' us	0.50
	10-3-74	15+60	75' us	0.50
	10-4-74	16+20	300' us	8.00

Date	Station	Offset	Volume in Cu. Yds.
10-4-74	16+75	150' us	1.50
10-4-74	16+50	128' ds	0.50
10-7-74	15+60	35' us	6.00
10-10-74	15+70	40' ds	1.00
10-10-74	15+70	135' ds	3.00
10-11-74	15+80	10' ds	2.00
10-11-74	15+80	centerline	4.00
10-11-76	15+80	265' us	38.00
10-11-74	15+80	258' us	3.00
10-11-74	15+80	223' ds	3.00
10-14-74	15+50	125' ds	0.25
10-14-74	15+50	273' ds	5.75
10-15-74	15+40	100' us	0.50
10-15-74	15+50	273' - 278' us	35.50
10-16-74	14+80	35' us	0.50
10-16-74	15+50	273' - 278' us	0.50
10-16-74	15+30	30' us	17.00
10-17-74	15+30	30' us	0.50
10-17-74	14+80	35' us	8.50
10-17-74	15+40	12' ds	8.50
10-18-74	15+50	50' ds	6.00
10-18-74	15+30	12' us	6.00
6-3-75	14+63	84' us	5.00
6-3-75	14+37	82' us	3.00
6-6-75	14+70	110' us	39.50
6-10-75	15+20	150' ds	13.50
6-11-75	15+20	150' ds	1.00

Date	Station	Offset	Volume in Cu. Yds.
6-11-75	15+15	115' ds	4.25
6-11-75	15+10	90' ds	1.50
6-11-75	15+00	60' ds	1.25
6-11-75	14+90	30' ds	0.50
6-11-75	14+78	3' ds	7.00
6-11-75	14+25	35' us	5.50
6-11-75	14+40	80' us	3.00
6-11-75	15+30	110' us	1.50
6-11-75	15+35	115' us	1.00
6-11-75	15+60	150' us	1.00
6-11-75	15+80	245' us	2.00
6-11-75	15+40	120' us	17.00
6-11-75	15+25	106' ds	1.00
6-13-75	15+00	105' ds	0.25
6-13-75	15+00	40' ds	8.75
6-13-75	15+00	29' ds	15.00
6-13-75	14+63	15' ds	0.25
6-13-75	14+30	15' us	1.50
6-13-75	14+03	30' us	2.00
6-13-75	14+25	15' us	0.25
6-13-75	15+30	centerline	2.00
6-13-75	15+50	190' us	5.00
6-13-75	15+00	123' ds	3.00
6-13-75	15+00	111' ds	7.75
6-13-75	15+00	98' ds	2.00
6-13-75	14+80	15' ds	0.25
6-13-75	14+75	7' us	0.25

Date	Station	Offset	Volume in Cu. Yds.
6-13-75	14+60	95' us	2.75
6-13-75	15+40	120' us	0.50
6-13-75	15+50	220' us	1.00
6-13-75	15+50	245' us	1.50
6-16-75	3+60	us slope trench	37.00
6-16-75	4+43	us slope trench	7.00
6-16-75	3+44	ds slope trench	10.00
6-16-75	4+21	ds slope trench	10.00
6-18-75	15+00	125' ds	7.00
6-18-75	15+00	120' ds	110.00
6-19-75	14+80	115' ds	12.00
6-19-75	15+00	120' ds	27.50
6-19-75	15+00	150' ds	7.50
6-19-75	15+00	140' ds	20.00
6-19-75	15+00	90' ds	4.00
6-19-75	15+00	75' ds	2.00
6-19-75	15+00	150' ds	14.50
6-19-75	15+00	70' ds	1.00
6-19-75	15+00	127' ds	1.50
6-20-75	15+00	103' ds	0.25
6-20-75	15+00	105' ds	0.25
6-20-75	15+00	50' ds	0.25
6-20-75	15+00	68' ds	0.25
6-20-75	14+45	10' ds	0.25
6-20-75	14+40	5' ds	3.50
6-20-75	14+30	centerline	0.25
6-20-75	14+50	100' us	0.25
6-20-75	14+60	110' us	0.50

Date	Station	Offset	Volume in Cu. Yds.
6-20-75	14+70	120' us	0.25
6-20-75	14+52	104' us	6.00
6-20-75	14+18	10' us	0.50
6-20-75	15+00	140' ds	4.00
6-20-75	14+95	85' ds	1.00
6-20-75	14+95	75' ds	4.00
6-20-75	14+90	30' ds	12.00
6-20-75	14+15	5' ds	1.00
6-20-75	14+45	90' us	5.00
6-20-75	14+25	25' ds	1.00
6-20-75	15+20	115' ds	1.00
6-20-75	15+20	120' us	1.00
7-1-75	Key way rt. of spillway	110' us	6.00
7-1-75	"	115' us	0.25
7-1-75	"	120' us	3.00
7-1-75	"	100' us	0.25
7-1-75	"	12' us	7.00
7-1-75	"	25' ds	0.25
7-1-75	"	60' ds	1.25
7-1-75	"	80' ds	12.00
7-1-75	"	75' ds	0.50
7-1-75	"	100' ds	0.50
7-1-75	"	125' ds	29.00
7-2-75	15+10	205' us	12.00
7-2-75	15+15	210' us	2.00
7-2-75	14+50	110' us	1.50
7-2-75	14+85	130' us	9.50

Date	Station	Offset	Volume in Cu. Yds.
7-2-75	14+35	100' us	9.00
7-2-75	14+50	10' ds	1.00
7-2-75	15+10	75' ds	0.50
7-2-75	15+25	125' ds	1.00
7-9-75	14+30	100' us	9.00
7-9-75	14+55	centerline	8.00
7-9-75	14+90	160' us	14.00
7-9-75	14+90	225' us	6.00
7-9-75	14+18	2' us	1.00
7-9-75	14+80	55' ds	6.00
7-9-75	14+85	80' ds	2.00
7-9-75	14+90	101' ds	2.00
7-10-75	14+10	8' us	2.00
7-10-75	14+25	5' ds	2.00
7-10-75	14+30	10' ds	6.00
7-10-75	15+00	210' us	1.00
7-10-75	15+20	115' ds	7.00
7-10-75	15+40	150' ds	6.00
7-11-75	4+18	27' ds	16.00
7-11-75	4+42	27' us	12.00
7-11-75	15+20	115' ds	28.00
7-11-75	15+30	132' ds	1.00
7-11-75	15+40	150' ds	1.00
7-11-75	13+85	10' us	5.00
7-11-75	14+50	23' ds	1.00
7-11-75	14+70	125' us	28.00
7-11-75	14+00	98' us	2.00
7-11-75	14+10	107' us	3.00

Date	Station	Offset	Volume in Cu. Yds.
7-11-75	14+40	122' us	1.00
7-11-75	14+60	125' us	1.00
7-11-75	15+00	148' us	4.00
7-11-75	14+80	133' us	10.00
7-11-75	13+85	8' us	4.00
7-11-75	15+10	80' ds	4.00
7-14-75	13+75	20' us	1.00
7-14-75	13+80	78' us	14.00
7-14-75	14+10	108' us	1.00
7-14-75	14+50	30' ds	12.00
7-14-75	14+45	25' ds	2.00
7-14-75	15+10	80' ds	2.00
7-18-75	15+20	135' ds	7.00
7-18-75	15+10	125' ds	30.00
7-18-75	15+00	120' ds	17.00
7-18-75	15+20	125' ds	8.00
7-18-75	15+00	50' ds	1.00
7-18-75	15+10	68' ds	3.00
7-21-75	15+00	110' ds	3.00
7-21-75	14+90	95' ds	3.00
7-21-75	14+60	55' ds	2.00
7-21-75	14+45	40' ds	4.50
7-21-75	13+90	85' us	5.00
7-21-75	14+07	107' us	3.00
7-21-75	14+25	125' us	0.50
7-21-75	14+10	5' ds	24.00
7-24-75	14+00	15' us	13.00

Date	Station	Offset	Volume in Cu. Yds.
7-24-75	14+02	10' us	11.00
7-28-75	14+25	145' us	6.50
7-28-75	14+06	110' us	1.00
7-28-75	13+85	97' us	2.00
8-1-75	13+85	95' us	1.00
8-1-75	14+00	100' us	7.00
8-4-75	13+86	9' us	33.00
8-4-75	14+12	115' us	7.00
8-4-75	14+12	138' us	13.00
8-4-75	14+12	168' us	11.00
8-4-75	13+96	7' us	8.00
8-5-75	14+00	100' ds	2.00

Please describe:

D. The method of material selection, preparation, placement and compaction, in the key trench, of the "specially compacted earthfill" shown in the cross section marked "Foundation Key Trench" on USBR Drawing No. 549-D-9. If special difficulties were encountered in selection, preparation, placement or compaction at any points along the length from Station 18+00 to Station 2+00, please describe each.

(D) The material selected for zone 1 special compaction in the foundation key trench was excavated in Borrow Area "A" with a Barber Greene wheel excavator. Borrow Area "A" material was pre-wet by irrigation sprinklers. The wheel excavator removed material in cuts up to 13 feet in depth and the material received a thorough blending of gradation and moisture by this method. Selection of the best available material for compacting with hand tampers was accomplished by the contractor's quality control engineer and pit foreman. The Bureau inspector in the foundation key trench area inspected the special material on the basis of moisture and also the amount of caliche as well as the suitability of the material for compaction against the rock. The contractor's quality control personnel and the Bureau inspector selected material with moisture content near optimum, low caliche content, and highest possible plasticity available from the borrow area.

Moisture was controlled in specially compacted material by mixing dry material with material which was too wet to reduce moisture content or by adding water to material which was too dry. Special compaction material was deposited near the abutment and then placed by dozers and laborers using hand shovels. Proper moisture content was determined by the inspector and checked by the lab test.

Material was compacted using gasoline and air tampers in irregular areas along the abutments and key trench and by a loaded Euclid 74-TD end dump truck or by a loaded Caterpillar model 992 front end loader. Material was compacted in 3-inch lifts by the gasoline and air tampers and in 6-inch lifts by the loaded equipment method. If a laboratory test of specially compacted material revealed that moisture limits were exceeded, failing material was removed, reworked, and then replaced. The area was recompacted when failure was due to low density. Rework area was generally 50 to 100 feet on each side of the test failure.

Between Stations 2+00 and 18+00, a total of 425 density tests of the specially compacted material were taken in the foundation key trench and along the right abutment zone 1 foundation. The average optimum moisture content of this material was 19.1 percent and placed at an average of 0.6 percent dry of optimum. The average "C" value of this material was 98.2 percent and "D" value averaged 97.2 percent. The silty material was difficult to compact in the foundation key trench special compaction area and along the abutment in the special

compaction areas. This is illustrated by the fact that of the 425 tests taken between Stations 2+00 and 18+00, 114 tests failed either for moisture or density deficiencies and required reworking or additional compaction; however, these areas were retested after being reworked and brought up to specifications requirements.

Field experience with this silty material demonstrated that the inherent nature of the material, particularly its low plasticity, made compaction by hand tampers difficult and a very concentrated effort was required to obtain a good job. However, there were no areas not placed to specifications requirements and particular attention was given to obtaining both moisture and density uniformity along the abutment rock contact in these special compaction areas.

Please refer to our reply to Question F., "Cleanup and Special Compaction - General" for additional information.

Please describe:

E. The method of material selection, preparation, placement, and compaction in the key trench between Station 18+00 and Station 2+00 of the core material. If special difficulties were encountered in selection, preparation, placement or compaction at any points along the length from Station 18+00 to Station 2+00, please describe each.

(E) Material in the key trench core area was selected by the same method as in paragraph (D). The borrow area was prepared in the following manner: A cut depth was determined from topographic notes to establish the desired drainage pattern. The borrow pit was then divided into material blocks. Three-inch holes were augered to the depth of cut required on a 200-foot grid and proctor optimum moistures were run on the drill cuttings and noted on the borrow pit drawings. Moisture was added to the pit by sprinkling the required water for the design cut on the area at least 3 weeks prior to excavation. Constant monitoring was possible by utilizing a speedy moisture teller. Material on the zone 1 fill received extra water from water trucks if required. The material was spread in about 8- to 9-inch-thick uncompacted lifts and rolled with two Caterpillar 825B self-propelled sheepsfoot rollers with caron wheels and two Ferguson SP-120-P self-propelled sheepsfoot rollers. Two Caterpillar motor graders with scarifier attachments provided supplemental scarifying on embankment as moisture was being added.

The method of excavation in the borrow pit by the Barber Greene wheel excavator resulted in a very homogeneous mixture of zone 1 material. Moisture and gradation reached a high degree of uniformity by the mixing action of the wheel excavator and the subsequent loading into the trucks by the belt. Further uniformity was attained by spreading and working of the material on the fill. The average density of all zone 1 fill placed was 98.3 percent of laboratory maximum with an average optimum moisture of 19.6 percent and placed at an average of 1.0 percent dry of optimum.

No special difficulties were encountered in placing the core material to the required density.

Please describe:

F. The manner in which the contact area under the core of the dam outside of the key trench was prepared to receive the first core material. If special difficulties were encountered at any location along the length of dam between Station 18+00 and Station 2+00, please describe.

(F)

CLEANUP AND SPECIAL COMPACTION - GENERAL

Placement of zone 1 embankment at Teton Dam began in the cutoff trench on October 18, 1973 with zone 1 material being transported by beltline conveyor from the left abutment to the trench bottom. Special compaction of the zone 1 material began on October 19, 1973, initiated by two laborers operating pneumatic tamping hammers and gas powered wackers along the perimeter of the embankment area from station 17+75 on the dam axis to station 19+50, 200 feet upstream.

While the zone 1 dam embankment material consisting of clay, silt, and sand could have rocks with dimensions of 5 inches or less, the zone 1 embankment material placed in locations requiring special compaction consisted of clay, silt, and sand with rock fragments having maximum dimensions of no more than 1 inch. Any portion of the dam embankment where zone 1 material was placed and could not be adequately compacted by sheepsfoot roller was specially compacted. These areas include zone 1 material adjacent to rock abutments, concrete structures, and any steel pipe or steel structures which would be embedded in the zone 1 embankment. Special compaction was accomplished for an average horizontal distance of 2 feet from any surface contacted by the zone 1 embankment. Standard procedure for placing a lift of zone 1 fill consisted of dumping the material from belly dumps and placing the lift with dozers to a depth which would equal 6 inches when compacted. An uncompacted lift of 9 inches generally compacted to a depth of 6 inches. Areas of fill which could not be placed by dozer were placed by laborers with hand shovels. Equipment such as dozers and sheepsfoot rollers were not allowed to contact the abutment or any other surface requiring special compaction of adjacent embankment material to assure that no damage would occur to the surface and that no rock would be loosened or dislodged from the abutment.

Abutment cleanup along the zone 1 embankment consisted of removal of all vegetation, including roots, larger than one-fourth inch in diameter, leaving clean rock. Any earth attached to the rock was removed by air jet or hand shovel. Any grout which had been spilled in key trench areas was chipped out by jack hammer and cleaned by air jet. Cleanup of abutments generally progressed 2 to 10 feet above the elevation of the zone 1 fill. Material accumulated during cleanup was removed by rubber-tired backhoe.

Prior to placement of each lift of specially compacted zone 1 material, the abutment or other surface which had been cleaned by handwork and air jets was always sprayed with water to assure a proper bond of the fill material to this surface. A minimum of eight passes was made by a loaded Euclid 74-TD

end dump or other approved piece of rubber-tired equipment over specially compacted areas forcing the clay material into the wetted cracks in the rock abutment. (See photo P549-147-5732, Exhibit 34.) All surfaces were clean prior to placement. Areas not reached by wheelrolling were power-tamped by gasoline or air tampers to such a degree that the compaction and density requirements were met. (See photo P549-147-5731, Exhibit 34.)

Before placing a new lift of specially compacted zone 1 material, the previous lift was scarified by discing the surface. Any areas which could not be reached by the disc were scarified by hand with shovels to assure a good bond with the following lift and to prevent a smooth bonding surface which could possibly allow movement of water along this boundary in the future. Moisture was controlled in specially compacted material by mixing dry material with material which was too wet, to reduce moisture content, or by adding water to material which was too dry. Material was worked to the proper moisture content near the abutment and then placed. It was difficult to adjust the moisture content of material already in place along the abutment. Proper moisture content was determined by the inspector and checked by the lab test. Following a test, failing material was removed, reworked and then replaced to correct a failure in moisture content. The area was recompacted when failure was due to low density. Rework area was generally 50 to 100 feet on each side of the test failure.

Material was specially compacted around 36-inch pipe encasing dewatering pumps station 18+85, 75 feet upstream and at station 19+70, 175 feet upstream. (See photo P549-147-3254 NA, Exhibit 34.) Plastic dewatering pipes at the bottom of the trench were also embedded in specially compacted earthfill. Saturated material along the upstream and downstream toe of the embankment was removed with a Case 580B backhoe. The areas were then backfilled with gravel to prevent water from pooling or saturating placed embankment material. Zone 1 material was then specially compacted over the gravel beds, or French drains, using a rubber-tired 992 front loader with its bucket filled with zone 1 material.

Special compaction of zone 1 fill continued from station 18+50 to station 18+75, 190 feet upstream to 200 feet upstream, elevation 4937 to 5041, using gas-powered plate wackers around the pump encasement and drains. Rock abutments were cleaned of all vegetation and loose material by hand-work and air jets as construction of the embankment progressed.

By October 29, 1973, the pipe from the dewatering pump at station 18+85 was embedded in specially compacted material to elevation 4961. An area 6 feet wide for a depth of 4 feet above the pipe was compacted with gas-powered plate wackers, from the pump to the downstream toe of the embankment. With a 4-foot depth of material over the pipe, it was possible for a sheepsfoot roller to compact the fill in this area. On November 7, 1973, the two 50-hp dewatering pumps were removed from the embankment and the 36-inch pipe encasements were filled with concrete. Special compaction with wackers began around the zone 1 belt tower footings on this date at

station 16+65 on the right abutment. This was the last day of fill placement as snow closed the embankment for the season.

The contractor resumed embankment operations on April 4, 1974, with zone 1 material being placed and compacted at the toe of the right abutment and around the base of the beltline tower. After dropping through the tower, the material was loaded and spread by a Cat. 966 loader. The fill was compacted adjacent to the abutment by wheelrolling with the Cat. 966 loader and around the tower legs with gas-powered plate wackers. An access ramp was constructed for scrapers at the base of the tower during this operation. Elevation of the fill around the tower legs on April 11, 1974 was 5015.5 feet.

Laborers continued to clean the basalt formation at station 21+55 on the left abutment. Mud and grout along the grout cap on both abutments and loose debris were removed to a waste area on the downstream slope of the embankment by a Cat. 988 loader. Fill was compacted along the rhyolite wall of the right abutment with pneumatic tamping hammers. The abutment was wetted by a Cat. 631B water wagon to assure proper bond with the embankment.

On April 29, 1974, a 6,000 gallon Cat. 631B water wagon compacted fill material adjacent to the abutment making eight passes of the wheel at each location.

By May 29, 1974, areas of special compaction of zone 1 included material around the legs of zone 1 beltline tower station 16+58, 50 feet upstream from the dam axis, the right zone 1 abutment station 16+50 to station 17+50, 100 feet downstream to 340 feet upstream from centerline, and the left abutment from station 22+50 to station 24+00, 100 feet downstream to 300 feet upstream. Average elevation of embankment was 5022 feet.

On July 8, 1974, the contractor began using a Pierce Arrow pavement breaker or Hydra Hammer, with shoe area of the hammer approximately 1 square foot, for special compaction along the abutment and the materials handling tower supports. (See photo P549-147-4590 NA, Exhibit 34.)

On July 11, 1974, special compaction was interrupted to blast overhanging rock on the left abutment. Blasting, scaling, and cleanup continued for several days and special compaction resumed following this operation. Grout leaks from construction of the grout curtain in the right cutoff trench appeared along the right abutment special compaction area. Wet material was removed by motor patrol and new material was placed and compacted.

A "Ho-pac" compactor arrived for use on July 17, 1974 and a Case 580B backhoe with special vibrator attached to the end of the backhoe to be used for compaction arrived on July 22, 1974. Use of the "Ho-pac" was discontinued because of the high number of passes required to get adequate compaction. When it was not possible to compact zone 1 fill in very deep voids on the irregular abutments, it was necessary to fill these voids with backfill

grout to form an impervious rock foundation sealing off voids so earthfill material could not penetrate the foundation and cause an unstable embankment. A standard grout mix of 0.7:1 water-to-cement ratio was used for most backfill grouting operations.

On November 27, 1974, the contractor terminated operations for the season, resuming work on April 18, 1975, with cleanup on rock abutments. Standing water along the abutment on May 19, 1975, was drained toward the center of the fill and pumped downstream and fill material was scarified with an International TD15C dozer pulling discs and allowed to dry. Any material considered too wet along the abutments was removed by a Cat. 14E patrol, picked up by scrapers, and hauled to the zone 3 embankment to dry.

When construction of the right cutoff trench embankment began on June 10, 1965, a Cat. 966 rubber-tired loader was used for special compaction. After all concrete repair was complete at the Auxiliary Outlet Works gate shaft and right spillway counterforted wall station 10+55, elevation 5295 special compaction began on July 22, 1975. Gasoline-powered plate wackers were used there.

On October 20, 1975, cleanup of grout and debris began around the River Outlet Works gate shaft. Hand shovels and air jets were again used to clean the foundation. Placement of zone 1 material around the shaft began on October 24, 1975 with same special compaction procedures used as at previous concrete structures. By November 1, 1975, special compaction around the shaft was finished completing zone 1 special compaction on the dam embankment.

Please describe:

G. The manner in which core material was selected, prepared, placed, and compacted outside of the key trench, between Station 18+00 and Station 2+00. If special difficulties were encountered, please describe in detail by specific location.

- (G) There is no significant difference between this operation and the operation described in the answer to (E) previously given. The key trenches, except for special compaction areas, were a continuation of the zone 1 fill operation. The key trenches were wide enough to permit the spreading and rolling equipment to place the key trench areas in a concurrent operation with the main body of zone 1 fill. The method of selection, preparation, placement, and compaction was the same outside of the key trench areas as inside them. We do not know of any particular difficulties associated with this operation.

The following description of the training procedures used for construction personnel and quality control efforts will provide an insight into efforts made to select and prepare the zone 1 material for placement in the embankment:

Prior to the start of zone 1 placement in the fall of 1973, the Bureau of Reclamation supervisory personnel met with those from Morrison-Knudsen-Kiewit to determine how to precondition and excavate material from Borrow Area "A." It was decided that, for the short construction season left, Bureau materials technicians would work in the pit directly with M-K-K personnel and that the Bureau would provide laboratory facilities for preplacement testing. This would help train M-K-K personnel for control of the pit during the following construction season.

During the winter shutdown following the 1973 construction season, the Bureau conducted a training session covering testing procedures for earthwork and concrete. Bureau laboratory personnel conducted the session in the project laboratory.

M-K-K requested that their supervisory personnel be allowed to attend these sessions. After completion of the initial training session, M-K-K requested that the Bureau have an additional day's training covering earthwork testing so that they could have additional personnel receive this training. This was done.

Prior to and during the early part of the 1974 construction season, M-K-K had three people work in the Bureau project laboratory to receive training before they were allowed to work in M-K-K's mobile laboratory, which was set up in Borrow Area "A" specifically for preplacement testing of the material for specifications compliance prior to placement in the dam. From the start of the 1974 construction season and through completion of the dam, M-K-K handled the preconditioning and testing of the borrow area prior to placing. The Bureau of Reclamation tested the material as

delivered to the dam for specifications compliance. The Bureau provided technical assistance and provided special testing whenever requested by M-K-K to maintain adequate control of the borrow area. Considerable control testing was needed in Borrow Area "A" due to the wide range of optimum moisture contents. The optimum moistures ranged from approximately 16 percent to 24 percent. It was difficult to determine visually or by hand tests whether the material from the pit was within the specifications limits from placement moisture.

Please describe:

H. Similarities and significant differences in the appearance of the walls and floor of the key trenches in the right and left abutments.

- (H) The walls and floor of the key trench in the right abutment generally appeared to have more cracks in the rock than the walls and floor of the key trench in the left abutment.

Both abutments, however, have a highly fractured zone in the top of the canyon wall in the rhyolite.

Profiles through the key trenches are, of course, quite different because of the 1-1/2:1 slope adjacent to the spillway on the right abutment key trench.

It is recommended that similarities and differences of the key trenches can best be understood by inspecting the panoramic color photos with geologic overlays and the detailed geologic maps made of the key trenches during construction.

UNITED STATES DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Wallace L. Chadwick, Chairman
Arthur Casagrande
Howard A. Coombs
Munson W. Dowd
E. Montford Fucik
R. Keith Higginson
Thomas M. Leps
Ralph B. Peck
H. Bolton Seed
Robert B. Jansen, Executive Director

October 27, 1976

*P.B.J.
10/27/76*

TO: Robert B. Jansen

FROM: Clifford J. Cortright

SUBJECT: Review of "L-29 Construction Reports," Teton Basin Project,
Lower Teton Division, May 1972 through December 1973

I have reviewed subject reports available at the Project Office.

I find no obvious statements in the reports affording a direct clue to the cause of failure.

Copies of individual pages of the reports have been obtained wherever they appear to supply factual basic information of value in preparation of the Panel's report.

by J
Clifford J. Cortright

UNITED STATES DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
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Robert B. Jansen, Executive Director

October 31, 1976

TO: Panel Members

FROM: Robert B. Jansen

SUBJECT: Meeting with R.R. Robison and His Staff on October 29, 1976

At 8:30 a.m. on October 29, 1976, C.J. Cortright and I met with Project Construction Engineer R.R. Robison and members of his staff (P.P. Aberle, Jan Ringel, Harry Parks, Lynn Isaacson, and Keith Rogers) in Mr. Robison's office at the Teton Dam. This meeting had been arranged at my request to afford opportunity to amplify and clarify various records, especially on project surveillance procedures, and observations in the period June 3-5, 1976. I had provided in advance a list of initial questions, as follows:

R.R. Robison

1. What were the procedures for collecting, plotting, analyzing, and reporting on surveillance data? e.g., observation wells.
2. What were the time intervals in each step of this process?
3. What reports did you make, oral or written, on the dam's condition in June?
4. Can you provide any more information on the Project Technical Record?

P.P. Aberle

1. In your statement, you said that you wrote down the time of collapse as 11:57 a.m. on June 5. Into what record was this written?

Harry Parks

1. In your statement, you said that you saw seepage at 7:50 a.m. on June 5, 1976 coming out of toe about 50 feet from the north abutment wall. You saw leakage 50 feet from the north abutment above 5200 elevation at about 10:30 a.m. Can you describe these more fully and reconcile them with the observations of others?

October 31, 1976

Harry Parks (Cont'd)

2. Who was the first to see distress in the dam, and what did he see?

Jan Ringel

1. Did you inspect the leak alone at 8:00 a.m. on June 5, 1976? Or did somebody from the survey crew accompany you? Did you inspect leaks other than what was reported to you by the surveyors?

General Question

1. What was the time of each significant observation on June 5, 1976, as you can reconstruct it now?

In response to the first three questions, Mr. Robison described the oral instructions that he had given and the specific assignments that he had made to watch for any adverse conditions at the dam. He also discussed his communications with USBR offices in Boise and Denver. He and Mr. Aberle described their personal efforts in making patrols and in reviewing inspectors' reports. I asked for, and Mr. Robison agreed to provide, a written description of all this activity so that it can be documented for the Panel's report.

Regarding the fourth question, Mr. Robison said that the construction report data already made available to the Panel constitutes the Project Office's contribution to date to the Project Technical Record. He also agreed to provide us with copies of his weekly construction reports.

Mr. Aberle said that his entry of the time of dam collapse was made on a desk pad in the Project Office.

Responding to our first question to him, Harry Parks commented that he and his survey party were at such a distance from the seepage area at 7:50+ a.m. that all they saw was an apparent ponding of water at the toe of the dam. They were really too far away to say that it was "coming out of the toe." As to the location of the leakage at El. 5200, we were referred to the photography of the leak, which Parks and the others suggest is the best record. Parks recalls that he and his crew members all saw the water on the 5041.5 berm at the same time. He and Ringel confirm that Parks went into Ringel's office alone to report the leakage.

October 31, 1976

Jan Ringel said that immediately after being alerted by Parks, he went alone to the toe of the dam to inspect the reported leakage. He did not see other active leakage at that time, but he did note that water had been flowing down the right groin during the night of June 4, or early in the morning of June 5. This was evidenced by a shallow eroded channel that had not been there at 9:00 p.m. on the preceding night. Although there was no water in this erosion channel, it was damp in places.

In reply to the general question about the times of significant observations on June 5, there appeared to be agreement in retrospect that the loud burst and the concurrent rapid enlargement of flow at approximately El. 5200 occurred at about 10:30 a.m. rather than 10:00 a.m., as had been previously reported. Mr. Robison said that when this happened he did not have his car and he ran all the way to the office to telephone the sheriffs. The call to the Sheriff of Fremont County was logged at 10:43 a.m.

The times earlier and later in the morning as reported in the sworn statements are regarded by the Project staff as more accurate. Robison, Aberle, and Ringel agree that Robison and Aberle arrived separately at the dam headquarters building, but nearly at the same time - about 8:50 a.m. Since they almost immediately proceeded down to see the leakage, they believe that the reported time of this observation was reasonably accurate. The 11:57 a.m. time of collapse is seen as precise ("within one-half minute or so") because it was marked by the power outage and consequent stopping of an electric clock at the Project Office, whose power came through the system in the canyon just below the dam.

Mr. Robison reports that the flow first observed at El. 5045 was from the talus, not formation rock. I asked him if the talus at the toe of the right canyon wall could have carried appreciable flow without such flow being apparent on the surface. He believes this to be entirely possible.

When asked to describe the hole at El. 5200, Mr. Robison said that from his vantage point looking directly into it, the hole was a tunnel about six feet in diameter running roughly perpendicular to the dam axis and extending back into the embankment for about 35 feet - as far as he could see.

Regarding the early stages of this leak, which developed a short distance south and about at the same elevation as the 2 cfs spring at El. 5200, the "wet spot" observed by Berry at about 8:30 a.m. is believed

October 31, 1976

by Robison, Aberle and Ringel to have been a damp part of the erosion channel which they have described. They say that they noticed that this channel had curved out a little ways from the abutment, as can be seen in photographs.

Robison and Aberle also question the wet spot location reported by David Burch as 100 feet from the abutment. They say that examination of the failure photographs alone shows this distance to be inaccurate.

Parks says that the seepage observed by Berry and Ferber between 7:00 a.m. and 8:00 a.m. was at the toe, on the 5041.5 berm, and not at E1. 5200.

Answering a question about the location of the whirlpool on June 5, Robison and Aberle agreed that it was probably not as close to the abutment as they previously had estimated. Robison said that it was directly opposite the sinkhole on the downstream face. He and Aberle would now estimate that it may have been as far out as 13+70 or 13+80. (This would put it more in line with a section through the grout-cap break, the sinkhole, and the leaks at E1. 5200 and E1. 5045.)

We also discussed whether seepage and leakage amounts were all estimated by visual observation or if some measuring devices were used. Robison and Aberle said that devices were planned but not yet installed. They both have considerable experience in flow measurement and estimating, and they believe that their judgments of the spring and leakage flows are reasonably accurate.

Robison also informed us that the first observations of springs flowing from formation rock downstream from the spillway stilling basin on June 3 (the only flows observed from formation rock in the pre-failure period June 3-5) resulted from his dispatching Ken Hoyt, Construction Inspector, to that area to conduct surveillance. Hoyt thus made his discovery of the springs, which were observed later by Robison and by Aberle.


I asked how much discharge was actually passed through the auxiliary outlet on June 5. Robison said that it got up to about 900 cfs, compared with the specified operational limit of 850 cfs. Aberle said that the gates were open about 69 percent. They had consulted with the designer to get guidance on how much to exceed the limit and were told that up to even 1,000 cfs would probably be all right. However, they decided not to push it that far, so as to avoid possible damaging vibration.

Page 5
Letter to Panel Members

October 31, 1976

Mr. Robison raised the subject of treatment of the rock joints and cracks in the key trench. He says that during construction he and his men did not observe openings in the rock such as are now exposed in the breach on the right abutment. He says that if they had, they would have done something about them. Robison also stated that slush grouting is uncommon in USBR practice and that the designers did not expect the construction forces to initiate such treatment.

Sincerely,



Robert B. Jansen

cc:
C.J. Cortright
L.B. James



United States Department of the Interior

BUREAU OF RECLAMATION
OFFICE OF DESIGN AND CONSTRUCTION
ENGINEERING AND RESEARCH CENTER
P.O. BOX 25007
BUILDING 67, DENVER FEDERAL CENTER
DENVER, COLORADO 80225

IN REPLY
REFER TO: 1300
510.-

P.B.J.
11/8/76 NOV 3 1976

Mr. Robert Jansen
P.O. Box 1643
Idaho Falls, ID 83401

Dear Mr. Jansen:

Please refer to Mr. Chadwick's two telegrams dated June 11 and 14, 1976, addressed to me. I understand from Mr. Guy of my staff that all the requested information has been supplied except item 9, "Record Cofferdam Seepage and Pumpage from Foundation Area". We do not have very much data on these activities.

Drawing No. 549-147-170 which is contained in exhibit 32 shows the arrangement of the dewatering effort in the cutoff trench and the Q on November 30, 1973. I have enclosed an additional copy of this drawing for your ready reference. The contractor submitted a rather voluminous claim for differing site conditions in the cutoff trench. Exhibit E of the contractor's April 11, 1975 claim contains the report "Changed Conditions - Cutoff Trench Excavation - Teton Dam" by Woodward, Thorfinnson & Associates. This report contains photographs of some seepage into the COT. Also of interest would be the USBR report entitled "Cutoff Trench Report". Both reports are available at the Teton Project Office.

Seepage through the cofferdam occurred in June of 1974. The following is quoted from the 1974 L-29 Monthly Construction Report:

"River Runoff - The flow of the Teton River has been above normal through the month due to an above normal snowpack in the drainage area and continued above normal temperatures. The water flow through the river outlet works tunnel peaked at approximately 4,100 c.f.s. on June 20 with the upstream pool elevation at 5074.2. A temporary bulkhead previously installed at the upstream portal of the auxiliary outlet works tunnel prevented any water flows through the auxiliary outlet works tunnel. Some seepage occurred through the upstream zone 4 and zone 2 embankment and around the zone 4 and left abutment contact. Pumps were installed by the contractor to prevent this seepage from flooding the zone 1 embankment."



The locations of these seepages are approximately as follows:

Through cofferdam at centerline station 23+40

Around left end of cofferdam at centerline station 24+00±

Project photographs P549-147-4397 NA and P549-147-4426 show the upstream and downstream views of the cofferdam in June of 1974. You may also wish to discuss the cofferdam and COT seepage locations and quantities with specific project personnel.

Very truly yours,



Acting for

H. G. Arthur

Director

Design and Construction

Enclosure



United States Department of the Interior

BUREAU OF RECLAMATION
ENGINEERING AND RESEARCH CENTER
P.O. BOX 25007
BUILDING 67, DENVER FEDERAL CENTER
DENVER, COLORADO 80225

IN REPLY
REFER TO: 1300
510.-

NOV 3 1976

*P.B.J.
11/8/76*
Mr. Robert B. Jansen
P.O. Box 1643
Idaho Falls, ID 83401

Dear Mr. Jansen:

The information requested by the Independent Panel to Review the Cause of Teton Dam Failure in your letter of October 12, 1976, is listed below in the same numbering format as in your request. Exhibits 24 and 39 were the sources for the information.

1. Grain size distribution

a. Borrow area investigations, zone 1, design. - During the design phase, two borrow areas were investigated for zone 1 material, "A" and "B". Only "A" was used in construction of the dam. In exhibit 24 are the grain size curves for each borrow sample and the standard properties summary (memorandum of May 27, 1970). Twenty-one samples 51B-1 through 51B-21 representative of area "A" were tested.

Clay ($< 0.005\text{mm}$) 14 percent, standard deviation = 5.7 percent
Silt (0.005 to 0.074mm) 66 percent, standard deviation = 13.7 percent

Sand (0.074mm to No. 4 size) 19 percent, standard deviation = 11.3 percent

Two samples contained gravel, 12 percent and 30 percent respectively

Test pit A2 (18-foot depth) samples were composited for shear, percolation, compaction, and compression testing
Grain sizes for the composite sample (51Bx46) were:

Clay - 10 percent, silt - 74 percent, and sand - 16 percent

b. Undisturbed samples from cutoff trench, zone 1. - Results of tests are discussed in a memorandum of October 6, 1975, a part of exhibit 24. The four samples were obtained between stations 19+00.7 and 20+51.3. Mean grain size distribution is:

Clay - 19 percent, silt - 74 percent, and sand - 7 percent.



c. Dynamic analysis testing, zone 1. - Samples, Denison type, taken from a hole about 100 feet upstream centerline at about station 20+00 with embankment at about elevation 5130. Two-foot long samples were taken at 10-foot intervals and were numbered 51B-48 through 51B-67(20), exhibit 24, "Testing for Dynamic Analysis".

Clay 23 percent, standard deviation = 6.5 percent
Silt 71 percent, standard deviation = 7.2 percent
Sand 6 percent, standard deviation = 3.4 percent
Inplace dry density, 105.1 pcf, standard deviation = 6.5 percent
Inplace water content, 21.2 percent, standard deviation = 2.8 percent
Proctor density - 102.9, moisture 18.2

2. Atterberg limits

a. Borrow area investigations, zone 1 design. - See discussion under 1.a. Of the 21 samples tested 14 were nonplastic. The remaining had a mean liquid limit of 27 and standard deviation of 1.6; plasticity index of 4, and standard deviation of 2.0. See exhibit 24, memorandum of May 27, 1970.

b. Undisturbed samples from cutoff trench, zone 1. - See 1.b. Two samples were nonplastic and the other two averaged 26 for LL and 4-1/2 for PI.

c. Dynamic analysis testing, zone 1. - See 1.c. None of the 20 samples were nonplastic. LL = 27, standard deviation = 1.2; PI = 5, standard deviation = 1.9.

3. Permeability tests

a. Borrow area investigations, zone 1, design. - See discussion under 1.a. Composite sample (x-46) had permeability of 0.32 feet per year, placed at a dry density of 98.8 pcf, 21.9 percent moisture, and a 100 psi load. See exhibit 24, memorandum of May 27, 1970.

b. Undisturbed samples from cutoff trench, zone 1. - See 1.b. The four samples were taken to perform horizontal permeability tests on the undisturbed samples. In place densities varied from 85.7 to 89.4 pcf. Lateral pressure of 25, 55, and 75 psi were applied to the outside of the membranes. Testing was performed in the high-pressure permeability test apparatus which uses pressurized permanent water to dissolve entrapped air and usually shows a higher permeability than a standard permeability test. Pressures in the permanent water were held at 5 psi below the applied lateral pressure. Hydraulic gradients were varied from 36 to 495. Coefficients of permeability at 55 psi varied from 3.2 to 13.0 feet per year.

c. Dynamic analysis testing, zone 1. -No test.

d. Construction control record tests, zone 1. - Data sheets for record tests are in exhibit 39 following the zone 1 construction control tests (form 7-1352). Number of tests = 147, mean = 0.47 feet per year, standard deviation = 0.67, the range of values was 0.02 to 3.57 feet per year. Fourteen tests had values over 1.0 feet per year. When these are excluded, values are: Mean = -0.29 feet per year, standard deviation = 0.24.

e. Construction control record tests, zone 2. - Data sheets for these large permeability tests are at the end of exhibit 39. The 14 test values ranged from 0.71 to 2979.6 with a mean of 137 feet per year and a standard deviation of 474.

4. Triaxial tests

a. Borrow area investigations, zone 1, design. - See discussion under 1.a. Composite sample (x-46) was tested in an unconsolidated undrained test. $\tan \phi' = 0.64$ and $c' = 11.3$ psi corrected for pore pressure. Nonlinear parameters for constitutive models were not obtained as standard practice at the time these tests were made. See exhibit 24, memorandum of May 27, 1970.

b. Undisturbed samples from cutoff trench, zone 1. - No test.

c. Dynamic analysis testing, zone 1. - See 1.c. Testing was done on remolded specimens made from composited material from the 20 samples. Tests were consolidated drained. $\tan \phi' = 0.70$, $c' = 0.2$ psi. The tests were run with back pressure to insure saturation. Nonlinear parameters for the hyperbolic stress-strain constitutive model were only calculated for the "raw" lab data. Values for use in finite element analysis should be obtained from corrected and/or smoothed lab stress-strain curves. Preliminary parameters are: $K = 470$, $n = 0.12$, $R_f = 0.78$, $G = 0.35$, $F = -.17$ (slope indicates decrease in "G" with increase in confining pressure), $d = 3.8$. See exhibit 24 for plots.

5. Compression tests

a. Borrow area investigations, zone 1, design. - See discussion under 1.a. Composite sample (x-46) was tested. Placement dry density of 96.0 pcf, moisture content of 19.9 percent, degree of saturation of 72.5 percent. Maximum consolidation was 10.3 percent under 600 psi load and 10.4 percent saturated under 600 psi load. See exhibit 24, memorandum of May 27, 1970.

b. Undisturbed samples from cutoff trench, zone 1. - No test.

c. Dynamic analysis testing, zone 1. - See l.c. Testing was done on remolded specimens made from composited material from the 20 samples. Placement conditions were: dry density = 100.1 pcf, moisture = 18.5 percent, degree of saturation = 75.2 percent. Maximum consolidation was 7.95 percent at 300 psi load and 8.05 percent saturated under 300 psi load. See exhibit 24 for plots.

Very truly yours,



Acting for

H. G. Arthur
Director
Design and Construction

November 3, 1976

Mr. Harold G. Arthur
Director of Design and Construction
U.S. Bureau of Reclamation
Bldg. 67, Denver Federal Center
Denver, Colorado 80225

Dear Mr. Arthur:

It has come to our Panel's attention that regularized inspections of Teton Dam construction were made by representatives of your Denver organization, with records of related observations in the form of trip reports. Because we have not found such records in the documents previously furnished our Panel by your office, we will appreciate receiving the file of these reports and any related responses as early as is reasonably possible.

Thanks in advance for your usual cooperation.

Very truly yours,

Wallace L. Chadwick
Chairman

cc:
Dennis Sachs, USBR, Washington, D.C.

**UNITED STATES DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE**

Wallace L. Chadwick, Chairman
Arthur Casagrande
Howard A. Coombs
Munson W. Dowd
E. Montford Fucik
R. Keith Higginson
Thomas M. Leps
Ralph B. Peck
H. Bolton Seed
Robert B. Jansen, Executive Director

November 3, 1976

Honorable Thomas S. Kleppe, Secretary
United States Department of the Interior
Interior Building
Washington, D.C. 20240

Honorable Cecil D. Andrus, Governor
State of Idaho
Capitol Building
Boise, Idaho 83720

Gentlemen:

The Independent Panel to Review Cause of Teton Dam Failure has the following progress to report.

Technical working sessions were conducted in Idaho Falls in the period November 1-3, 1976, with eight of the nine Panel members in attendance. On November 1, inspection was made of the drilling sites and the foundation areas uncovered by excavation on the right abutment. An examination of the lower right canyon wall was accomplished by boat.

Excavation of the embankment remnant on the right abutment has been completed and the rock surface has been sluiced. The grout cap is missing in the 30-foot interval between Stations 13+86 and 14+16. It is fully intact above this breach and extends continuously although severely eroded at several locations below this point to Station 14+85, beyond which it is missing at least to the present river level.

Mapping of joints and cracks in the right abutment rock continues. This has been facilitated by the sluicing which was completed during the last week in October.

In mid-October, ponding tests were performed on rock joints adjacent to the grout cap between Stations 12+73 and 13+40 on the right abutment. There was resultant flow under the cap at one point. In view of this, the Panel's planned program for drilling and water testing in that area has been expanded and is underway. Additional ponding tests are also being made.

November 3, 1976

Letter to Honorable Thomas S. Kleppe and Honorable Cecil D. Andrus

Initial results have been received from two of the four laboratories performing tests on soil specimens taken from the dam remnant on the right abutment. These data are being studied by the Panel.

During the technical working sessions of November 1-3, consideration was given to finite-element stress analyses prepared by the Dynamic Analysis Corporation and by the University of California at Berkeley.

To obtain field correlation with these analytical data, a hole is being bored into the left abutment embankment at Station 26+25 for a hydraulic fracturing test.

Investigative exploration near the right end of the dam is well advanced. Drilling and water testing of nine holes in the foundation under the spillway crest indicated that the grouted rock at that location was satisfactorily impermeable. The boring at Station 4+34 was terminated at a depth of 600 feet, having penetrated the sediments underlying the volcanics for an interval of about 80 feet. Sediment samples will be tested. A larger-diameter hole is being started nearby, at Station 4+29, with the objective of exploring the sediments to greater depth and for taking other specimens that can be tested. The angle holes on either side of the 600-foot hole have been completed and water tested. The results of all these borings are being analyzed by the Panel.

A model of the right side of the dam and its abutment is being fabricated by a firm in Salt Lake City. It is expected to be ready for the Panel's use by November 15. The model should facilitate visualization of principal features that relate to the mechanism of failure.

The contractor's work in the river channel continues, with the objective of lowering the water level in the pool just below the dam during the next six weeks. Once this is accomplished, the Panel will make inspection of the base of the right abutment.

The Panel has continued its analyses of the data collected and of the hypotheses of failure previously reported. Progress has been made in the assembling and drafting of material intended for use in the final report, which is still expected to be ready for submittal by December 31, 1976.

In all phases of its work, the Panel has been helped immeasurably by the consistent cooperation of you and your agencies. Your support has been essential and is appreciated.

Page 3

November 3, 1976

Letter to Honorable Thomas S. Kleppe and Honorable Cecil D. Andrus

The next technical working sessions of the Panel are scheduled for December 7-10, 1976.

Respectfully submitted,

Wallace L. Chadwick, Chairman
Independent Panel to Review Cause
of Teton Dam Failure



United States Department of the Interior

BUREAU OF RECLAMATION

TETON PROJECT OFFICE

P.O. BOX 88

NEWDALE, IDAHO 83436

November 12, 1976

IN REPLY
REFER TO:

*P.B.J.
11/15/76*

Mr. Robert Jansen
Executive Director
Independent Panel
539 9th Street
Idaho Falls, ID 83401

Dear Mr. Jansen:

Enclosed are project memorandums concerning processing of observation well data and the project's observation program for reservoir leakage, which you requested.

Sincerely yours,

Robert R. Robison
Project Construction Engineer

Enclosures

cc: Director of Design and Construction, Denver, Colorado
Attn: 1300
Regional Director, Boise, Idaho
Attn: 200
Teton Dam Repository, Washington, DC
Attn: 1600
Mr. Dennis Sachs, Deputy Assistant Secretary, Washington, DC



UNITED STATES GOVERNMENT

Memorandum

TO : Project Construction Engineer

DATE: Nov. 10, 1976

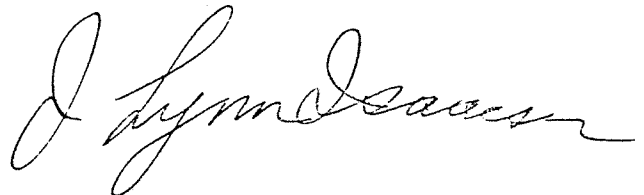
FROM : Contract Administration

SUBJECT: Compiling Data for Observation Wells

Subsequent to the receipt of a letter from the Regional Director, subject, monitoring ground level water wells, assignments were made to the various divisions for gathering and reporting this data. Contract Administration was assigned the responsibility of compiling the data received from field observations. Mr. Mike Brenchley was given the responsibility of developing a system of graphs and charts upon which the data from the observations could be compiled showing locations and number of wells observed. Charts were developed on which to compile the readings. Due to Mr. Brenchley resigning, the responsibility was given to Mr. Keith Rogers.

It was our intent that the readings should be compiled and forwarded to the Regional office and Denver office personnel interested in this data at least once each month. Subsequent to the water being stored in the reservoir, Mr. Rogers would receive the data and plot them on the charts as they were submitted in the field. I would review them periodically at least once each week to see if there was any significant changes in the water level shown. Approximately once each month we would assign a cutoff date that the charts would be brought up to date, the readings would be recorded and the data sent off to the various offices. The reservoir started filling so rapidly in the spring that recordings were read at more frequent intervals. Periodically we would meet with Mr. Robison and would look over the readings and discuss the changes noted. The last month before the dam failure, we noted a significant rise in water level shown on these readings. This was discussed with representatives of Director of Design and Construction, Denver. It was decided to make a special effort at this time to compile the data from the observation wells in order to forward them at a closer interval than in the past.

bcc: CA, Joseph Lynn Isaacson 11-10-76



Noted:


Project Construction Engineer



Buy U.S. Savings Bonds Regularly on the Payroll Savings Plan

UNITED STATES GOVERNMENT

Memorandum

TO : Project Construction Engineer

DATE: Nov. 9, 1976

FROM : Field Engineer

SUBJECT: Observation Program for Reservoir Leakage

At the time of closure of the river outlet works on October 3, 1975, an observation program for leakage of the reservoir was initiated. Mr. Al Stites, who was assigned to the inspection of the river outlet works area, was assigned to observe for leaks on the left abutment and powerhouse area. Mr. Frank Emrich, who was assigned to inspection of works in the R.O.W. gate chamber and shaft areas, was assigned to watch for leaks through the concrete in these areas. These leaks were measured and estimated as to the amount of water. However, they were very minimal at all times.

Mr. Gary Larson was assigned to watch for leaks in the A.O.W. shaft, spillway drains and the area to the right of the spillway. It was felt by the project forces that if any leakage would occur around or through the right abutment, it would initially show up in the gully located to the right of the spillway.

The area downstream of the spillway area was observed from across the river on a daily basis by the inspection forces and myself and leaks of any consequence could be detected by watching for water flows from the drain downstream of the spillway along the right abutment into the river. All inspectors were instructed to be aware for leakage and to report these leaks immediately.

During the month of May, the contractor (MK-K) cut a small hole into a water storage pond which was located high on the right abutment for the purpose of draining it. Water from this pond drained into the gully located to the right of the spillway. This water was detected almost immediately by the inspection forces and reported which shows the awareness of the program.

About ten days prior to June 4, I received a call from Mr. Duane Buckert, Project Manager for MK-K, stating that their Office Engineer, Vince Poxleitner, thought he saw a leak downstream of the spillway. This was checked out by the inspection forces and found to be negative.

After well no. 6 showed an exceedingly rapid increase of the water level, I made an inspection of the right abutment about 1200 to 1700 feet downstream of the dam and the gully in this area. This inspection was made on or about June 1, 1976, and no leaks were noted.



On Thursday, June 3, 1976, when the two leaks were found downstream of the spillway, I checked along the canyon downstream of these leaks an additional 500 feet and found no leaks.

On the morning of June 5, as I drove to the powerhouse area, I again visually checked the spillway drains and the gully to the right of the spillway and saw no leaks.

At least once a week I instructed the shift inspector to remind all inspectors to watch daily for possible leaks. These reminders were also made by myself several times during the weekly safety meeting held by the inspection forces each Monday morning.

As soon as the ice cleared from the reservoir area, the reservoir was inspected two to three times per week. The shoreline was patrolled near the damsite and potential land slides were noted and reported throughout the reservoir area.




NOTED: Robert R. Abicco
Project Construction Engineer

UNITED STATES DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Wallace L. Chadwick, Chairman
Arthur Casagrande
Howard A. Coombs
Munson W. Dowd
E. Montford Fucik
R. Keith Higginson
Thomas M. Leps
Ralph B. Peck
H. Bolton Seed
Robert B. Jansen, Executive Director

November 16, 1976

TO: R.B. Jansen, Executive Director

FROM:  L.B. James, Staff Geologist

SUBJECT: Exploration of the Rock Fissure Passing Through Station 4+34
at Teton Dam

The subject fissure, which was exposed during excavation of the keyway in the right abutment, was entered and examined shortly after its discovery by Mr. Steve Ellenberger, Construction Inspector for the U.S. Bureau of Reclamation. This fissure is shown in plan and cross-section on USBR Drawings Nos. 549-147-133 and 134, which are being reproduced for Chapter 5 of the Panel's report. There follows a summary of the interview I had with Mr. Ellenberger on November 11, 1976 during which he described his observations.

Mr. Ellenberger described that portion of the fissure that lies downstream of the keyway as averaging about 4 feet wide, except for some places where he could "outstretch his arms without touching either wall." At one place he noted a white popcorn-like lining on the walls, and at another a coating of red substance that rubbed off on his clothes. Blocks of rock with dimensions up to 4 to 5 feet on a side were encountered which he climbed over or crawled under as he made his way downward. Passage was finally blocked by a rock "the size of a pickup truck." He could look through a narrow opening into a room or passage that lay beyond, but could not see the end of the fissure. At this furthest point from the entrance, he judged that he had traveled laterally about 100 feet from the downstream wall of the keyway and that he was roughly 100 feet below keyway invert elevation. The walls remained fairly consistently 4 feet apart to this depth and showed no indication of converging below this point.

The segment of fissure lying upstream of the keyway was described as 1 to 1-1/2 feet wide and steep, but apparently flattening toward the north with depth. It was lined with stalactites and stalagmites, mostly about 3/8 inch in diameter. One stretch was coated with a mineral lining which displayed a "popcorn-like" appearance. Mr. Ellenberger edged his way laterally through this crack for about 100 feet where he squeezed through an opening into a chamber about 4 feet by 4 feet by 5 feet in dimensions.

Page 2

November 16, 1976

Memo to R.B. Jansen from L.B. James

SUBJECT: Exploration of the Rock Fissure Passing Through Station 4+34
at Teton Dam

He noted that the joint continued beyond this chamber but that its walls converged and turned. He was unable to explore further and could not determine whether the fissure reopened or pinched out entirely beyond this point.

Mr. Ellenberger noted that on cold days vapor could be seen emitting from the segment of the fissure that extended downstream of the keyway and this segment seemed warm and could be entered in winter without a jacket. Conversely, in the upstream segment he felt cold.

cc:

Panel Members
C.J. Cortright
F.B. Sherman

UNITED STATES DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Wallace L. Chadwick, Chairman
Arthur Casagrande
Howard A. Coombs
Munson W. Dowd
E. Montford Fucik
R. Keith Higginson
Thomas M. Leps
Ralph B. Peck
H. Bolton Seed
Robert B. Jansen, Executive Director

November 18, 1976

TO: Robert B. Jansen

FROM: Clifford J. Cortright

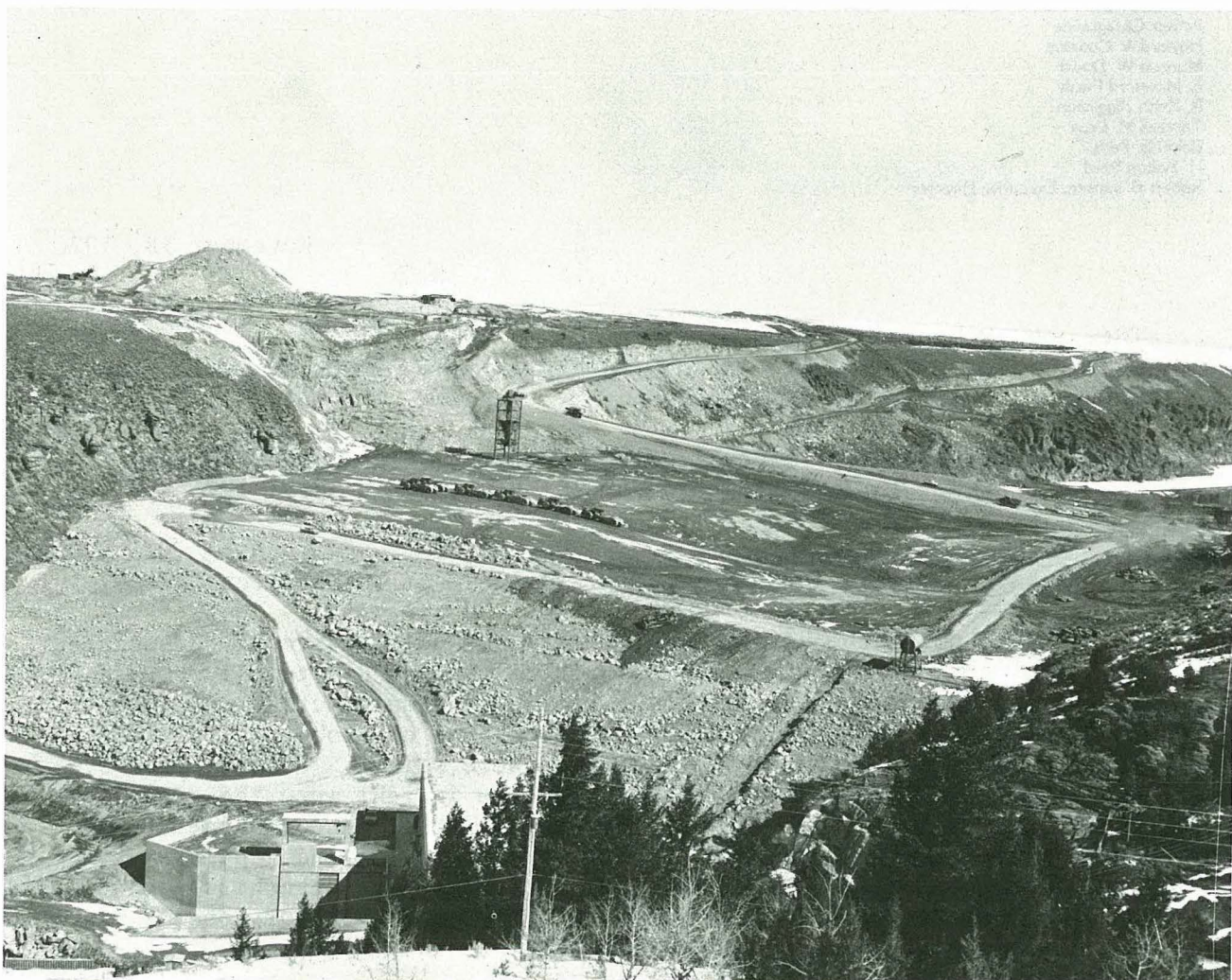
SUBJECT: Review of L-29 Construction Reports, Teton Basin Project,
Lower Teton Division, January 1975 through April 1976

I have reviewed subject reports available at the Project Office.

I find no statements in the reports that can be directly associated with the cause of failure. In fact, these reports are totally devoid of any statements commenting on the quality of construction, disputes, conformance with specifications, or discussions of problem situations encountered and the manner in which they were solved. Several photos, although taken from a distance, do give some insight into the general nature of the quality of the embankment foundation beneath Zones 1, 2, and 5 along the right abutment. Zone 1 in the key trench appears excavated into the rhyolite rock formations. Zone 1 elsewhere was stripped only to the rhyolite surface. The foundation for Zones 2 and 5 appears to be stripped of vegetation only.

Prints and captions of these photos are attached for later reference if needed.

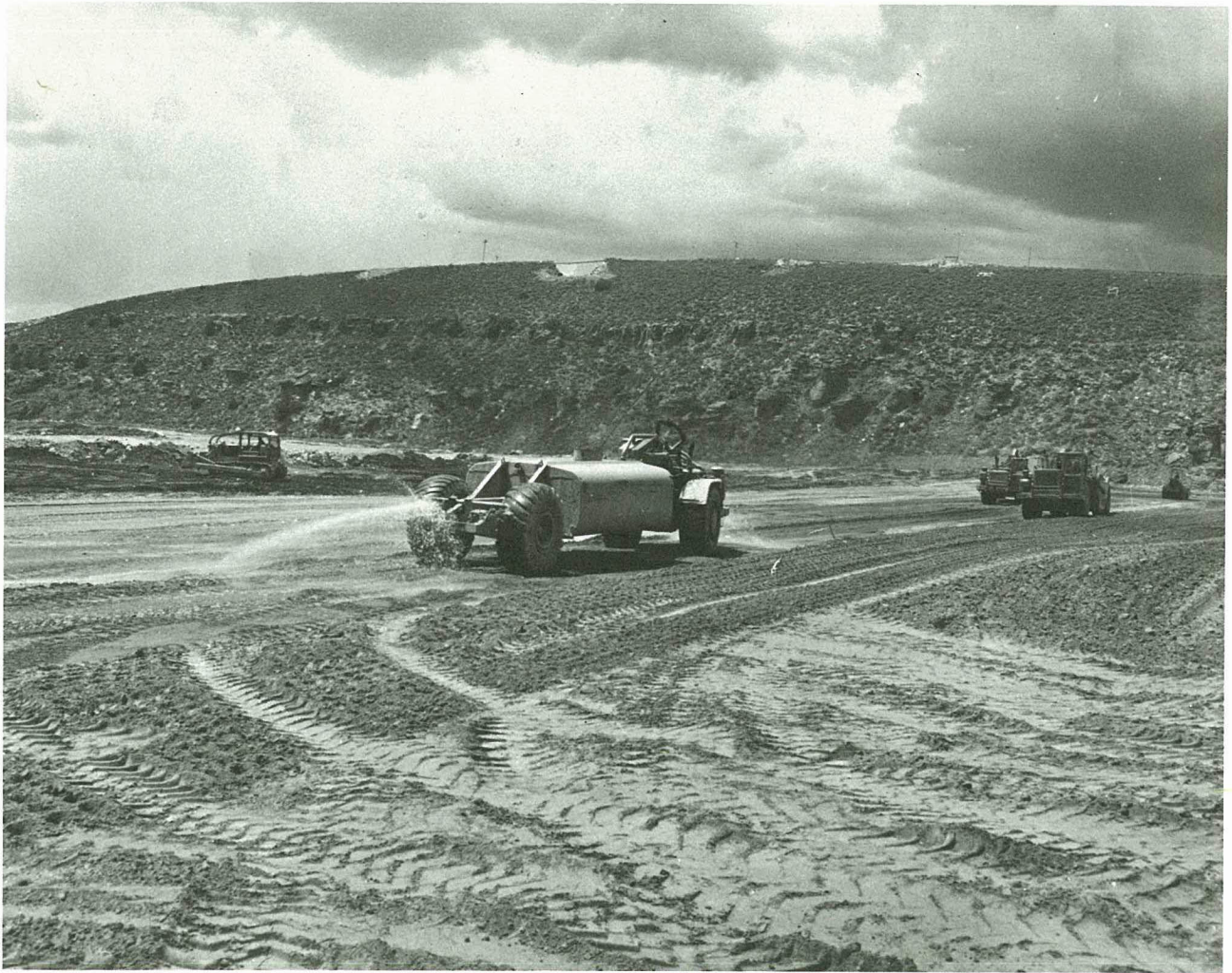
Encl.
Photos P549-147-5480
-5733
-5735
-5859
-5876
-5883



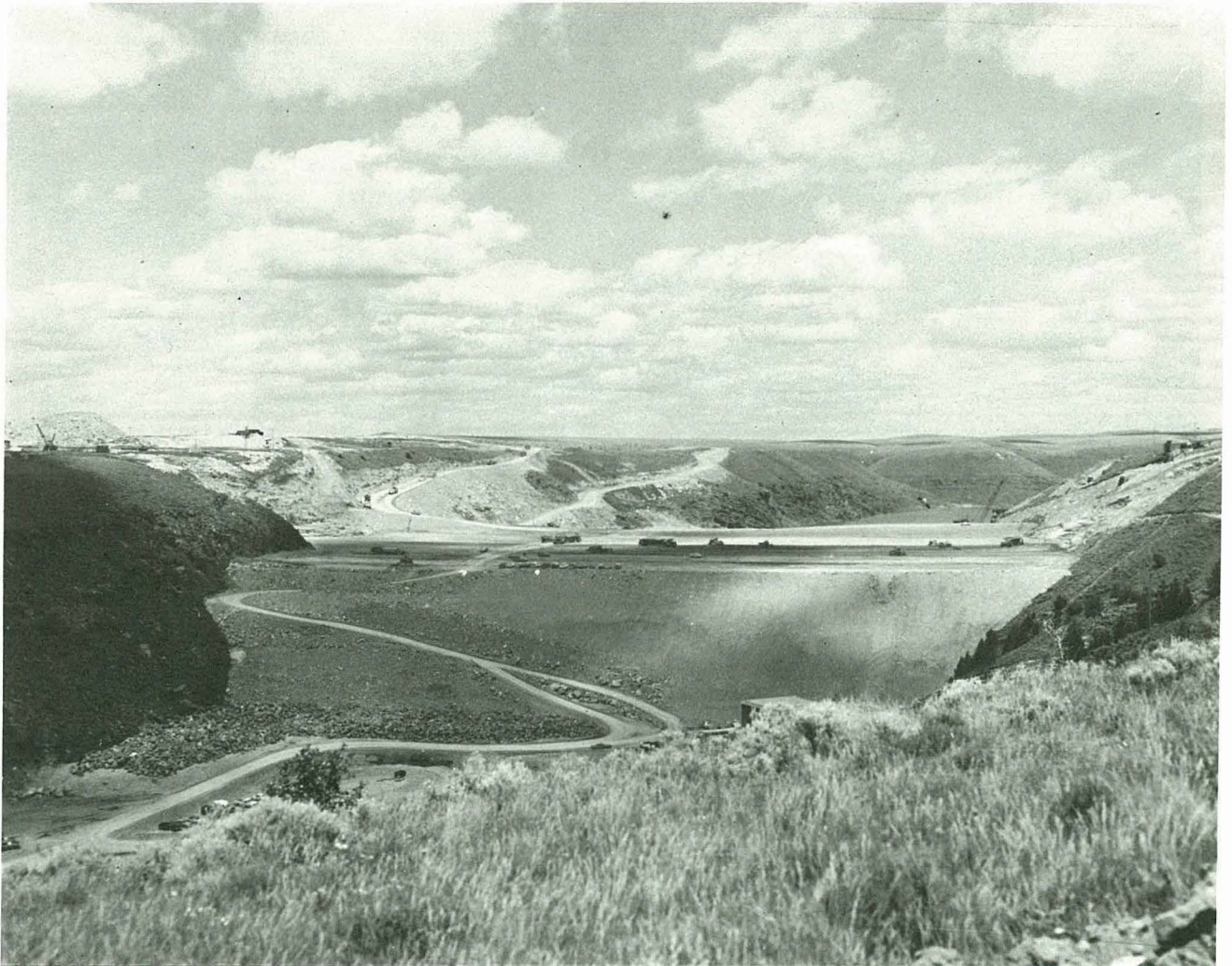
Project Photo P 549-147-5480 NA 4/1/75 Embankment at 5145



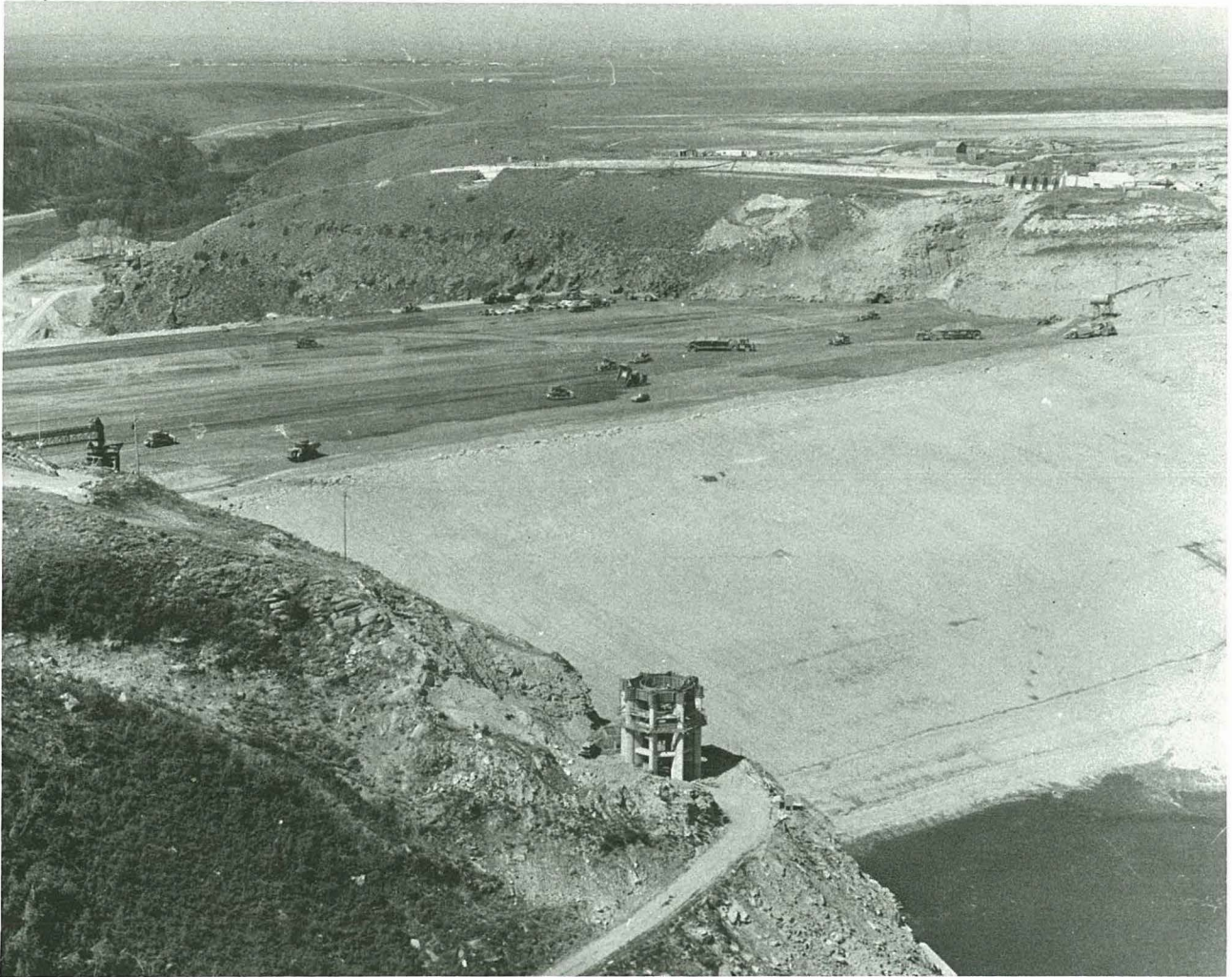
Project Photo P 549-147-5733 NA 5/27/75 Embankment at 5150



Project Photo P 549-147-5735 NA 5/27/75 Embankment at 5150



Project Photo P 549-147-5859 6/26/75 Embankment at 5170



Project Photo P 549-147-5876 NA 7/22/75 Embankment at 5185



Project Photo P 549-147-5883 NA 7/22/75 Embankment at 5185

UNITED STATES DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Wallace L. Chadwick, Chairman
Arthur Casagrande
Howard A. Coombs
Munson W. Dowd
E. Montford Fucik
R. Keith Higginson
Thomas M. Leps
Ralph B. Peck
H. Bolton Seed
Robert B. Jansen, Executive Director

November 22, 1976

TO: Robert B. Jansen
FROM: Clifford J. Cortright
SUBJECT: Teton Project Time Lapse Photo Record

The following Project Photographer's time lapse photo roll numbers were viewed by either Mr. Colè or myself:

9, 27, 29, 39, 97, 113, 115, 120, 125, 127, 133, 135,
148, 150, 155, 158, 162, 171, 175 and 179

The rolls viewed were selected on the basis that they might be informative with regard to foundation preparation and embankment placement at the right abutment in the general vicinity of the failure and that they might afford some insight into the cause of failure.

Unfortunately, the camera setup was usually at quite some distance from the main area of interest and no revealing detail of the abutment rock surface, quality of foundation cleanup, or manner of embankment placement against the abutment is visible.

A 16 mm reel exposed May 19, 1976 was also previewed. The canyon wall above the downstream right abutment groin and portions of the downstream embankment slope are visible in the background while the camera was recording a slope dressing operation by bulldozing. At the distance recorded, the degree of detail is not very great. No evidence is apparent of failure-related phenomena such as moisture, seepage, or leakage.

Clifford J. Cortright

UNITED STATES DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Wallace L. Chadwick, Chairman
Arthur Casagrande
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Thomas M. Leps
Ralph B. Peck
H. Rolton Seed
Robert B. Jansen, Executive Director

December 1, 1976

TO: Robert B. Jansen

FROM: Clifford J. Cortright

SUBJECT: Review of L-29 Construction Reports, Teton Basin Project,
Lower Teton Division, January 1974 through December 1974

I have reviewed subject reports available at the Project Office.

I find no statements in the reports that can be directly revealing
as to the cause of failure.



UNITED STATES DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Wallace L. Chadwick, Chairman
Arthur Casagrande
Howard A. Coombs
Munson W. Dowd
E. Montford Fucik
R. Keith Higginson
Thomas M. Leps
Ralph B. Peck
H. Bolton Seed
Robert B. Jansen, Executive Director

December 10, 1976

Honorable Thomas S. Kleppe, Secretary
United States Department of the Interior
Interior Building
Washington, D.C. 20240

Honorable Cecil D. Andrus, Governor
State of Idaho
Capitol Building
Boise, Idaho 83720

Gentlemen:

The Independent Panel to Review Cause of Teton Dam Failure has continued with its work under your charge and submits the following report of its progress.

The Panel held its final technical working sessions in Idaho Falls December 7 through 10, 1976, with all members participating. On December 7 the Panel examined the recently completed model of the right side of the dam and its abutment. This model depicts well the principal site features which are pertinent to the failure.

Site Work

Mapping of joints and cracks in the rock of the right abutment of the dam has been completed and the results are being analyzed.

Ponding and water-pressure testing of the foundation along the grout cap on the right abutment have been completed. The tests accomplished in that area, during the past month, showed some water flow through the jointed rock under low pressure immediately under the grout cap. This evidence of the rock condition supplements the ponding test results reported by the Panel on November 3, 1976.

The Panel's program of hydraulic fracturing tests in three borings in the dam embankment remnant on the left side of the canyon has been completed.

December 10, 1976

Letter to Honorable Thomas S. Kleppe and Honorable Cecil D. Andrus

Investigative exploration of the foundation near the right end of the dam to assess the possibility of differential settlement is approaching completion. The previously reported drilling of a large-diameter hole into the deep underlying sediments has been advanced to a depth of about 900 feet, having penetrated the sediments for an interval of about 400 feet. With the completion of this drill hole, the Panel's investigation at the damsite will be concluded.

Excavation of the river channel and related work under Bureau of Reclamation Contract No. DC-7232 with Gibbons and Reed Co., have progressed so that some lowering of the reservoir and the intermediate pool just downstream from the dam has been possible. This has exposed more of the foundation rock for inspection.

Laboratory Testing

Soils testing results have been received from all five laboratories providing support services to the Panel: Northern Testing Laboratories in Billings, Montana; Waterways Experiment Station of Corps of Engineers in Vicksburg, Mississippi; Bureau of Reclamation laboratory in Denver, the University of California laboratory at Davis, and Geo-Testing, Inc., of San Rafael, California. The results have been analyzed and are being used in developing the Panel's conclusions.

Analyses

Analyses of data collected from record examination and from field investigation and testing have been essentially completed. The results have been used to weigh the various hypotheses of failure reported earlier.

As investigation results have been obtained and correlated between data sources, the full record has been supplied to the Interior Review Group.

Final Report

The preparation of material for the Panel's final report has progressed on schedule, and the report will be completed by the contract date of December 31, 1976. This completion within schedule has been facilitated greatly by the continuing full support forthcoming from your offices.

Respectfully submitted,

Wallace L. Chadwick, Chairman
Independent Panel to Review Cause
of Teton Dam Failure

UNITED STATES DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

Wallace L. Chadwick, Chairman
Arthur Casagrande
Howard A. Coombs
Munson W. Dowd
E. Montford Fucik
R. Keith Higginson
Thomas M. Leps
Ralph B. Peck
H. Bolton Seed
Robert B. Jansen, Executive Director

December 11, 1976

Memorandum

To: Robert B. Jansen
Executive Director

From: Laurence B. James
Staff Geologist

Subject: Bore Hole Photography
DH652

In response to your request, I have reviewed the letter to you on this subject from Don C. Banks, Chief, Engineering Geology and Rock Mechanics Division, Waterways Experiment Station, U.S. Army Corps of Engineers dated 3 December 1976, including attachments. Attachments include a location map and profile, narrative borehole camera log, final log (computer print-out), joint pole diagram, joint rosette, joint classification, tabulation of joint orientations and tabulation of joint effective porosities.

The joint rosette shows that the predominant joint orientation is northwesterly similar to that observed in exposures in the walls of Teton Canyon. This is a particularly significant finding because while many joint characteristics may be determined by examination of drill core, it is not possible to determine the strike of joints from such inspection. Thus the survey indicates that a northwesterly joint orientation persists near the right end of the dam. It also confirms the existence of a large number of low-angle joints in the vicinity of the drill hole.

Laurence B. James

WIRE MESSAGE

NON-RECORD COPY

DEC 15 1976

ROUTE TO	INITIALS	DATE
1 1310		

PROGRAM

December 15, 1976

To: Teton Independent Panel, Idaho Falls, Idaho
 Attn: Cliff Cortwright

From: ^{ACTING} Project Construction Engineer, Newdale, Idaho

Subject: Survey Data for Dam Station 12+00 - Teton Basin Project, Idaho

ORIG. TO: 200, 1200-1310

Dam station 12+38.27 150.00 feet upstream of dam copy 50 was set by co-ordinates from Tri station #7 sighting Tri station "6001". Equipment used was a T-2 theodolite transit and a H. P. distance meter. Then station 12+38.27 78.00 feet upstream was set from station 12+38.27 150.00 feet upstream by using a 100 foot chain and a T-2 theodolite transit sighting Tri station #7 a TH43 transit and a 100 foot chain was then used on remainder of control points.

Setting instrument on station 12+38.27 78.00 feet upstream and back sighting station 12+38.27 150 feet upstream, a deflection angle of 90°00' was turned to the right and a distance of 8.54 feet was chained to set the bisect of the delta angle of P.I. 12+38.27 78.00 feet upstream. Then setting on this point a deflection angle of 12°30' was turned to the left, back sighting station 12+38.27 78.00 feet upstream and a distance of 56.81 feet was theoretically chained. This distance was later found to be 46.81 feet. Station 11+90.00, 78.00 feet upstream should have read 12+00.00 78.00 feet upstream. Then a deflection angle of 90°00' was turned left, back sighting station 12+38.27 78.00 feet upstream, bisect of the angle point. Points station 11+90.00 63.00 feet upstream and station 11+90.00 10.00 feet downstream were then chained out. These last two points were used for locating all other features including setting grout cap center line points and 15.00 feet upstream and 15.00 feet downstream. The stations that were used for the Independent Panel investigations would be affected on the block samples, trenches, and the stationing on the grout cap. Only the drill holes on the left side of the spillway would be affected. The points that were located on the 100 foot downstream and the 150 foot upstream line of dam axis for geology control, would also be affected. In essence, 10 feet should be added to all stations.

We have subsequently tied the point at dam station 12+00, 10 feet downstream to Tri station "2" and "GAMMA", in order to insure that stationing is correct.

We regret this incident and apologize for the great inconvenience caused.

Peter P. Abels
 5-1-1976

Regional Engineer, Boise, Idaho
 Director of Design and Construction, E & R Center
 Attn: 1310, Sammy Guy

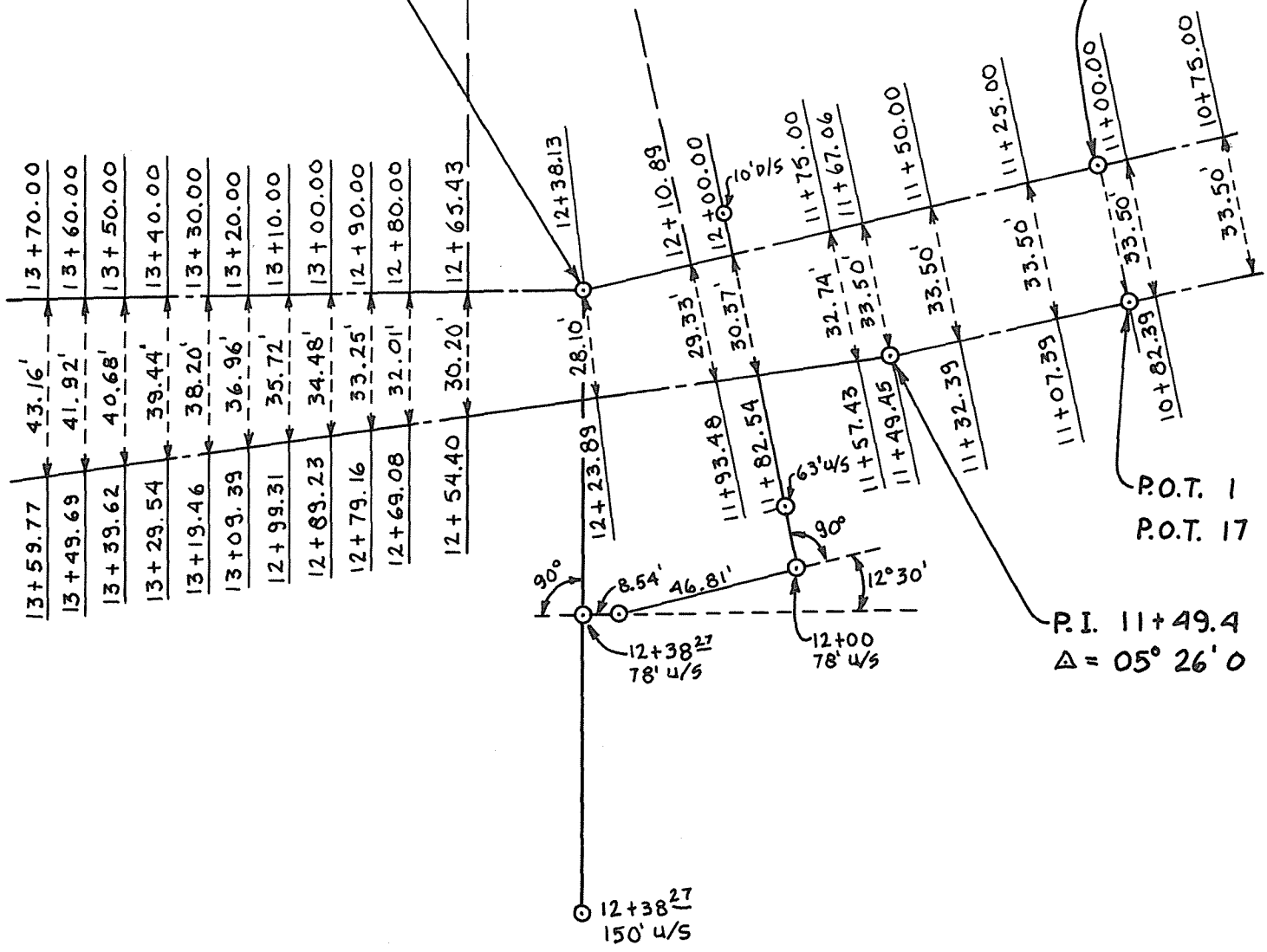
cc:

131

P.I. $12 + 38.27$
 $\Delta = 12^\circ 30'$ Rt.
 $R = 250'$
 $T = 27.38'$
 $L = 54.54'$

P.T. $12 + 65.4$
 P.C. $12 + 10.89$

P.O.T. $11 + 06.56$ & DAM
 P.O.T. $18 + 30.00$ & SPILLWAY



APPENDIX C

WITNESS ACCOUNTS OF FAILURE

INTERROGATORIES BY DIVISION OF INVESTIGATION SPECIAL AGENTS, OFFICE
OF AUDIT AND INVESTIGATION, OFFICE OF THE SECRETARY, ON BEHALF OF
THE TETON DAM PROJECT REVIEW COMMITTEE

June 25, 1976

Name:
Address:
Employer: USBR _____ Contractor _____
Title of Position:
How Long Employed:
All of Teton Project?

Where did you observe events of failure? (Exact location if possible)

Why were you there?

What time did you arrive at scene?

Who alerted you of possible problem? What Time?

How long did you stay?

Did you change locations?

State your description of what you saw from each site .
Did you see:

1. The lower water seepage? Where was it? What time noted? What color was the water? Estimated volume? How fast did it increase?
2. The upper water seepage? Where was it? What time noted? What color was the water? Estimated volume? How fast did it increase? When were you aware that the dam was in eminent danger? When did you realize that it would collapse?
3. The whirlpool upstream? Was there more than one? Estimate its circumference when first seen. Describe its activity - enlarging? moving? Did you realize the significance? Where was it? What time observed? How long was it visible?

Any tremors earlier?

Check inspection route on previous shifts.

BUREAU OF RECLAMATION WITNESS STATEMENTS TO TETON DAM FAILURE

Peter P. Aberle, Field Engineer Fifth West South Rexburg, Idaho 83440	356-7631
Andrew L. Anderson, Electrical Engineer 53 S. Third E. Rexburg, Idaho 83440	356-3924
Wilburn H. Andrew, Mechanical Engineer Virginia H. Perkins Dormitory #32, Ricks College, Rm 59 Rexburg, Idaho 83440	356-2579
Richard Berry, Surveyor 275 So. First East Rexburg, Idaho 83440	
Stephen Elenberger, Construction Inspector Victor, Idaho	
Charles L. Entwisle, Inspector 440 N. 7th W. St. Anthony, Idaho 83445	624-3012
Clifford W. Felkins, Surveyor 430 N. 3 W. Rigby, Idaho 83442	745-7922
Myra Ferber, Surveyor Box 124 St. Anthony, Idaho 83445	624-4106
Alvin J. Heintz, Inspector 151 N. 2nd E. St. Anthony, Idaho 83445	624-7982
Kenneth C. Hoyt, Inspector Rt. 1, Box 202-12 St. Anthony, Idaho 83445	624-3228
Harry A. Parks, Surveyor (Chief of Crew) Kit Circle Trl. Ct. #5 St. Anthony, Idaho 83445	624-4273
Jan R. Ringel, Engineer (Supr.) 520 Targhee St. St. Anthony, Idaho 83445	624-3873
Robert R. Robison, Proj. Constr. Engineer 581 Taurus Drive Rexburg, Idaho 83440	356-7218
Alfred D. Stites, Inspector P.O. Box 155 St. Anthony, Idaho 83445	624-3885

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Peter P. Aberle, Rt. 1, Box 247C, Rexburg, Idaho
_____ , being duly

sworn make the following voluntary statement to Vincent L. Duran,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I have been employed as Field Engineer, GS-13, Teton Dam Project, Bureau
of Reclamation, Newdale, Idaho, since March 1976 and have a total of 15
years service with the Bureau of Reclamation. From October 1972 to
August 1974, I served as Chief of Grouting, and from August 1974 to
March 1976, I served as Chief Inspector and Chief of Grouting.

Starting on about June 3, 1976, I observed small springs in the right
abutment downstream from the toe of the dam. These springs were clear
water and did not appear to be serious in nature, but warranted monitoring
by visual observation as frequently as routine inspections of the entire
operation at the dam.

Between 8:20 and 8:30 a.m. on Saturday, June 5, 1976, I received a call from
Jan Ringel at my home and he told me of a leak at the right abutment toe
area of the dam. Ringel estimated the leak to be about 20 to 30 sec. ft.
I asked my wife to call Mr. Robison and I left for the dam. I drove directly
to the powerhouse area and briefly inspected the leak from the left side
abutment area. I noted that the water was muddy and estimated the volume
to be the same as that given me by Ringel. I do not believe the water was
running long because there was very little erosion in the gravel at the toe
of the dam.

At approximately 9:00 a.m. I went to the project office and met Mr. Robison
and Jan Ringel. Mr. Robison and I walked out on the top of the dam and
walked down the downstream face of the dam to a leak located at the 5200
feet elevation, near the right abutment wall. The water in this leak was
running at about 2 sec. ft. and was only very slightly turbid. The leak
appeared to be coming from the abutment rock. The leak at the toe of the
dam was running turbid water from the abutment rock at an estimated volume
of 40 to 50 sec. ft.

At about 9:30 a.m. Mr. Robison and I went to the office area and discussed the matter with Mr. Buckert and asked him to mobilize two dozers and a front end loader in order to channel water away from the powerhouse area and to riprap a channel to the tailrace area.

At about 10:00 a.m. I was coming out of Buckert's office, when I heard a loud burst of water. I ran down to the visitor's view point and saw that a leak had occurred at the 5200 ft. elevation about 15 feet from the right abutment wall. The water was muddy and flowing at a volume of about 5 sec. ft. I went back to Buckert's office and asked him to mobilize all possible equipment and we discussed what might be needed to open the river outlet tunnel. At about 10:30 to 10:45 a.m. two dozers went down the face of the downstream side to move rock into the leaking area at the 5200 elevation. The reason for the delay in the dozer operation was the fact that men had to be called from home since Saturday was not a working day for most employees.

At 11:00 a.m. Alfred Stites and I saw a whirlpool begin to form at station 1300 (about 150 feet from the spillway) and about 10 to 15 feet into the water from the edge of the riprap. We were standing on the top of the dam toward the north end and the whirlpool was forming in the upstream reservoir. As we watched this two dozers were coming across the top of the dam from the left and I instructed them to push riprap and zone 2 material toward and into the whirlpool. I saw only one whirlpool and as I watched it, it gradually grew larger. The whirlpool was approximately 0.5 feet in diameter at the beginning and was located in an area consisting of clear water. I noticed that the water along the right bank was turbid about 150 feet upstream from the dam and about 15 to 20 feet out from the edge of the abutment. This turbid water was first noted at 9:30 a.m. by me before the whirlpool started and was thought to be turbid due to wave action. I wish to point this out due to the possibility of abutment failure. At about 11:15 a.m. the two dozers working on the downstream face of the dam at 5200 elevation began having problems. One of the dozers was falling into the opening and the second was trying to pull the other dozer out. At approximately 11:30 a.m. both dozers were lost into the hole caused by the flow of water.

At about 11:40 a.m. I left the top of the dam heading for the office and I noticed that at 11:45 a.m. the two dozers working on the upstream side of the dam began leaving the work area. I was standing in front of the project office which is located beyond the south end of the dam and saw the top of the dam collapse into the rushing water. I looked at my watch and it was 11:57 a.m. and I wrote this time down.

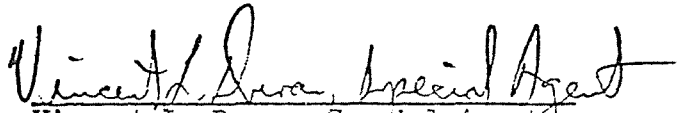
I was of the opinion that the collapse of the dam was definitely going to happen shortly after 11:30 when the two dozers were lost.

A number of the Bureau of Reclamation employees were involved in controlling crowds of onlookers on both sides of the dam, from the time of the collapse until late in the afternoon. I cannot at this time estimate the number of onlookers.

I have carefully read the foregoing statement consisting of two and one-quarter pages and declare it to be true and correct.


Peter P. Aberle
Peter P. Aberle

Subscribed and sworn to before
me this 23rd day of June 1976


Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U. S. Department of the Interior

State of Idaho

ss

County of Bonneville

I, Andrew L. Anderson, 53 S. Third E., Rexburg, Idaho, being duly sworn make the following voluntary statement to Vincent L. Duran, who has identified himself to me as a Special Agent of the U.S. Department of the Interior. No threats or promises have been made to obtain this statement.

I am employed as Electrical Engineer, GS-12, Teton Dam Project, Bureau of Reclamation, Newdale, Idaho. I have worked there since November 1974. Previous Bureau of Reclamation experience of 12 years.

At Teton I was working on all electrical work, primarily in power house. On Saturday, June 5, 1976, I was at home at 11:00 a.m. I received a call from Peter Aberle. He told me dam was leaking and wanted me to come out to get river outlet gates open to release water. I arrived at dam between 11:15 and 11:20 a.m. I got pickup at office and went to outlet shaft house -- left side and upstream at dam. This took about five minutes. I noticed heavy equipment on far side of dam on top and Robison's vehicle. Did not notice specifically what they were doing on the whirlpool. I went into shaft house to check power to gates. There was power, disconnect switch was off and locked. This was normal condition because of work in the outlets. The auxillary shaft on right side of dam was open and water flowing.

After determining we had power I went over to Robison on top of dam - right side. At this time, no later than 11:30 a.m., I saw leak inside right abutment about 1/3rd way down. Also saw one bulldozer at the opening stuck at top of opening. Asked Robison what he wanted me to do. He instructed me to go to power house area and get the gates operational at the penstocks and check for workers in the outlet. On way down I met Wilburn Andrew, he told me power house secure and he was going to notify fishermen down stream. I continued down and met Dick Cuffe and Hopkins. They were leaving and told me to leave also. I checked gates, everyone leaving and bulldozers were falling in hole. I went up top, saw huge hunks of dam falling. Within two or three minutes Aberle came in and said dam breached. Time was about 11:57 a.m.

Seepage water was muddy and the increase was very rapid but cannot estimate the volume. I was of the opinion there was eminent danger when I talked to Robison at about 11:30 a.m. Within six hours most of water gone from the reservoir.

I have carefully read the foregoing statement, consisting of 2 pages and declare it to be true and correct.

/s/ A. L. Anderson
Andrew L. Anderson

Subscribed and sworn to before me on
this 18th day of June, 1976.

/s/ Vincent L. Duran, Special Agent

Vincent L. Duran, Special Agent
U.S. Department of the Interior

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Wilburn H. Andrew, 257 N. 2nd W., Rexburg, Idaho
_____, being duly

sworn make the following voluntary statement to Vincent L. Duran,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I am employed as a mechanical engineer; GS-12, Teton Dam Project, Bureau
of Reclamation, Newdale, Idaho and have held this position since January 20,
1975. I have been employed by the Bureau of Reclamation since August 1972.

At 9:00 a.m. on Friday, June 4, 1976, Stites and I walked around the
right abutment (north side) area at the toe of the dam for the purpose
of looking for leaks. We were doing this because one or two spring
leaks had developed further down the stream in the abutment wall about
the day before. We did not see any leaks around the toe of the dam
or any where on the downstream face of the dam.

On Saturday morning, June 5, 1976, Peter Aberle telephoned me at home
and asked me to come to the dam immediately because there was an emergency.
I arrived at the dam sometime between 10:15 and 10:30 a.m. and reported
to Mr. Robison at the Project office. Robison told me there were some
bad leaks in the dam and asked me to check all the valves in the powerhouse
to be certain they were closed. While driving from the office to the
powerhouse, I observed the upper seepage in the downstream face of the
dam at about 5200 elevation and a short distance from the right (north)
abutment wall. There was a sizeable flow of water which was muddy, but
I cannot estimate the volume.

I went into the powerhouse and checked the various butterfly valves
and assured myself they were all closed. This was in preparation
to the possibility of opening the river outlet tunnel. I went outside
the powerhouse with a camera and took a picture of the upper leakage
at the 5200 elevation near the right abutment. I did not notice that
there was a sizeable increase in the volume of water flowing through
the dam opening. After taking the picture, I ran into Dick Cuffe
and Lloyd Hopkins at the power house and they told me there were four
fishermen about one-quarter mile or more downstream of the dam. I
drove downstream to try to locate the fishermen and found the fishermen
near a residence and out of sight of the dam. I yelled to the fishermen
to leave the area immediately and they advised me that they would do so.
The fishermen were in a rubber raft when I located them on the river.

*with
toward*

I drove back upstream ~~to~~ the powerhouse and saw a crane evacuating the area and Barry Roberts advised me that I should leave the powerhouse area and go to higher ground. I would estimate the time to be about 11:40 a.m. I drove up to the south rim road and observed the top of the dam collapse. I would estimate the collapse of the dam to have been at about 11:45 a.m. but this is not an exact time. I was not checking the time in the face of all the turmoil.

The river outlet tunnel was never opened because it had to be evacuated before it was completely cleared of equipment.

I had no full realization that the dam was actually going to collapse until I saw the top fall. I never saw the activity at the top of the dam, including the whirlpool, because all of my activities were in the powerhouse and the downstream area.

I remained at the dam site until about 8:30 p.m. Much of this time was spent working on crowd control, but I cannot estimate the number of people who came to the dam. At about 7:00 or 8:00 p.m. I observed several springlike flows of water on the face of the rock wall upstream of the grout curtain on the north or right side. I made this observation from the south side of the dam. I noticed one flow was approximately 25 feet upstream from the grout curtain and about 100 to 125 feet down from what had been the top of the dam. I would estimate this flow at about 200 gallons per minute. There were no observable leaks or flows of water from the rock face within 200 feet downstream of the grout curtain.

I have carefully read the foregoing statement consisting of one and three-quarter pages and declare it to be true and correct.

Wilburn H. Andrew

Wilburn H. Andrew

Subscribed and sworn to before
me this 22nd day of June 1976

Vincent L. Duran, Special Agent

Vincent L. Duran, Special Agent
U. S. Department of the Interior

STATE OF IDAHO

SS

COUNTY OF MADISON

I, Dick R. Berry, 269 S. First E., Rexburg, Idaho

_____, being duly sworn make the following voluntary statement to Vincent L. Duran, who has identified himself to me as a Special Agent of the U.S. Department of the Interior. No threats or promises have been made to obtain this statement.

I am employed as Survey Technician, GS-5, Teton Dam Project, Bureau of Reclamation, Newdale, Idaho and have held this position since September 1975. I had previously been employed with the Soil Conservation Service, U. S. Department of Agriculture since May 1974.

On June 4, 1976 I recall seeing seepage near the right abutment wall below the toe of the dam. The water was clear and not really running--- just settlement. There were no leakages or seeps at the dam.

On Saturday, June 5, 1976, I arrived at the Project Office a little before 7:00 a.m. Harry Park's Volkswagen was in the parking lot. Clifford Felkins and I arrived in a white Chevrolet pickup truck. These were the only two vehicles in the parking lot at the time. I had no watch with me at work on that date.

At 7:20 a.m. on June 5, I left the Project Office and drove down the upper south rim road to check three site rods on the south rim across from the spillway. I was checking the site rods for the purpose of going to the spillway and doing survey work on its walls. While checking the site rods I saw a small seepage on the north side downstream face of the dam, right at the abutment and dam joint. This was approximately one-third of the way up the dam, but not as high as the change in slope. There was slight erosion, slow flow of water, but I do not recall it being muddy. The seepage appeared to be almost new. I returned to the office and Harry Parks, who was in the crew, reported the seepage to Jan Ringel about 7:35 a.m. We then drove across the dam and parked just south of the spillway. I checked the water level in the reservoir on the upstream side, but do not recall the level. The water was very calm and there was no discoloration and no evidence of a whirlpool.

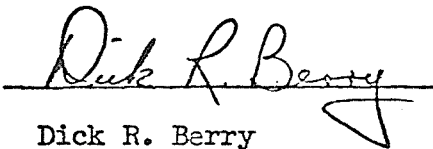
We then started work on the spillway at about 8:30 a.m. Just before we went into the spillway I saw a wet area at the end of the sage area just off the abutment on the downstream face of the dam. I do not recall this being running water, just a wet area. We went into the spillway and surveyed the left wall. My view of the leak was blocked. At about 10:15 a.m. I heard noise from a lot of equipment. At 10:30 a.m. I went to the top of the spillway to start the right wall and noted that the upper hole had expanded to 35 feet in diameter with a flow of muddy water 3 to 4 feet wide and six inches deep. There was a dozer trying to fill in the hole.

At 11:00 a.m. I was back at the top of the spillway and saw the hole had expanded toward the top of the dam and had elongated to 100 feet and took more of the face of the dam. There was a lot of activity on the dam. I recall saying something about sounding like a waterfall sometime about 11:15 a.m. ~~at~~ ^{at} 11:30 a.m. We continued to survey until about 11:40 a.m. at which time Aberle called us out of the spillway because of danger. I arrived at the top of the spillway at about 11:45 a.m. and saw that there was a little bridge of dam material across the top. I thought at this time that the dam was gone. At about 12:00 noon I saw the top of the dam break through.

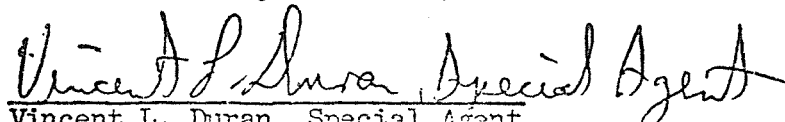
At 11:45 a.m. I saw ^{ONE} ~~two~~ ^{RRB} dozers leaving the upstream face just before the top collapsed. I also believe there was a pickup truck going across the top. I evacuated to the north side. I observed the dam until about 12:15 p.m. and then head for St. Anthony, Idaho. I was not involved in crowd control.

I was not aware of any earthquake or tremors.

I have carefully read the foregoing statement, consisting of one and three-quarter pages and declare it to be true and correct.


Dick R. Berry

Subscribed and sworn to before
me this 22nd day of June 1976.


Vincent L. Duran, Special Agent
U. S. Department of the Interior

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Charles L. Entwisle, 440 N. Seventh W., ~~St.~~

St. Anthony, Idaho, being duly

sworn make the following voluntary statement to Vincent L. Duran,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to

obtain this statement.

For three and one-half years I have been employed as Construction Inspector, GS-9, Teton Dam Project, Bureau of Reclamation, Newdale, Idaho. I have been employed by the Bureau since May 7, 1962.

On Saturday, June 5, 1976, at about 9:30 a.m., I received a telephone call from Jan Ringel who asked me to come to the dam because there was an emergency. I arrived at the dam at about 10:30 a.m. Upon my arrival at the office I answered the telephone and was told by Wilburn Andrew that the butterfly valves at the power house were secured. I then proceeded to the top of the dam to relay the information to Robert Robison. As I approached the top of the dam I saw a washout area about 40 feet square in the downstream face of the dam near the north or right abutment and about one-third the way up the face. There were two dozers pushing material into the openings. The water was muddy, but I cannot estimate the volume.

I proceeded out across the top of the dam to see Robison. As I approached the north or right side a small whirlpool about 10 feet from the upstream face of the dam just off the right abutment was forming in the reservoir. The time of this was about 10:50 a.m. The whirlpool was about two feet in diameter and the ~~water~~ ^{vortex} was about six inches. It appeared to be stationary, but grew in size as I watched it. Two dozers were activated and began pushing rip rap into the whirlpool.

The downstream leakage and the whirlpool grew in size and the two dozers working on the downstream side were washed away by the water. I would estimate the time of this to be about 11:30 a.m., but this is strictly a guess. Shortly after this the downstream face washed out to within 10 feet from the top of the dam. At this point I felt the dam was going to wash away.

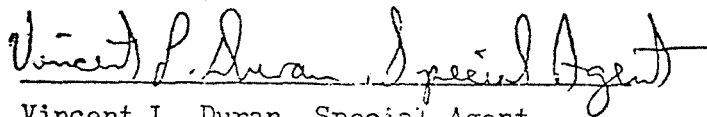
The two dozers working on the whirlpool were told to evacuate and as they moved across the top of the dam to the south side the top of the dam collapsed. To my recollection the collapse occurred at about 12 noon.

I immediately after the collapse drove down the north side of the river warning people of the collapse and returned to the project office about 12:30 p.m. Throughout the afternoon we were working on safety precautions for on-lookers coming by, but I cannot estimate how many people were there.

I have carefully read the foregoing statement, consisting of 2 pages, and declare it to be true and correct.


Charles L. Entwisle

Subscribed and sworn to before
me on this 21st day of June 1976.



Vincent L. Duran, Special Agent
U.S. Department of the Interior

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Clifford Felkins, 430 N. 3rd W., Rigby, Idaho
_____, being duly

sworn make the following voluntary statement to Betty J. Foyes,
her

who has identified ~~him~~ ^{XXX} herself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I am employed as a Surveying Aid, GS-3, Teton Dam Project, Bureau of Reclamation, Newdale, Idaho and have held this position since May 3, 1976. I have had no other Federal Service except with the U. S. Navy.

On Friday, June 4, I noticed for the first time some wetness in the waste area near the right abutment wall of the dam. There was no water flow, just wetness.

On Saturday, June 5, 1976, I arrived at the dam at about 7:00 a.m. driving a little Chevrolet pickup truck and parked it in the parking lot at the Project Office. Harry Parks had arrived a little before me and had parked his Volkswagen in the parking lot. My pickup truck is white. These were the only two vehicles in the parking lot.

On June 5, the first thing that I saw connected with the later events of the dam collapse was a water flow coming from the toe of the dam. It was a steady flow of water, but I cannot estimate the volume. To the best of my recollection the water flow was clear. I noticed this flow while I was standing across the river on the canyon wall from the spillway. I was with Harry Parks and we came to the survey office, which is a building immediately behind the Project Office, and reported the leak to Jan Ringel. This was about 8:30 a.m. We then went back to the spillway, which is located on the north or right side of the dam in order to check the alignment of the walls on the spillway. During part of the time when we were working on the alignment of the spillway the leak was out of our view. We started our work on the alignment from the top of the spillway on the left hand side. This was approximately 9:15 a.m. We worked our way halfway down the spillway on the left hand side. When we were working the lower half the leak was out of our view. When we completed our work on the left side of the spillway, we came up to the top of the spillway by walking along the left side, but outside the spillway. While we were making our way to the top of the dam, at about 10:15 a.m., we observed a hole on the right abutment (north side) about one-third of the way up the dam, just below the change in elevation.

I would estimate the hole was about 10 foot in diameter at this time. A cat was beginning to move riprap into the hole. I was personally concerned about the trouble at the dam, but nevertheless continued on to the top of the spillway to begin work on the right side alignment. When we reached the top of the dam I observed another cat moving into the dam to begin work, but I did not see where it went.

We began our work on the right side of the spillway, working down. We could see the construction supervisors from Morrison-Knudsen and Bureau supervisors directing operations and making observations of the dam. We tried to continue our work, but naturally were distracted by the activity and kept watching the supervisors running around. We were never at a point where we observed the whirlpool which later formed on the reservoir side of the dam. We did see two more cats move onto the dam and begin pushing riprap into the reservoir side of the dam. I would estimate this was around 11:00 a.m.

I do not recall the time when we first observed the upper water seepage. We were standing near the top of the dam in the spillway and observed the second hole beginning to form just as we were coming out of the spillway. We were leaving the spillway on the instruction of Pete Aberle who told us to get out. I did not actually see any water come out of the upper hole because the dam caved in and the two holes became one large one. The water that came through was muddy. I cannot estimate the volume, but it was a lot of gallons. The volume increased very rapidly.

I noticed the two cats on the top of the dam just before the dam collapsed. I recall that there was also a pickup truck on the top of the dam. When the dam collapsed between 11:45 and 12:00 noon, the cats and the pickup truck had just left the top of the dam, proceeding to the left side (south).

I never really believed that the dam was going to fail. When they told us to get out of the spillway I knew the dam was in imminent danger. I could not really believe the dam had collapsed even after the event had occurred.

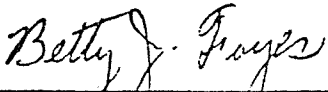
Just before we came out of the spillway, right before 11:30 I heard what appeared to be sound like water rushing and there was a slight vibration. I would estimate that this occurred when the dam was actually crumbling.

After the dam collapsed we collected our equipment and got into a Jeep and drove immediately to St. Anthony, Idaho, stopping along the way at a farm house in order to call our families in St. Anthony and Rigby. I would estimate we left the dam shortly after 12:00 noon. I noticed before we left that there were a lot of members of the public observing the dam from the visitor's observation platform on the other side. Since we were across the river we did not assist in crowd control.

I have carefully read the foregoing statement consisting of two and a fraction pages and declare it to be true and correct.


Clifford Felkins

Subscribed and sworn to before me
this 22nd day of June 1976.


Betty J. Foyes, Special Agent
U. S. Department of the Interior

Again while we were surveying the spillway I was unable to observe the leak. At about 11:45 a.m. Peter Aberle called us out of the spillway and we started toward the top. Just before Aberle called us I heard a loud noise, which sounded like water forcing through the leak area.

At about 11:50 a.m. I arrived at the top of the spillway and saw two ~~beard~~ ^{beard} ~~several persons who had been working in that area, start to~~ ^{and} ~~and~~ ^{and} ~~across the top of the dam from the north side toward the south~~ ^{side} ~~side~~ ^{side} Shortly thereafter the remaining portion of the top of the dam on the north side collapsed, I would estimate the time to have been 11:57 a.m. I went home at about 12:15 a.m.

I have carefully read the foregoing statement consisting of one and one-third pages and declare it to be true and correct.

Myra A. Ferber
Myra A. Ferber

Subscribed and sworn to before me
this 23rd day of June 1976.

Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U. S. Department of the Interior

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Alvin J. Heintz, 105 N. Second E., St. Anthony,
Idaho, being duly

sworn make the following voluntary statement to Vincent L. Duran,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

Since October 1971, I have been employed as Construction Inspector,
GS-9, Teton Dam Project, Bureau of Reclamation, Newdale, Idaho. I have
been employed by the Bureau since 1955.

At about 10:30 a.m. on Saturday, June 5, 1976, Pete Aberle telephoned
me at my home and asked me to come to the dam because there were some
leakage problems. I arrived at the office at about 10:55 a.m. and after
finding no one in the office drove across the top of the dam and found
Aberle on the north or right end of the dam near the spillway. I would
estimate the time to be 11:00 a.m.

As I drove across the dam I could see water spewing from the downstream
face of the dam near the north or right side abutment. I cannot estimate
the elevation of the leak. The water was flowing rapidly and was eroding
fill materials thereby making it muddy. There were two dozers on the
face of the dam pushing rock into the hole.

As I was talking to Aberle we noticed a small whirlpool forming in the
reservoir on the upstream side of the dam. The whirlpool was about two
feet in diameter, close to the north or right abutment and about 10 to
15 feet out from the dam. This was the only whirlpool I saw and to my
knowledge it stayed in the same location.

I remained on the top of the dam near the north end and helped direct
two dozers pushing riprap into the whirlpool. While working I saw the
downstream flow of water increase in volume and the whirlpool increase in
size. I cannot give estimates of the volume of water or the size of
the whirlpool or times of any significant increases.

At about 11:45 a.m., we instructed the two dozers on the top of the dam
to leave and I went off the north or right side of dam. The top of the
dam collapsed at about 11:50 a.m. This time estimate is not specific.
I never really considered that the dam would fail until the last minute.
To my knowledge there was no earthquake before the problems began.

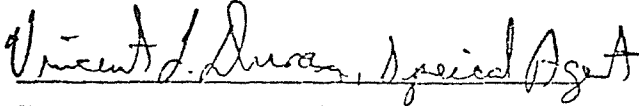
A.J.H.

Shortly after the collapse I left the north side and proceeded downstream to warn residents. I returned to the offices on the south side of the dam, ~~and~~ ^{helped} assist in crowd control. I cannot estimate the number of people who came to the dam after the failure, but we had problems keeping people off the rim edges and the dam itself.

I have carefully read the foregoing statement, consisting of 1 and 1/8th pages, and declare it to be true and correct.


Alvin J. Heintz

Subscribed and sworn to before me on this
23rd day of June 1976.



Vincent L. Duran, Special Agent
U.S. Department of the Interior

STATE OF IDAHO

SS

COUNTY OF MADISON

I, Kenneth C. Hoyt, Rt. 1, Box 202-12, St. Anthony,

Idaho, being duly sworn make the following voluntary statement to Vincent L. Duran, who has identified himself to me as a Special Agent of the U.S. Department of the Interior. No threats or promises have been made to obtain this statement.

I have been employed as Construction Inspector, GS-9, Teton Dam Project, Bureau of Reclamation, Newdale, Idaho since March 30, 1975, and I have a total of 16 years service with the Bureau of Reclamation.

Before June 5, I saw seepage in the bottom beyond the toe of the dam. This seepage was visible for about two or three days prior to June 5, and was 150 feet downstream of the toe of the dam. I never saw the seepage clearly, do not know the condition or volume. It was a slight flow and was of no great concern to me as it appeared rather natural.

On Friday, June 4, I saw nothing unusual at the dam. There were no leaks or no whirlpools up to 4:30 p.m. when I quit work.

On Saturday, June 5, 1976, at about 10:30 a.m., Pete Aberle called my home and left a message with my wife that I should be on standby to come to work on the midnight shift that night. I called Aberle back about 10:40 a.m. and he told me to come to the dam immediately. I drove to the dam and arrived on top of the dam at about 11:15 a.m. I saw a large stream of water running off the downstream side of the dam at about 5,200 slope and about 20 to 30 feet from the right abutment. (The elevation for the water level was 5,324 feet elevation. The elevation of the opening to the spill water was 5,306.) The stream of water was at about the change in slope elevation. The water was muddy. I also saw two dozers pushing rock into the hole created on the downstream face.

I also saw a whirlpool on the upstream face of the dam in reservoir water. The whirlpool was about 150 feet across the top of the dam from the spillway and about 15 feet out from the face of the dam into the water. It was rather close to the rock and abutment wall. The whirlpool was about 10 feet in diameter. There were two bulldozers pushing riprap into the pool. The water was clear. The dozers were creating discoloration in the water. When I saw the whirlpool I felt the dam was gone. The whirlpool gradually grew and was visible until I left the dam.

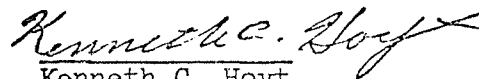
Someone said the dozers on the downstream face were gone. I looked and saw them tumbling down stream. Shortly thereafter the dozers on the top of the dam pulled back and headed to the south side of the dam. I followed the dozers in a pickup truck. When I got to the river outlet shaft house on top of the dam I turned around and saw the top of the dam collapse. I looked at my watch and noted the time to be 11:58 a.m.

Thereafter, I spend time controlling crowds. There were a number of people wandering around. I cannot estimate the number at this time. It was a very dangerous situation.

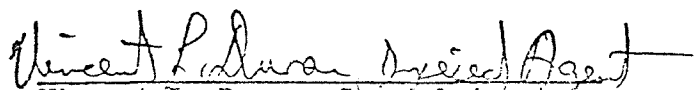
At about 2:00 p.m., Andrew Anderson and I went one mile upstream to check the water elevation, which at the time was 5,217 feet. At 2:30 p.m. the water level was 5,170 feet. There was no one around the area. At the time I could see a lot of water running out of rock on the right abutment across from the boat ramp. This was water in the rocks from the reservoir. There was no such water prior to the filling of the dam.

I am not aware of any earthquake tremors in the area.

I have carefully read the foregoing statement, consisting of 2 pages, and declare it to be true and correct.


Kenneth C. Hoyt

Subscribed and sworn to before me
on this 27th day of June 1976.


Vincent L. Duran, Special Agent
U.S. Department of the Interior

STATE OF IDAHO

SS

COUNTY OF MADISON

I, Harry Parks, Kit Circle Trailer Court,
Space 5, St. Anthony, Idaho, being duly sworn make the
following voluntary statement to Vincent L. Duran, ^{Betty J. Foyes} who have identified ~~themselves~~
themselves
to me as X Special Agents of the U.S. Department of the Interior. No threat
or promises have been made to obtain this statement.

I am employed as Supervisory Surveying Technician (Party Chief), GS-8, Teton Dam Project, Bureau of Reclamation, Newdale, Idaho, and I have held this position since April 1975. I previously worked for the Bureau of Reclamation at Forest Grove, Oregon from November 1968 to April 1975. I have been employed by the Bureau of Reclamation since November 1961.

About June 3, 1976, I observed a small stream of water appearing along the bottom of the waste area about 1400 feet downstream from the toe of the dam. I was on the top of the south rim when I observed this water and so I could not say at this time whether the water was clear, muddy, etc. I was aware that Robison and Aberle were watching the flow on at least one occasion.

On Saturday, June 5, 1976, I arrived at the project office a couple minutes before 7:00 a.m. I was driving a green Volkswagen. I parked the Volkswagen in the Reclamation parking lot. I was the first person to park a vehicle in the lot and Chris Felkins arrived shortly thereafter driving his white Chevrolet pickup truck. We left the office about 7:35 a.m. in a survey truck and traveled down the south rim road downstream for the purpose of checking the survey sights in order to perform a survey on the spillway on the north side of the dam. At about 7:50 a member of the survey party noticed water seepage. I then observed the water which was running out of the toe of the dam at about 50 feet from the north abutment wall. I cannot estimate the volume but it was barely what could be called a stream at all. The water appeared muddy, but this may have been caused by the material over which it was flowing. We drove back to the office and I reported the water leakage about 8:00 a.m. to Jan Ringel.

After reporting the water, we departed the project office and drove across the top of the dam and parked our vehicle near the spillway bridge on the dam. At about 8:20 a.m. I checked the water elevation on the reservoir or upstream side of the dam, near the spillway inlet, and it was 5301.7 elevation. This was about three feet of the gate level of the spillway. At this time I noted nothing unusual on the reservoir side of the dam so far as the water was concerned. There was no whirlpool and in fact the water was unusually calm. There was no

discoloration of the water. There were no fishermen or any other persons on the reservoir side at this time. This area is posted against fishing.

I went down into the spillway and made no observation of the downstream face of the dam at this time. I was working in the spillway and my view was blocked of the downstream face of the dam, and it was not until about 9:30 a.m. that I could see a dozer coming off the top of the dam to work on the downstream side. At about 10:30 a.m. I came up to the top of the spillway. I walked onto the sage area and observed a leakage about 50 feet from the north abutment and somewhere above the 5200 elevation. I cannot estimate the volume of the water but it was a running stream. I would estimate the hole was about five feet in diameter and the water was muddy. We watched the water about five minutes and the hole may have increased as much as a foot during this time. He does not recall seeing any dozers working at the hole at this time.

We then went back down the spillway to continue our survey work. I was aware of a lot of activity at the top of the dam in that there were a lot of people moving about and two dozers moved across the dam. Between 11:15 a.m. and 11:30 a.m. I could hear water flowing and made the assumption that it was coming out of the hole, but I could not see it from where I was working. At about 11:45 a.m. Pete Aberle called to the survey crew and told us to leave the area. I did not have the feeling at that time that the dam was in imminent danger of collapse and if I had, I would have left the spillway earlier. I would estimate that it was close to 11:50 when I reached the top of the dam. At this time the hole on the downstream face of the dam had eroded almost to the top and muddy water was rushing out of it. There was a pickup truck on the top of the dam and two dozers. The dozers were pushing riprap into the water on the upstream side.

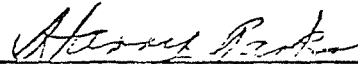
I did not see the whirlpool which developed on the upstream side of the dam. I did not see the water on the upstream side of the dam at all until the dam broke. I was standing a few feet from the spillway bridge in the middle of the road. I saw half of the top of the dam go and shortly thereafter the other half (upstream) went. I was wearing a watch but did not note the time, but it was close to noon.

The first time that I became aware that there was imminent danger of the dam collapsing was when edge of the hole came close to the bottom of the road. This was shortly after 11:50 a.m.

I am not aware of any earthquake tremors. The only tremors I am aware of was when the spillway tremored a little bit about 11:45 and I believe this was caused by the rush of the water.

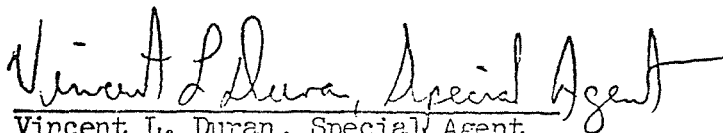
I departed from the north side of the dam at about 12:05 p.m. I did not participate in any crowd control operation, since there were no members of the public on the north side at that time.

I have read the above statement consisting of two and one-quarter pages and declare it to be true and correct.

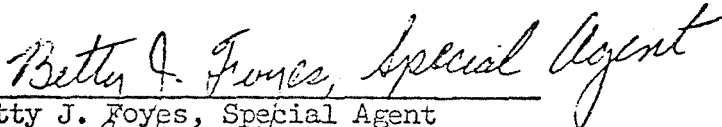


Harry Parks

Subscribed and sworn to before me
this 22nd day of June 1976



Vincent L. Duran, Special Agent
U. S. Department of the Interior



Betty J. Foyes, Special Agent
U. S. Department of the Interior

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Jan R. Ringel, 520 Targhee Street, St. Anthony,
Idaho, being duly

sworn make the following voluntary statement to Vincent L. Duran,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I am employed as Civil Engineer, GS-11, Teton Dam Project, Bureau of Reclamation, Newdale, Idaho. In this function I act in the capacity of Chief of Surveys and Principal Inspector. I have been employed on the project since September 1972. I previously had one and one-half years service with the Bureau of Reclamation.

On Saturday, June 5, 1976, I arrived at work at 7:00 a.m. I had two survey crews working. My office is in a trailer behind the office complex at the project. Mr. Parks checked the staffs for the spillway control on the south side of the dam opposite the spillway. They were on the canyon rim and noticed the lower leak on the dam near the toe at about 5,041.5 elevation. At about 7:30 a.m. Parks reported sightings to me. I drove down to the powerhouse and walked over to the leak. The water was muddy. The water was running between the rocks on the right abutment and not through the dam. I estimate the water flow to be about 20-30 cfs at this time. I did not detect any increase at that time.

The only other noticeable thing at this time was some springs at the base of the dam against the abutment--200 feet below the other. This had been there for one or two days previous. This was clear water running at about 10 gallons per minute. Mr. Aberle and Mr. Robison had previously checked this.

At about 8:20 a.m. I telephoned Mr. Aberle at his home in Rexburg. At about 8:50 a.m. Mr. Aberle and Mr. Robison arrived at the dam. I briefed them lightly and we drove over the top of the dam to the right abutment. At this time Mr. Robison and Mr. Aberle walked down the downstream face of the dam to look at the leak. I drove the pickup around the rim road to meet them at the bottom. When I arrived, I walked directly to the right abutment. I stopped momentarily at the powerhouse and took some pictures of the leak, then proceeded to the riprap stockpile where Mr. Robison and Mr. Aberle were observing and deciding what to do with the water running out of the abutment. We then proceeded to the pickup and went to the Morrison-Knudsen Company and Peter Kiewit Sons' Office to contact Mr. Buckert

to mobilize some equipment, namely two dozers and one front end loader to make a channel from the water source to the river so that the water would not get into the powerhouse. After our conversation with Mr. Buckert we returned to the office to get some help. We called Mr. Al Stites and Mr. Al Heintz to check on the contractor's work and look for other leaks along the canyon. Mr. Robison wanted to know the reservoir water elevation so I returned to the right abutment where Mr. Parks was working in the spillway to get this information because he had read it at approximately 8:00 a.m. that morning. The reservoir was at elevation 5301.7. I then returned to the office to give this information to Mr. Robison. I then went out the front door of the Bureau of Reclamation office to talk to Mr. Aberle, who was returning from Morrison Knudsen Company and Peter Kiewit Sons' office. This was approximately 10:30 a.m.

At about 10:30 a.m. I heard water running. Mr. Aberle and I ran down to look over the side of the Canyon. At this time we discovered the upper leak on the right side at approximately 5200 elevation, and approximately 15 feet from the abutment. The water was washing zone 5 material - varying sizes, down the slope. The water was a muddy color and was running at 10-20 CFS, I would guess very roughly. Mr. Aberle ran back to the Morrison-Knudsen Company and Peter Kiewit Sons' to inform them of the new development and I ran into the Bureau office to tell Mr. Robison. I then went back down to the powerhouse to get the gates open if someone was available. Stites was there. I saw the two cats working on the downstream face of the dam. I told Andrew to prepare to open the gates but this was never done.

I then drove up to the top of the dam. At approximately 10:50 a.m. a whirlpool developed on the upstream face of the dam. This was at the right of the dam about 15 to 20 feet away from the dam. Gibbons and Reed dozers were pushing in riprap. I cannot estimate the circumference of the whirlpool or its activity. I only saw it momentarily. I realized then that we had big trouble. I did not watch continuously.

When the whirlpool developed two dozers from Gibbons and Reed Company immediately started working on the upstream face of the dam trying to push riprap and zone 2 material into the whirlpool to stop the leak.

I saw a pickup truck going to Wilbur Peterson and Sons, the clearing contractor. John Blowers and Miller went to get a cat. I went to tell them where we needed work. They did not have a key to the cat. I went and got one for them and returned to the dam. This occurred between 11:00 - 11:30 a.m. When I returned to the dam the cats on the upstream face were pulling off. This was about 11:40 a.m. The operators of the downstream cats were running across the dam. The dam collapsed at 11:57 a.m.

I recall that there was a farmer in a green pickup truck at the dam on the north side sometime between 9:00 and 11:00 a.m. The man said "what is going on here?", "Is it serious?" I told him yes, the dam is breaking. The man said "I am going to get out of here. I have a farm down below." I do not know the name of the man and cannot identify him.

Within two hours of the collapse of the dam, there were at least 15 people on the north side of the dam around the spillway and on the edge of the collapsed area. There was considerable problems with crowd control throughout the afternoon.

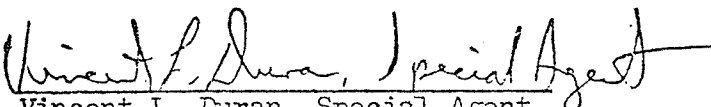
I am not aware of any earthquake tremors.

I have carefully read the foregoing statement consisting of two and one-half pages and declare it to be true and correct.



Jan R. Ringel

Subscribed and sworn to before
me this 23rd day of June 1976



Vincent L. Duran, Special Agent
U. S. Department of the Interior

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Robert R. Robison, 581 Taurus Drive, Rexburg, Idaho

_____, being duly
sworn make the following voluntary statement to Vincent L. Duran and Betty J. Foyes
have themselves
who ~~has~~ identified ~~himself~~ to me as ~~X~~ Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I am employed as Project Construction Engineer, GS-14, Teton Dam Project, Bureau of Reclamation, Newdale, Idaho and have held this position since August 1971. I have been employed by the Bureau of Reclamation since 1951. I received a B.S. degree in engineering in 1950 from the University of Utah.

While there were rumors as early as April 1976 that there were leaks at the dam, there is no basis to these rumors, because there were no leaks.

On June 3, 1976, several small seeps in the rhyolite (volcanic rock) appeared about 1400 to 2000 feet downstream from the toe of the dam in the north abutment wall. The water was clear and all of these seeps totaled about 100 gallons of water per minute. This was felt to be a good sign because the dam was being filled and it indicated the water table gradient was acting in a normal manner. The water was clean enough to drink and if there had been a problem the water would have been turbid. I felt the area should be monitored by sight inspections and other mechanical means, the latter of which were never put into effect. I took pictures of the seepage and reported the matter to the E&R Center, Bureau of Reclamation, Denver, Colorado.

On June 4, 1976, a small seepage occurred about halfway between the toe of the dam and the end of the spillway along the north abutment. This flow was approximately 20 gallons per minute and I had no concern because the water was clear. I checked this leak at about 4:30 p.m. on June 4 before leaving the dam and determined that there was no problem. At this time I also observed the entire downstream face of the dam and observed nothing unusual. I also observed that there was nothing unusual on the upstream reservoir side of the dam.

On June 5, 1976, at 8:30 a.m. I received a telephone call at my home from Pete Aberle's wife. She told me that Ringel had called Aberle and said there was a large leak in the dam. I left my home immediately and arrived at the Reclamation Office at about 9:00 a.m. Aberle and I drove to the downstream toe of the dam and I observed a major leak at the downstream toe at the right abutment at about 5045 elevation. The water was flowing at about 50 cubic feet per second, was moderately turbid and was coming from the abutment rock. This was not connected to the other seepages mentioned above. I felt this seepage was coming straight out of the abutment rock and not through the dam.

I also saw another leak at about 5200 elevation in the junction of the dam embankment and the right abutment. The water was slightly turbid and issuing from the rock at about 2 cubic feet per second. The water from this leakage was not flowing at a great enough volume to even reach the toe of the dam.

At about 9:30 a.m. Aberle and I went to the south rim area of the dam and located Duane Buckert, Project Manager for Morrison-Knudsen and Kiewit. We discussed control measures and decided to excavate a channel at the toe of the dam to protect the powerhouse. At this point I felt that the situation was critical but we could control the leaks, since they were coming from the abutment rock. I made calls to the Bureau of Reclamation Regional Office in Boise, Idaho and talked to Harry Stivers, Assistant Regional Director, since the Regional Director was not available, and the E&R Center, Bureau of Reclamation, Denver, Colorado. These calls were only for the purpose of alerting those offices to the problem. I also considered the matter of alerting area residents at this time, but decided that an emergency situation was not imminent and he did not want to cause a panic. These calls were made between 9:30 and 10:00 a.m.

At about 10:00 a.m. I observed a large leak developing about 15 feet from the right abutment in the dam embankment at an approximate elevation of 5200 feet. This leak was on the downstream face of the dam and was adjacent to the smaller leak at the same elevation. At first the flow of water was about 15 cubic feet per second and it gradually increased in size. The water was turbid. By about 10:30 a.m. two Morrison-Knudsen dozers were sent to the area of this leak and instructed to push rock into the hole.

At about 10:30 a.m. to 10:45 a.m., I notified the sheriff's offices in Madison and Fremont Counties and advised them to alert citizens of potential flooding from the Teton Dam and to be prepared to evacuate the area downstream. I also received a call from Ted Austin, a radio announcer in St. Anthony, Idaho and advised him of the possible danger. There was no equivocation on my part about advising people of the danger at this time.

RRR

At about 11:00 a.m. I saw a whirlpool developing on the upstream side of the dam in the reservoir at about 10 to 15 feet into the water from the face of the dam and less than 100 feet from the abutment wall. I had looked for a whirlpool at about 10:30 a.m. and had not seen one. The whirlpool was approximately six feet in diameter, was stationary, and appeared to be increasing in size. The water on the reservoir side was clear. The approximate elevation of the whirlpool was 5295. I would estimate that at this time the volume of water going through the upper leak on the downstream face of the dam was 100 cubic feet per second.

At about 11:00 a.m., or soon thereafter, two Gibbons and Reed dozers came across the top of the dam and were directed to begin pushing zone 2 and riprap material into the whirlpool area. The dozers had to create a ramp down the face of the upstream side of the dam in order to get the riprap into the whirlpool and were never completely effective.

At about 11:30 a.m. the two Morrison-Knudsen dozers on the downstream face of the dam were lost in the washout area and carried downstream by the rush of water. I may possibly be the individual in the center of the Time magazine picture, walking away from the dozers as they were falling into the washout area.

At about 11:45 a.m. the two Gibbons and Reed dozers working on the upstream whirlpool were pulled off their job of pushing riprap into the whirlpool and they proceeded to leave the top of the dam, heading for the south side. At this time I was on the road heading toward the Project Office and I saw the top of the dam collapse from this location. I did not note the time, but when I got to the office the clocks had stopped at about 11:57 a.m. because of power failure and I assume this was the time of the collapse. Aberle told me he noted the time of collapse to be 11:57 a.m.

At 12:10 p.m. I departed the dam site for Rexburg, Idaho, in order to place telephone calls to Bureau officials in Boise, Idaho and Washington, D. C.

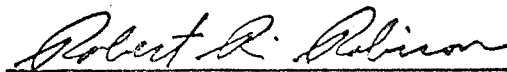
When I noted the whirlpool developed at about 11:00 a.m. I realized there was imminent danger of the dam collapsing. From this time on there were numerous people making telephone calls alerting people in the area of the danger.

I am not aware of any earthquakes or earth tremors which may have caused the ultimate collapse of the dam.

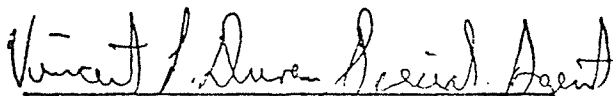
Contractor personnel were busy during the morning hours attempting to clear equipment out of the river outlet tunnel on the south side of the dam in anticipation of opening the river outlet tunnel to relieve the pressure of the water on the dam. The contractor's employees had to evacuate the tunnel before they had accomplished their task. I doubt that the opening of the tunnel would have been effective in preventing the collapse of the dam.

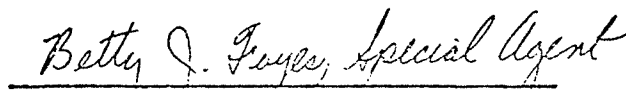
At the time of the dam collapse there was no schedule of work shifts for Bureau of Reclamation employees that would have required persons at the dam 24 hours a day. On Saturday June 5, 1976, the only scheduled Bureau of Reclamation workers were the survey crews. There were scheduled quality control inspections according to the work being done, but there were no scheduled physical plant inspections of the dam on a routine basis by the inspectors.

I have carefully read the foregoing statement, consisting of three and one-quarter pages and declare it to be true and correct to the best of my knowledge and belief.


Robert R. Robison
Robert R. Robison

Subscribed and sworn to before
me this 23rd day of June 1976


Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U. S. Department of the Interior


Betty J. Foyes, Special Agent
Betty J. Foyes, Special Agent
U. S. Department of the Interior

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Alfred B. Stites, P. O. Box 155, St. Anthony, Idaho

_____, being duly
sworn make the following voluntary statement to Vincent L. Duran,
who has identified himself to me as a Special Agent of the U. S.
Department of the Interior. No threats or promises have been made to
obtain this statement.

I am employed as a Construction Inspector, GS-9, Teton Dam Project,
Bureau of Reclamation, Newdale, Idaho. I have held this position
since June 1962 and have 16 years service with the Bureau of Reclamation.

On Saturday, June 5, 1976, Jan Ringel telephoned me at my home and
asked me to come to the dam immediately because there were problems.
I arrived at the dam site about 10:15 a.m. and Ringel told me there
was a leak in the downstream face of the dam and asked me to see about
getting a dozer to channel water away from the powerhouse. At about
10:30 I proceeded to the powerhouse and saw a leak in the dam on the
downstream face at about 5200 elevation and near the right abutment wall.

When I arrived at the dam I talked to John Bellagante, who was preparing
to take a dozer to the leak area. I also ran into Llewellyn Payne, who
was going into the river outlet tunnel with three other men to remove
equipment in order that the tunnel could be opened.

I then walked up the downstream face of the dam and passed the two
dozers which were working at the 5200 elevation and trying to fill in
the hole. The seepage water was muddy, but I cannot estimate the volume.
I arrived at the top of the dam at about 10:40 a.m. and within three or
four minutes I noticed a whirlpool forming in the reservoir on the upstream
side of the dam about 22 feet into the water from the face of the dam. ~~and~~
~~about 22 feet into the water from the face of the dam.~~ The whirlpool was approximately
1 1/2 feet in diameter at the outset, briefly got smaller, and then began
increasing in size. The water in the area of the whirlpool
appeared to be slightly muddy. I watched the whirlpool for possibly
five minutes and then ran back down the downstream face of the dam to the
area of the powerhouse on the left side (south). Before I left the top
of the dam two dozers were beginning to push riprap into the whirlpool.
This was about 10:45 a.m. or shortly thereafter.

When I arrived at the powerhouse area I noted that one of the dozers working on the downstream face was falling into the washed out area and the other dozer was attempting to pull it out. A very short time thereafter both dozers were washed away in the stream of water. The volume of water at this point had increased tremendously and the water was very muddy.

Shortly after 11:00 a.m. Payne and his fellow workers evacuated the river outlet tunnel and three other persons in the powerhouse area were evacuating motorized equipment to higher ground near the dam. I drove to the upper south rim opposite the spillway and observed the washout area on the downstream face continually increase and portions of the dam falling into the vacuum. This was during the period 11:30 a.m. until almost 12:00 noon when the top of the dam finally collapsed. I felt that the dam was definitely going to collapse shortly after 11:00 a.m. when the two dozers were washed away.

I remained at the dam until about 10:30 p.m. and much of this time was spent trying to keep spectators behind the visitors point on the south rim. I cannot estimate the number of spectators that were there during the day.

During the afternoon, after the water had receded, it appeared to me that the grout cap was still in place. I noticed some water was running out of the right abutment, upstream of the grout cap, but I did not observe any water running out of the abutment downstream of the grout cap.

I have carefully read the foregoing statement consisting of one and one-half pages and declare it to be true and correct.

** Alfred D. Stites

Alfred D. Stites

Subscribed and sworn to before
me this 22nd day of June 1976

Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U. S. Department of the Interior

GIBBONS & REED-CONTRACTOR WITNESS STATEMENTS

Harold F. Adams
Route 3, Box 259
Rigby, Idaho

Dave Burch, Mechanic
P.O. Box 384
Ashton, Idaho

Jerry Dursteler, Master Mechanic 524-1396
280 Wilson Drive
Idaho Falls, Idaho

Perry Ogden, Mechanic 356-7920
Rexburg, Idaho

Lynn Walker, Superintendent 458-4304
Behind June's Bar
Teton

COPY

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Harold F. Adams, Rt. 3, Box 259,
Rigby, Idaho, being duly

sworn make the following voluntary statement to Vincent L. Duran,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I am employed as a mechanic with Gibbons and Reed Company on the Teton Dam
Project, Newdale, Idaho. I just started on that project about June 1, 1976.
Previously with company three years.

On Saturday, June 5, 1976, I arrived at Gibbons and Reed yard behind
Bureau office at 7:00 a.m. to work on equipment. As I drove in I saw a
small trickle of water on downstream slope of dam against the north abutment
and about 100 feet from top of dam. About 30 feet out there was a wet
spot.

At about 8:00 a.m. I walked from the shop out to south rim to look at
leak again. Now small stream coming out from where we saw wet spot. At
about 9:30 or 10:00 a.m. Dursteler told us to look at leak. From south
rim I saw a 6 or 8 inch diameter flow of water. Dursteler said we had
trouble.

I went about 2 miles downstream out of site of dam to get equipment out
of possible danger area. Just before leaving I told my wife to be on the
alert because of leak. I was (sic) downstream about 30 or 40 minutes.

When I got back water flow had increased and Gibbons and Reed dozers out
on top of dam working. The time was between 10:00 and 11:00 a.m. I
watched from visitor viewpoint.

I would estimate dam collapsed at top somewhere around 11:30 a.m. and the
dozer had gotten off just before that.

I cannot be specific about times. No earthquake or tremor. I never saw
upstream side during the day.

I was in the area until 5:30 p.m. but did not get involved in crowd control.

I thought dam would go at about 9:30 a.m. when the flow of water had increased.

I did take note of Morrison and Knudsen tractor activity and saw them get washed away. I do not know the time.

I have carefully read the foregoing statement, consisting of 3 pages and declare it to be true and correct.

/s/ Harold F. Adams

Subscribed and sworn to before me
this 22nd day of June 1976

/s/ Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U.S. Department of the Interior

When I crossed the top of the dam driving my D-8 cat there was a large flow of water coming from the hole in the dam on the downstream side. I would estimate that the hole was 10 to 12 feet in diameter and the water was muddy and rocky. The M-K dozer operators were pushing riprap and gravel into the hole. When the M-K dozers were caught in the hole, M-K personnel asked me to try to save their dozers by me backing my cat over the face of the dam and pulling them out, but it was too late. The dozer on the extreme righthand side of the Time Magazine photograph, Page 57, is mine. I am the man on the left of the two individuals standing near the cat in the picture. The other man who is standing to my right is the M-K employee who had requested my help in pulling out their cats. The red truck to my left belongs to Owen Daley, an M-K employee.

I had started pushing riprap from the face of the dam towards a whirlpool or funnel which had developed on the reservoir side of the dam shortly after 11:00. The whirlpool was directly across from the spot where the hole appeared on the downstream face of the dam. When I first saw the whirlpool, it was very small, maybe a foot across and was very muddy and it was surrounded by clear water. I saw no other mud on the upstream side. The water on the reservoir side was very calm. There was very little wind. The whirlpool was about 20 feet out from the upstream face of the dam and about 100 feet from the north abutment. We tried by using the riprap to build a ramp to the whirlpool but never succeeded. Two M-K men then came and took the cat I was driving and the one Perry was driving since neither of us are cat operators. It was after I got off the cat that the picture was taken which appears in Time magazine. Perry and I left at this point to obtain a 968 cat loader to load fines to help plug the hole on the downstream side. At the time we left the two M-K men were operating the cats at the top of the dam, having lost theirs in the hole. There was also a pickup truck on the top of the dam. I was sitting in the 988 Loader near the M-K shops when the dam collapsed at about 12:00 noon or a little later.

I first became aware that the dam was in danger of collapsing when the water started running through the hole on the downstream face of the dam at 10:00 a.m.

I at no time felt earthquake tremors at the dam.

I saw only one whirlpool on the reservoir side of the dam and when I left the dam I would estimate it was 20 feet in diameter.

I have read the above statement consisting of four and one-half handwritten pages and declare it to be true and correct.

/s/ David L. Burch
David L. Burch

Subscribed and sworn to before
me this 22nd day of June 1976

/s/ Betty J. Foyes
Betty J. Foyes, Special Agent
U. S. Department of the Interior

STATE OF IDAHO

SS

COUNTY OF MADISON

I, Jerry Dursteler, 280 Wilson Drive
Idaho Falls, Idaho, being duly sworn make the following voluntary statement to Vincent L. Duran, who has identified himself to me as a Special Agent of the U.S. Department of the Interior. No threats or promises have been made to obtain this statement.

I am employed as Master Mechanic, Gibbons and Reed, Teton Dam Project, Newdale, Idaho. I have been on this job since February 1976.

On Saturday, June 5, 1976, Perry Ogden and I arrived at the company yard behind the Reclamation offices at about 10:00 a.m. We came to do maintenance work on equipment. When at the office, I heard water running. I drove downstream from the dam on the upper south rim road to look at the spillway and to see if water was flowing over it. I saw wetness on the downstream face of the dam and seepage against abutment wall. This was about at the slope change in the dam. I cannot be more specific. The water was muddy, but was merely a light stream. I went back to my truck. By then the wet spot had started flowing. This was a very small flow. I returned to my office and told Adams and Burch there was a problem. The three of us walked behind the Reclamation offices on the outside of the dam to look at the dam. The leakage had increased considerably and started eroding a hole. This was about 10:15 a.m.

I then returned to my office and Perry Ogden and I started toward the dam in a truck. We ran into Robison and agreed to move two dozers out on top of the dam for whatever purpose. I radioed Lynn Walker and asked him to come to the site. Ogden and Burch moved two dozers onto the dam. I remained in the office area taking pictures of the downstream canyon walls and some of the face of the dam. I took pictures from the visitor's viewpoint, downstream rim and from the Morrison and Knudsen yard.

Between 10:15 and 10:30 a.m. two Morrison Knudsen dozers were pushing material into the downstream face of the hole. The hole was very large with a big stream of water. Gibbons and Reed dozers got onto the top of the dam. I saw Morrison-Knudsen dozers wash out but have no idea of the time.


John Bellaganti and Owen Daley operated Gibbons and Reed dozers pushing rock into the whirlpool area on the upstream side of the dam. I never saw the whirlpool. I watched activities, but cannot give time elements. I was watching primarily from the visitor's observation point. Gibbons and Reed dozers were pulled out and just barely cleared the top of the dam when it collapsed.

I believe the time element was two hours from the time I arrived, therefore the dam collapsed about 12:00 noon.


During the events the area at the visitor's center was completely full. I do not know any of the people who were on the visitor center.

During my observation the water was muddy and the area of leakage grew bigger at a very fast pace. I am not aware of any earthquake-like tremors.

I have carefully read the above statement consisting of one and one-half pages and declare it to be true and correct to the best of my knowledge and belief.


Jerry Darsteler

Subscribed and sworn to before
me this 22nd day of June 1976


Vincent L. Duran, Special Agent
U. S. Department of the Interior

Darsteler stated orally that at about 10:30 a.m., when the Morrison-Knudsen dozers were lost on the downstream side of the dam he realized the collapse of the dam might be imminent.

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Perry W. Ogden, Rt. 1, Rexburg, Idaho

_____, being duly
sworn make the following voluntary statement to Vincent L. Duran,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I am employed as a mechanic with Gibbons and Reed Company doing Canal
construction work at Teton Dam Project, Newdale, Idaho. I have been with
Gibbons and Reed at Project since February 1976. Previously with Morrison-
Knudsen-Kiewit at the project about 2 years.

On Saturday, June 5, 1976, I was scheduled to do maintenance on equipment
at the shop area behind the Reclamation offices. I arrived at shop at
7:00 a.m., went right to our shop area. I was out of view of most of
dam, but could see top part. Shortly after I arrived, Dave Burch told me
there was a wet spot on the downstream side of dam. I walked over to
visitor's viewpoint on south rim and saw a wet spot at about 100 feet
from top of dam against abutment. No flowing water--just a wet spot.

Between about 8:30 a.m. and 9:00 a.m., Dursteler arrived at work and told
me water was running through dam. I went to Reclamation office and talked
to Robison. He asked for all dozers we could get to dam area. I went
down road and got dozer and returned to top of dam with dozer. This
was about 9:30 a.m. On downstream face there was good flow of water and
a hole about 30 feet in diameter. Morrison and Knudsen dozers working on
this hole. Burch arrived with a dozer and the two of us crossed the dam
and started pushing riprap into whirlpool. This probably about 10:00 a.m.
or so. Whirlpool developed at this time about 4 feet in diameter.

Sometime after this Bellegante came up and told me the dozers on downstream
face were gone. He and Daley came up and took over Burch and my dozers.
About 10 minutes later Burch and I drove a pickup to the southside of dam
and went to viewpoint.

I stood on viewpoint about one hour watching, with exception of one phone call to my wife. During this hour the dam kept eroding and more water flowing. I was certain dam was going to collapse. The Gibbons and Reed dozers cleared out almost last few minutes and came across dam. Just about noon when dam collapsed. I recall looking at my watch right after top fell and it was 11:55 a.m.

No earthquake or tremors.

There were a large number of visitors. Visitor area full and was lined up along entrance road.

I have carefully read the foregoing statement, consisting of 3 pages and declare it to be true and correct.

/s/ Perry Ogden

Subscribed and sworn to before me
this 22nd day of June 1976.

/s/ Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U.S. Department of the Interior

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Jerry Lynn Walker, Teton Trailer Court, Teton, Idaho
695 East 1st North, Pleasant Grove, Utah (permanent), (Temporary)
being duly
sworn make the following voluntary statement to Betty J. Foyes,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I am employed as Superintendent, Gibbons and Reed Construction Company,
and have held this position for about 12 years. I have been employed
by Gibbons and Reed at the Teton Dam Project, Bureau of Reclamation,
Newdale, Idaho for about three months. Gibbons and Reed has had a
contract since about April 1976 to construction irrigation canals
and a water pipeline to other canals below the dam.

On June 5, 1976, I arrived at the Teton Dam about 10:30 a.m. because
subordinate Gibbons and Reed employees had called me on the radio at
my home to advise me that the dam was leaking. I would estimate that
I was called about 10:15 a.m. I immediately went to a point just
downstream from the visitor's observation point, on the southside of
the dam. At that time I observed a hole approximately 3' in diameter
located at the 5200 elevation, near the abutment wall (north). There
was a sizeable flow of muddy water coming from a portion of the hole
and it had begun to wash out a trench. There was a dozer coming down
the slope of the dam toward the hole. At this point I knew the dam
was gone and I went back to my office to call my family. I then returned
to observe the dam after making one other call. The time which elapsed
was probably 15 minutes. This would place my return to the dam shortly
before 11:00 a.m. By then the second dozer was in position and the
two dozers were trying to push rock into the growing hole. The hole was
growing fast and was about 10 to 12 feet in diameter at this time.
The stream of muddy water had increased in volume correspondingly.

By the time I had arrived at the dam at 10:30 a.m. two D-8 dozers belonging
to Gibbons and Reed had been dispatched to the top of the dam to
work on the upstream face and push riprap into a whirlpool which had
developed. Two of my mechanics had obtained the D-8 dozers and had
begun this work. The two Morrison-Knudsen dozers on the downstream
face of the dam were lost at about 11:15 to the best of my recollection.
At this point I went to the top of the dam to order my two dozers
to stop work and leave the top of the dam. While I was standing on
the visitor's observation point and after the two M-K dozers were lost

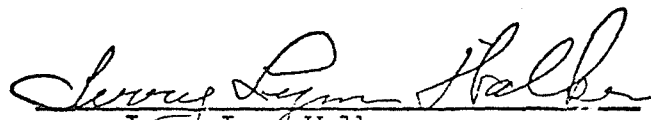
a crack developed above the hole. The crack was in the shape of a semi-circle with the arc at the top; was about 30 feet above the hole; and I would estimate that it may have been as much as 100 feet in total length. The earth starting sluffing down from the crack towards the hole and caused an offset in the earth on the face of the dam as it sank. As this earth fell in a small hole developed above the crack. I would estimate this was about 10 to 15 feet above the crack and was initially six or seven feet in diameter. I then left the visitor's observation point and drove to the top of the dam. I would estimate that I reached the top of the dam at about 11:40 a.m. My cats were already coming across the top of the dam towards the south side. As soon as I saw that my cats were getting off the dam, I drove back to the visitor's observation point and observed that while my cats were about one-third of the way across the top of the dam sluffed down about 100 feet. About 11:55 a.m. the dam failed.

I at no time was in a location where I could observe the whirlpool which had formed on the upstream side of the dam.

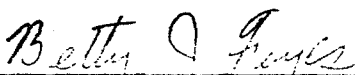
I at no time felt earthquake tremors at the dam site either before June 5 or on June 5, 1976.

About 7:30 p.m. on June 5, 1976, after the water was down to the lowest level it would reach at that point, I was at the upper curve of the M-K access road on the south rim of the dam and I observed a six inch stream of water coming out of the north side abutment rock. The water was clear as a bell. The water was coming from a spot about 100 feet down from where I would estimate the crest of the dam had been. We took some photographs and I may be able to furnish a picture of this occurrence. We have some other photographs of the collapse of the dam and I will make arrangements to have a set of the photographs furnished to the Department of the Interior.

I have carefully read the foregoing statement consisting of one and one-half pages and declare it to be true and correct.


Jerry Lynn Walker

Subscribed and sworn to before
me this 23rd day of June 1976


Betty J. Foyes, Special Agent
U. S. Department of the Interior

MORRISON-KNUDSEN, KIEWITT EMPLOYEE WITNESS STATEMENTS

John P. Bellegante
Teton Villa Apts.
Rexburg, Idaho

Duane E. Buckart
Kit Circle #14
St. Anthony, Idaho

Jay M. Calderwood
Victor, Idaho

Roy C. Cline
Kit Circle #22
St. Anthony, Idaho

David O. Daley
330 W. 8th St., Space 6
St. Anthony, Idaho

Llewellyn L. Payne
P. O. Box 37
Ashton, Idaho

Vincent M. Poxleitner, Jr.
P.O. Box 22
Teton City, Idaho

Barry W. Roberts
Kit Circle No. 1
St. Anthony, Idaho

Donald D. Trupp
P.O. Box 3
Newdale, Idaho

COPY

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, John P. Bellegante, 262 N. Second W.,
Rexburg, Idaho, being duly

sworn make the following voluntary statement to Vincent L. Duran,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I am employed as excavation superintendent, Morrison-Knudsen and Kiewit
at Teton Dam Project, Newdale, Idaho. I have worked there since March
1975.

On Saturday, June 5, 1976, at about 10:00 a.m., Duane Buckert telephoned
me and told me the dam was leaking and to come to project. I arrived
at the project at 10:30 a.m. and went directly downstream of the dam
in the area of the powerhouse. Buckert told me by radio he wanted to
try and fill in hole. I saw a leak near north abutment side at about
5250 feet elevation. There was a fast flow of water down slope and
there were several gallons per minute coming down. Water was muddy.

Prior to Saturday, there were leaks on north side at toe elevation of
dam. These were on north side. There were three that I know of. Clear
water on these leaks. This was about Tuesday or Wednesday.

On Saturday, I went on top of dam, got a dozer and instructed Owen Daley
to get another dozer. The two of us went down and started pushing
rock into face of leak on the downstream face. This was about 10:40 a.m.
To my knowledge there was no increase in the leakage.

My dozer settled into hole created by leak. I got a cable at top of
dam and hooked the two dozers together. We were unsuccessful and both
dozers went with the water. I would estimate this to be about 10:55 a.m.
to 11:00 a.m. I looked down into the hole. White water was gushing
out of the north abutment through the rocks and creating the muddy water.

I then went to top of dam. Others had found whirlpool on upstream face
and were directing dozers to push riprap into whirlpool area. Whirlpool
was about 18 inches in diameter near the north abutment wall about 15 feet
from upstream face of the dam. I did not notice it getting bigger. I
could feel the dam area settling and pulled the dozers out. Dozers went
to southside and I went to northside. I cannot give time elements of this.

Shortly afterwards, the dam collapsed. I do not know the time. At no point did I think the dam was going to collapse. These were the last thoughts I had until immediately before it went.

I proceeded down the northside rim with others notifying people and eventually returned to office on southside. I was in the area until about 5:00 p.m.

I have carefully read the foregoing statement, consisting of 3 pages and declare it to be true and correct.

/s/ John P. Bellegante

Subscribed and sworn to before me
this 19th day of June 1976.

/s/ Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U.S. Department of the Interior

COPY

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Duance E. Buckert, Kit Circle #14
St. Anthony, Idaho, being duly

sworn make the following voluntary statement to Vincent L. Duran,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I am Project Manager, Morrison-Knudsw-n-Kiewit on Teton Dam Project, Newdale,
Idaho. I have been on the project for two years.

On Saturday, June 5, 1976, I arrived on job at approximately 8:30 a.m.
and drove out on top of dam. As I drove out I saw water coming out of
side at abutment below toe of dam. The water was clear. I was told
Robison had been notified. Robison, Aberle, Ringel and I met regarding
this leak and decided to channel this water to reduce erosion and keep
away from power house. I agreed to get people and went to the office to
make telephone calls to employees. While doing this, Aberle came in about
10:00 a.m. and told me another leak had appeared.

At about 10:20 a.m. I went out and saw leak on downstream face of dam. This
leak at about elevation 5200 and about 10 or 12 feet out from abutment. The
area eroded out was about six feet by six feet. The flow, I cannot estimate,
but it was muddy and erosion was occurring. I sent two dozers in to
push rock into the hole. I then went down into tunnel area at power-
house to get it cleaned out for possible opening. The erosion was increasing
at the leak area.

I went back up on top of the dam and ran into Robison. He told me two
dozers had been lost on the downstream face. We talked about opening the
river outlet tunnel. This was about 11:00 to 11:20 a.m. A whirlpool had
developed on the upstream reservoir of dam. I did not actually see the
whirlpool, but saw dozers pushing materials in.

I proceeded to office when Poxleitner told me he did not believe the
dam would hold. This was sometime around 11:20 a.m. I then went to the
office and made telephone calls to notify area residents of the danger.
During this time I observed the increasing turbulence of water, but did
not actually see the final collapse. I saw the dozers on top of dam
leaving and collapsing earth behind them. The time of 11:57 a.m is close
to the actual time of failure.

I realized the loss of dam when I heard about the whirlpool at about 11:30 a.m. This was the first time the facts really dawned on me.

I was not involved in crowd control. This was handled by the Bureau of Reclamation.

I have carefully read the foregoing statement, consisting of 3 pages and declare it to be true and correct to the best of my knowledge.

/s/ Duane E. Buckert

Subscribed and sworn to before me this
19th day of June 1976.

/s/ Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U.S. Department of the Interior

COPY

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Jay M. Calderwood, Victor, Idaho

_____, being duly
sworn make the following voluntary statement to Vincent L. Duran,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I am general excavation foreman for Morrison-Knudsen and Kiewit on Teton
Dam Project. I worked on the project at Newdale, Idaho, from March 1972
to present.

On Saturday, June 5, 1976, Ray Short, timekeeper, telephoned me at
home and told me there was a leak in the dam. I arrived at the dam at
11:30 a.m. I went directly on top of dam. I saw a hole about 20 feet
in circumference, about 15 feet from right abutment and about 2/3rds
way up from bottom.

There was a large amount of water, muddy and washing the hole bigger all
the time. I thought then we could not stop the water and the dam would
go. I jumped on a dozer on top of dam, worked on pushing riprap into
the whirlpool, which was on the upstream side about 12 feet to 14 feet
in water near the right abutment, not far out. The whirlpool was about
20 feet to 30 feet in circumference and 5 feet to 6 feet in depth. It
continued to get larger.

I pulled the dozer back to southside and within two minutes the top fell
in. This was about 11:50 a.m. or thereabouts. When I looked at my watch,
it was 12:00 noon and the dam had collapsed about 5 to 10 minutes before.

I have carefully read the foregoing statement, consisting of 1-1/2
pages and declare it to be true and correct.

/s/ Jay Calderwood

Subscribed and sworn to before me
this 19th day of June 1976.

/s/ Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U.S. Department of the Interior

COPY

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Roy C. Cline, Kit Circle #22
St. Anthony, Idaho, being duly
sworn make the following voluntary statement to Vincent L. Duran,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I am employed as master mechanic at Morrison-Knudsen-Kiewit on the Teton
Dam Project. I have been on the project since January 1972.

On Saturday, June 5, 1972, at about 10:30 a.m., Duane Buckert telephoned
me and told me there was a leak. I arrived at 11:00 a.m. and went
directly to the powerhouse. I saw a stream of water from right abutment
about 3/4th way up the dam. Volume was equivalent to what would run out
of a 10-inch pipe. It appeared clear at the time. I proceeded to make
a roadway behind power in preparation to opening the river outlet. I
did this and then moved a truck. At about 11:30 a.m., I looked at the
dam from powerhouse and saw the two dozers had disappeared, the hole was
big, and a large volume of muddy water. I cannot estimate the size of
hole or volume of water. I moved crane and other equipment to top on
southside. When I reached the top I saw the final collapse of the dam
top. I would estimate I arrived at top and saw collapse at about 11:55 a.m.
I proceeded to office and arrived at noon.

I felt the collapse was imminent at about 11:30 a.m. when dozers were
gone, and I began leaving the powerhouse hole.

I left the area shortly after 12:00 noon.

I have carefully read the foregoing statement, consisting of 2 pages
and declare it to be true and correct.

/s/ Roy C. Cline

Subscribed and sworn to before me
this 19th day of June 1976.

/s/ Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U.S. Department of the Interior

COPY

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, David O. Daley, 330 W. 8th St., Space 6,
St. Anthony, Idaho, being duly

sworn make the following voluntary statement to Vincent L. Duran,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to obtain this statement.

I am employed as equipment operator with Morrison-Knudsen-Kiewit on Teton Dam Project, Newdale, Idaho. I have worked there since March 15, 1972.

On Saturday, June 5, 1976, at 10:10 a.m., I got a call asking me to come to the dam. The timekeeper called and did not give me details. I arrived at 10:25 a.m. I stopped at office briefly and then went on to dam. I saw leak on north side within 15 to 20 feet of abutment and about 100 to 150 feet from top of dam. A small stream of water was flowing, but could not see if water muddy. From that point on the hole got bigger and more water flowed. Water definitely muddy.

I would guess Bellegante and I lost our dozers in the flow of water on the downstream face at about 11:15 a.m. The two of us went up to the top of dam and I operated a Gibbons and Reed dozer trying to fill in the whirlpool on the upstream reservoir side of the dam.

The whirlpool was about 30 feet out into water and about 20 feet in circumference. The pool was rather close to the north wall. I operated one dozer about one-half hour before we pulled them out. We got dozers on top of dam and headed toward southside of the dam. This was about 11:45 a.m. or possibly a little later. As we were driving off the dam the top caved in.

I never believed the dam was going to collapse until the last minute when we pulled the dozers off. I cannot give specific or estimated time of collapse. I have heard it was 11:57 a.m.

After the collapse I watched the water go down the river a short while and then left for home. I did not get involved with onlookers.

I have carefully read the foregoing statement, consisting of 2
pages and declare it to be true and correct.

/s/ David Owen Daley

Subscribed and sworn to before me
this 19th day of June 1976.

/s/ Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U.S. Department of the Interior

We worked on crowd control as much as we could. Also number of cars that came to the dam.

I have carefully read the foregoing statement, consisting of 2½ pages and declare it to be true and correct.

/s/ L. L. Payne

Subscribed and sworn to before me
this 19th day of June 1976.

/s/ Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U.S. Department of the Interior

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Vincent M. Poxleitner, Jr., P.O. Box 211
Tetan, Idaho, being duly

sworn make the following voluntary statement to Vincent L. Duran,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to obtain this statement.

I am employed by Morrison-Knudsen, Kiewit, at the Teton Dam Project. I have been employed here since June 22, 1973. I am the Project Engineer.

On Saturday, June 5, 1976, Duane Buckert, Project Manager, telephoned me at home shortly after 9:00 a.m. He asked that I come to the dam immediately because there was a leak. I arrived within 15 minutes at the office. As I came through gate saw water running out of downstream face of dam. The leak was on right-hand side of dam just off the abutment at about 5150 to 5200 elevation. The water was turbid from what I could see. The volume was the equivalent of what you see coming out of 12" pipe.

Buckert was moving a tractor across top of dam and I followed him out on top of dam. This was about 10:00 a.m. The flow had not changed much and was turbid. We had another dozer on its way to work on downface. Talked to Robison at his office, then Buckert at powerhouse, then back up to top of dam.

By the time I got to top of dam, whirlpool had developed on upstream side of dam. I cannot give times. The whirlpool about 25 feet from upstream face of dam and 75 feet from right abutment. About 3- $\frac{1}{2}$ feet to 4 feet in diameter. Stayed constant for awhile. There were two Gibbons and Reed dozers pushing riprap into hole. I did not feel the dam was going to callapse. I thought everything was salvageable. I was working to change operation of the dozers on upper side to build a ramp in order that trucks could bring in material.

The dozers on downface were having trouble. The TD 15 was tied to the eight, which was nosed into the hole. I went to get another dozer to help them and by the time I got turned around they were gone.

I would estimate it was about 11:00 a.m. shortly after dozers were gone. I was on top of dam. Two dozers still working on upstream face. Did not pay attention to whirlpool.

Shortly thereafter I moved two pickup trucks off the dam. At about 11:30 a.m., I would estimate, I called Buckert and told him dam going pretty fast and to have Bureau of Reclamation get people out downstream.

About 10 or 15 minutes later we pulled the dozers off the upstream face of dam. They went to southside and I went to northside of dam. Within one minute or one and a half minutes the dam collapsed. At 11:55 a.m. the dam collapsed. I looked at my watch when this happened. I went downstream within two or three minutes to help people. I did not return until about 1:00 p.m.

I have carefully read the foregoing statement, consisting of 3 pages and declare it to be true and correct.

/s/ V. M. Foxleitner, Jr.

Subscribed and sworn to before me
this 19th day of June, 1976.

/s/ Vincent L. Duran, Special Agent

Vincent L. Duran, Special Agent
U.S. Department of the Interior

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Barry W. Roberts, Kit Circle No. 1,
St. Anthony, Idaho, being duly

sworn make the following voluntary statement to Vincent L. Duran,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to obtain this statement.

I am employed as office engineer of Morrison-Knudsen, Kiewit Teton Dam Project, Newdale, Idaho. I have been on the project since December 1973.

On June 5, 1976, I came to the office shortly after 9:00 a.m. on personal business. Ringel, Robison and Aberle followed me through the gate and proceeded across the dam. As I came in I noticed the downstream right side of dam was wet. This was at the slope change and against the right abutment. I cannot estimate the volume.

Shortly before 10:00 a.m., Robison requested of Buckert assistance in getting river outlet operational and dozers to work on the downstream slope. For the next half hour I was in the warehouse.

At about 10:45 a.m. I was in the powerhouse ~~area~~^{area} to get opening of river outlet operational. At this time there was considerable water coming through the dam. I cannot estimate the volume. There was no chasm. The leakage area was considerably larger than when I arrived. I did some work in the power house area and at about 11:30 a.m. everyone at the powerhouse decided to evacuate. I thought at this time the dam was going to break..

On the way up several people stopped on the south ridge. Water flowing and there was a small bridge on the top of the dam on right side. I proceeded to visitors overlook and by the time I arrived the dam had collapsed. I estimated the collapse at 11:45 a.m. This was estimated because I had no watch with me.

I did not see anything on the upstream face of dam. Everything I did was on downstream side.

I do not recall seeing the dozers working on downstream face or their loss.

I have carefully read the foregoing statement, consisting of 2-1/4 pages and declare it to be true and correct.

/s/ Barry W. Roberts

Subscribed and sworn to before me this
19th day of June, 1976.

/s/ Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U.S. Department of the Interior

COPY

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Donald D. Trupp, P.O. Box 3,
Newdale, Idaho, being duly

sworn make the following voluntary statement to Vincent L. Duran,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I am employed as medical supervisor with Morrison-Knudsen and Kiewit,
Teton Dam Project, Newdale, Idaho. I began working on the project
April 19, 1972.

On Saturday, June 5, 1976, at about 10:30 a.m. I was approaching the
dam on the upper south rim road and saw water leaking through the
dam on the downstream side, north side and approximately 1/4 to 1/3 down
from top and close to the side. The hole was four to six feet in
diameter with muddy water flowing. I went to the first aid office on
the project and at about 10:35 a.m. telephoned my wife. I stayed in
trailer and Morrison-Knudsen office the rest of the time. I saw the
flow gradually increase and saw the dozers working on downstream side
of dam. I saw them having problems.

I did not see the whirlpool activity on the upstream side of the dam.
I saw the dam collapse, but cannot estimate the time. I only saw the
progressive increasing of the leakage.

At about 11:30 a.m., I telephoned relatives in Wilford, Idaho, and told
them they had better be ready for danger, because I thought the dam might
collapse.

I recall at eight minutes to 12:00 noon, by my watch, several of us put
out the alarm and the dam collapsed very shortly after this.

I have carefully read the foregoing statement, consisting of 2 pages
and declare it to be true and correct.

/s/ Donald D. Trupp

Subscribed and sworn to before me
this 19th day of June 1976.

/s/ Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U.S. Department of the Interior

WITNESS STATEMENTS BY OTHERS

Henry L. Bauer
Box 173
Teton City, Idaho

Dave Christensen
1420 Benton St., Apt. 1
Idaho Falls, Idaho

Ted V. Gould
455 N. South W.,
St. Anthony, Idaho

Richard B. Howe
Rexburg, Idaho

John F. Lee
276 N. First E.
Rexburg, Idaho

Eunice J. Olson
223 North 4th East
St. Anthony, Idaho

Mr. Lynn Schwendiman
Mrs. Lee Ann Schwendeman
Rt. 1, Box 122
St. Anthony, Idaho

COPY

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Henry L. Bauer, Box 173, Teton City, Idaho
_____, being duly

sworn make the following voluntary statement to Vincent L. Duran,
who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I am retired.

I farmed on northside of Teton River where the dam built for 30 years. My
farm was upstream of where the dam ultimately built. I always thought the dam
would be beneficial.

On Saturday, June 5, 1976, I stopped on the northside of dam approximately
1/2 mile upstream. ~~This was approximately 1/2 mile upstream.~~ This was
between 10:30 and 11:00 a.m. I looked over the reservoir for about 20
to 30 minutes. The water was very calm, there was no wind. The reservoir
1,000 feet wide at this point.

I decided to go down to dam. Time approximately 11:15 a.m. to 11:30 a.m.
I saw a truck dump material on the upper face of the dam as I approached. I
noticed a whirlpool 8 feet across against abutment and face of dam. Large
commotion and muddy water. Water away from whirlpool was semi-clear. Then a
large part of time - 20 feet wide and 20 feet high sluffed off into whirlpool --
one big chunk. This created extra commotion in whirlpool and boiled up more.
In a matter of one minute the top section of the dam dropped and the dam
had collapsed. I never looked at my watch and am not sure of the time of
collapse.

I did not see dozers working on the upper side at the whirlpool area.

I talked to a man in pea green pickup truck--he said to get out of area and
warn everyone I could. I first stopped at Ken Remington potato farm, warned him,
and continued to warn others.

I saw no fishermen in the reservoir when I made observations. I saw no other
people on canyon.

No earthquake or tremor--no water ripple as a result.

I have carefully read the foregoing statement, consisting of 2 pages and declare it to be true and correct.

/s/ Henry L. Bauer

Subscribed and sworn to before me
this 23rd day of June 1976.

/s/ Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U.S. Department of the Interior

/s/ Betty J. Foyes, Special Agent
Betty J. Foyes, Special Agent
U.S. Department of the Interior

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Dave Christensen, 1420 Benton St., Apt. 1, Idaho Falls, Idaho,
being duly sworn make the following voluntary statement to Ivan L.

Kestner, who has identified himself to me as a Special Agent of the
U. S. Department of the Interior. No threats or promises have been
made to obtain this statement.

I am a Receiving Foreman at the Idaho Supreme Company Plant, Firth,
Idaho. On June 5, 1976, my parents, wife, and children and I, visited
the Teton Dam at approximately 10:00 a.m. and remained 10 or 15
minutes. Our observations were made solely from the observation
platform at the dam, and we did not view the reservoir or the reservoir
side of the dam. Upon arrival we saw a muddy stream coming from the
mountain wall adjacent to the far or north end of the dam. We could
see the muddy water mingling at the bottom of the dam with the com-
paratively clear water flowing through the dam outlet. This stream
originated at about 20 to 30 feet from the dam bottom. As we watched
we could see a free flow of water, volume unknown, but no gush of
water. Just before leaving we noticed a darker wet streak on the dam
face, starting from a point about 2 feet wide, about 30 or 40 feet
from the place where the dam joined the mountain, and very near the
top of the dam. This streak grew 15 or 20 feet wide as it reached
the bottom of the dam. When we left about 10:15 a.m. we could see no
signs of employee activity of any nature. We did see a bulldozer parked
on top of the dam.

I have carefully read the above statement and declare it to be true and
correct.

/s/ David Wayne Christensen

Subscribed and sworn to this
23rd day of June 1976.

/s/ Ivan L. Kestner
Ivan L. Kestner, Special Agent
U.S. Department of the Interior

I was back at the dam at about 5:30 p.m. and saw water running out of abutment on the south side right where the dam abutted against canyon wall.

I have carefully read the foregoing statement, consisting of 2½ pages and declare it to be true and correct.

/s/ Ted W. Gould

Subscribed and sworn to before me
this 23rd day of June 1976.

/s/ Vincent L. Duran, Special Agent
Vincent L. Duran, Special Agent
U.S. Department of the Interior

Lloyd Hopkins, 56 N. Second W., Rexburg, Idaho

Employed as Supervisory Electrician by Wismer and Becker at Teton Dam Project, Newdale, Idaho.

Hopkins said during the period 6:00 p.m. to 7:30 p.m. on Friday, June 4, 1976, he was at the dam around the power house area, which is located on the south or left side at the downstream foot of the dam. He said he observed the entire downstream face of the dam more than once during this period and saw no evidence of any leaks or water running anywhere on the face of the dam.

Hopkins said at about 10:00 a.m. on Saturday, June 5, 1976, Dick Cuffe, his supervisor, asked him to go to the dam because there were problems. He said he went directly to the power house and arrived at about 10:30 a.m. He said upon his arrival he saw a leak in the downstream face of the dam near the north or right abutment and below the top of the dam. He said he could not be more specific about the location nor could he estimate the volume of water. He said the water was muddy. He said he saw one dozer falling in the hole created by the leak and another dozer trying to pull it out. He said shortly after this the two dozers were washed away by the water, but he cannot estimate the time.

Hopkins said he checked the availability of electricity at the power house in order to possibly open the river outlet tunnel. He said while he was doing his work several men were in the tunnel moving equipment out in order that the tunnel could be opened.

Hopkins said that at about 11:00 he and the several other workers in the tunnel and powerhouse area decided there was eminent danger and evacuated the area. He said he went to the Bureau of Reclamation offices on the south or left side of the dam. He said when he got to the offices he saw two dozers, which were working at the top of the dam, preparing to withdraw from the top of the dam. He said he does not know what time this was, but he knows the top of the dam collapsed shortly thereafter. He said he did not see the collapse, because he was preparing to leave for Rexburg, Idaho, and do what he could to protect his home from the flood.

This is not a signed statement because Mr. Hopkins was departing for California.

I am employed as an Engineering Clerk, GS-4, Targhee National Forest. St. Anthony, Idaho, and have been so employed for three years. I have 25 years of Federal Service.

On June 5, 1976, I visited the Teton Dam in the company of my son, Dale Howard, and his wife and his three daughters. We arrived at approximately 9:30 a.m. or 9:45 a.m. and spent some time observing and taking photographs. Immediately upon arrival our attention was drawn to a stream of water beginning about one third the distance from the top of the dam, and running down the angle between the dam face and the adjoining rock wall. This was on the far or north side of the dam. I have no way of estimating the flow of water other than to say it reminded me of a small woodland stream. As we watched for about half an hour the stream grew noticeably larger and it was visibly creating a gully. We wondered whether something should not be done about this, but we saw no signs of any activity associated with the stream and concluded it was a normal phenomenon. At perhaps 10:00 a.m. or 10:15 a.m. we noted a wet spot on the dam face, slightly below the level at which the stream originated. This grew as a visible wet spot and eventually began falling in. We were on the point of leaving the dam when a large collapse into this hole occurred. We then came back to watch further. This was approximately 11:00 a.m.

Some minutes after this a small bulldozer came down the face of the dam and the operator appeared to inspect the hole in the company of a second man who walked down. This dozer then left and we saw considerable activity in terms of pickup truck movements from this time on at the dam top and nearby areas. About 11:15 a.m. a larger bulldozer arrived at

the growing fissure in the dam face and began pushing in earth and rock from below the hole. It was joined by the smaller bulldozer which began moving earth and rock to a position in which the larger dozer could push it into the fissure. In all I believe the dozers worked about 20 to 30 minutes before the earth gave way beneath the dozers and they were lost. For a few minutes an attempt was made to retrieve the larger dozer, which went into the hole first, ~~then~~^{and} by pulling it free with a chain or cable from the smaller dozer. Then both dozers were lost in the mud slide.

We remained on the observation platform adjacent to the Reclamation Administration Building until dam collapse occurred at approximately 12:00 noon. My son had been taking pictures with his Yashika camera with telephoto lens until he ran out of film just before the top of the dam collapsed. I then began taking pictures with my Instamatic camera.

My observations and that of my party were limited to the face of the dam as described above. I took no particular note of the surrounding terrain and had no opportunity to see the reservoir lake behind the dam. The stream we originally noted appeared to be clear water until it began washing away the bank and became muddy. The flow from the hole in which the bulldozers were lost was a mud flow until it became mixed water and mud.

I have read the above statement consisting in all of four typed pages,
including this page, and I declare that it is true and correct.

Robert Johnson

Subscribed and sworn to before me
this 22nd day of June, 1976.

Ivan L. Kestner

Ivan L. Kestner, Special Agent
U.S. Department of the Interior

STATE OF IDAHO)
) SS
COUNTY OF MADISON)

I, Richard B Howe, of Rexburg, Idaho
_____, being duly

sworn make the following voluntary statement to Ivan L. Kestner,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I am a reporter for the KID Radio AM and Television Station, Idaho Falls, Idaho. On the morning of June 5, 1976, the day of the Teton Dam collapse, I piloted a light aircraft near the dam, passing about three miles north and 1000 feet above the dam. I was too distant to note seepage or breaks on the dam face, but did clearly observe the reservoir lake behind the dam. No turbulence or unusual features were visible in the water or the adjacent landscape. This observation took place about 10:00 a.m.

At approximately 11:45 a.m. on the same day, June 5, I learned that a warning had been given that the Teton Dam was in danger of collapse. I immediately went to the airport at Rexburg and flew to the dam with cameraman Paul Jenkins, arriving within minutes after actual collapse of the dam. I estimate our arrival at about 12:00 noon. I began broadcasting an account of the flood, as visible from the air, and Jenkins secured the only TV film footage taken in close proximity to the time of collapse. His footage was seen on CBS Network Television that evening.

I have carefully read the foregoing statement, consisting of one page only, and I declare it to be true and correct.

(signed) Richard B. Howe

Subscribed and sworn to before me this
22nd day of June, 1976.

(signed) Ivan L. Kestner
Ivan L. Kestner, Special Agent
U.S. Department of the Interior

STATE OF IDAHO)
) SS
COUNTY OF MADISON')

I, John F. Lee, 276 N. First E., Rexburg, Idaho, being duly sworn make the following voluntary statement to Vincent L. Duran, who has identified himself to me as a Special Agent of the U. S. Department of the Interior. No threats or promises have been made to obtain this statement.

I am self-employed.

On Saturday, June 5, 1976, I leaving house to go fishing when my daughter called and told me heard dam breaking. I told her would stop by the dam and look. As I drove into visitors overlook on south rim at about 11:40 a.m. there was a hole in north side of dam about 3/4 way up on downstream side. The hole appeared to be 30 feet in diameter. I could not see water, but dirt was caving in from all sides. Small chunks like scoop shovels. Still could see white gravel at floor of canyon and muddy water running. My brother Ore E. Lee, who was with me, commented that the dam going. The chunks of earth falling were now as big as a pickup. No water visible-looked air pressure blowing out from canyon wall. I looked bottom of canyon now a fuel tank going toward power house going upstream. Then large chunks of dirt, size of house falling in. The increase in size of chunks happened in about 30 seconds. Then water came over north rim of dam top and left area. This about 11:55 a.m. I did not look at watch. As I leaving Don Ellis, KKKX, came in to broadcast and I listened to his Broadcast as I heading home.

All the action at the north canyon wall on downstream side. Appeared water coming out wall at first.

Water hit Rexburg at my home at 2:32 p.m.

I have carefully read the foregoing statement, consisting of 2 pages and declare it to be true and correct.

/s/ John F. Lee

Subscribed and sworn to before me
this 24th day of June, 1976.

/s/ Vincent L. Duran
Vincent L. Duran, Special Agent
U. S. Department of the Interior

STATE OF IDAHO)
) SS
COUNTY OF FREMONT)

I, Eunice J. Olson, 223 North 4th East
St. Anthony, Idaho, being duly

sworn make the following voluntary statement to Ivan L. Kestner,

who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I am the Resource Clerk, GS-5, Targhee National Forest, St. Anthony, Idaho
(U.S.Forest Service) and reside at St. Anthony, Idaho.

On the morning of June 5, 1976 I visited the Teton Dam with two guests,
Ms. Myrtle Worfolk and Miss Heather Chapman, both residents of Griffith
Australia. We arrived at the dam at approximately 10:30 a.m. and upon reaching
the observation platform found that two bulldozers were beginning work
on the visible face of the dam at a point where a mud leak appeared to have
developed. At that time the flow from the fissure had a lava-like appearance
and seemed to consist solely of mud. It was absorbed into the dam face to
a large degree. We watched as the bulldozer operators attempted to scrape
earth and rocks into the fissure. At approximately 11:00 a.m. I became aware
that the hole was growing in an accelerated way and the two bulldozers were
in danger. Within a very short time the dozers were lost and the operators
scrambled to safety. We continued to watch until approximately 12:00 noon
when total collapse occurred. I never had opportunity to look at the reservoir
lake and I did not observe any other leaks or fissures other than that dealt
with by the two bulldozers. Just prior to actual collapse Project Engineer
Robison caused us to move back from the observation platform for safety.

Ms. Worfolk and Miss Chapman each had cameras and took pictures of the collapse
but to date I have been unable to retrieve these pictures.

I have carefully read the above statement and declare it true and correct.

Subscribed and sworn to
before me this 22nd day of
June, 1976.

Ivan L. Kestner
Ivan L. Kestner, Special Agent
U.S. Department of the Interior

Eunice J. Olson

COPY

STATE OF Idaho)
) SS
COUNTY OF Madison)

We, Mr. Lynn Schwendiman
by Mrs. Lee Ann Schwendeman, Rt. 1, Box 122,
St. Anthony, Idaho, being duly

sworn make the following voluntary statement to Betty J. Foyes,
who has identified himself to me as a Special Agent of the U. S.

Department of the Interior. No threats or promises have been made to
obtain this statement.

I, Lynn Schwendeman am employed at the Idaho Stud Mill, St. Anthony, Idaho and
on Saturday morning, June 5, Lee Ann was notified by CB radio that the Teton
Dam was leaking. This was 11:00 a.m. exactly.

We drove out to the dam, leaving our residence about 11:00 a.m. and arrived at
the visitor's observation center on the south side of the Teton Dam about 11:30
a.m. or thereabouts. We took a camera and film with us. When we got to the dam
there was just a big hole about 2/3rds of the way down on the downstream face
of the dam, about 75 to 100 feet from the north abutment. The water was
pouring out of the hole and it had more the appearance of boiling mud than
water.

The two dozers working on the downstream side of the dam had already fallen in
the hole and we could see one of them bouncing on top of the wave of water
going down river. There were no vehicles on top of the dam to the best of
our recollection.

We would estimate that the top of the dam collapsed about 11:55 a.m. As the
dam continued to collapse we were impressed by the fact that in the area about
halfway down the dam, as evidenced by the dark arc-shaped area on the south side
of the break in our picture number 4, the dirt had apparently not packed since
it came off like sand rather than in chunks. The same is true of the abutment
side of the hole. What dam fill was on the north side (canyon side) of
the dam went fastest. There was no indication that there was any breakage on
the abutment wall itself. It looked like a natural canyon wall. It
looked like all that went was just the fill part.

We had no impression of earthquakes or tremors, just the roar of the water.
We took Polaroid pictures, seven in number of the hole in the dam and the dam
collapsing. The No. 4 picture mentioned above is one of the 7. We wish to
retain the originals at the suggestion of Senator Richard Egbert (State
Senator from Driggs).

We have read the above statement consisting of two and one-quarter pages and declare it to be true and correct to the best of our knowledge and belief.

/s/ Lynn Schwendiman
Lynn Schwendiman

/s/ Lee Ann Schwendiman
Lee Ann Schwendiman

Subscribed and sworn to before
me this 23rd day of June 1976.

/s/ Betty J. Foyes

Betty J. Foyes, Special Agent
U.S. Department of the Interior

LAW ENFORCEMENT OFFICIALS - Notification Of Dam Collapse.

Ford Smith, Sheriff of Madison County (County seat - Rexburg) advised in a telephone interview of June 21, 1976, that the Teton Dam was located on the joint boundary of Madison and Fremont Counties. He said he was advised by his dispatcher of the threatened dam collapse at a time he (Smith) recalls as 10:50 a.m., June 5, 1976. He said the dispatcher called him immediately after receiving telephone notification from someone at the dam. He said that in the excitement generated by the call, the call from the dam was not logged officially by the dispatcher, and calls in general were not logged for sometime thereafter. Sheriff Smith said he did not immediately accept the warning as valid, but he concluded that the matter was too serious not to act on the call and he began telephoning everyone he knew in the potential flood path, starting with a citizen residing one and one-half miles from the dam. He said he believes it was 11:40 or 11:45 p.m. that he was told the dam was actually gone. He said none of his officers reached the dam site prior to the collapse but individual officers had driven as far as the village of Teton, warning households as they went, before they were turned back by flood waters entering Rexburg.

Blair K. Siefert, Chief of Police, Rexburg, Idaho, advised that his office, like the Sheriff's, made no official record of notice of the impending dam collapse. He said he was on a fishing trip and near Felt, Idaho about 25 miles above the Teton Dam, when he learned that the dam had collapsed or was on the point of collapsing. He said he "drove like hell" to return to Rexburg, arriving at 1:45 p.m., a short time before flood waters reached the town.

Thomas F. Stegelmeier, Sheriff of Fremont County (Countyseat - St. Anthony) advised on June 22, 1976, that his office officially logged a warning from the dam of pending collapse as of 10:43 a.m., June 5, 1976. He said he immediately telephoned the Project Engineer, Robert Robison, at the dam and confirmed that Robison wanted persons living below the dam warning of the danger of collapse. Stegelmeier said he telephoned Ted Austin of Radio Station KIGO who also placed a call to Robison. He said Austin and Deputy Sheriff Craig Reinhart then left in the same vehicle for the dam, but it is his understanding that the dam had collapsed or was in the final stages of collapse before their vehicle reached the dam. He said there were false radio accounts that St. Anthony was wiped out by flood, but in actual fact the flood was diverted by the terrain and did not damage property in St. Anthony.

Ivan L. Kestner, Special Agent

APPENDIX D

FINITE ELEMENT ANALYSES

**APPENDIX D
HYDRAULIC FRACTURING AND ITS POSSIBLE ROLE
IN THE TETON DAM FAILURE**

by

H. Bolton Seed, T.M. Leps, J.M. Duncan and R.E. Bieber

INTRODUCTION

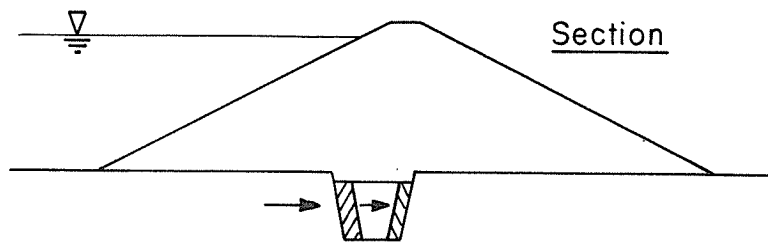
In recent years, cracking leading to excessive loss of drill water in the cores of a number of embankment dams has been attributed to the phenomenon of hydraulic fracturing; that is, a condition leading to the creation and propagation of a thin physical separation in a soil whenever the hydraulic pressure exerted on a surface of the soil exceeds the sum of the total normal stress on that surface and the tensile strength of the soil. A similar condition has also been suspected of occurring in the cores of several embankment dams due to reservoir water pressures. This has usually been the case in compressible cores of dams with more rigid outer shells, where the tendency for the core to settle or compress more than the shells results in a major reduction in stresses within the core. As a result, water pressures may exceed the sum of the normal stresses and tensile strength of the soil on certain planes within such zones of reduced stress, and cracking may develop along these planes.

To date there does not seem to have been any case reported where similar hydraulic fracturing has occurred as a result of construction of a steep-walled key trench although the conditions required to produce hydraulic fracturing are as well-developed for this type of construction (compressible fill adjacent to relatively rigid rock) as they are for the cores of rockfill dams (compressible impervious soil adjacent to relatively stiffer rockfill). Accordingly the possibility of hydraulic fracturing developing in the key trench of Teton Dam was considered to merit serious consideration, and detailed studies have been conducted to investigate this possibility.

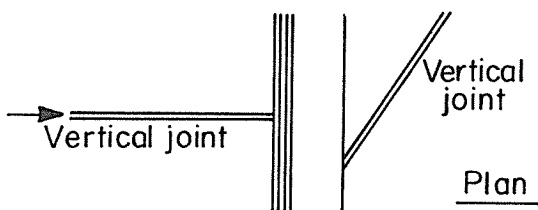
The general hypothesis whereby failure could have occurred as a result of internal erosion due to leakage through cracks in the key-trench fill caused by hydraulic fracturing is illustrated schematically in Fig. 1. If the grout curtain were fully or highly effective, the highly pervious nature of the upstream rock along vertical and horizontal joints would lead to a condition of essentially full hydrostatic pressures developing along some zones of the upstream face of the key trench. Even if the cutoff allowed some seepage under the key trench, high water pressures might still develop against the upstream face of the key-trench fill. As shown in Fig. 1, step 1, these pressures could cause fracturing of the fill where it came in contact with the joints. The resulting cracks would tend to be along the minor principal planes and would propagate longitudinally along the wall of the trench, permitting water to have access to the wall over a considerable length of the key trench.

In a coincident or second step (step 2 in Fig. 1) the water pressures thus developed would tend to produce multiple fractures along transverse planes with low normal stresses acting on them due to the arching action of the fill over the soil in the key trench. This would provide access for the water to the downstream face of the key trench.

Once this stage was reached, further fracturing could occur along minor principal planes for soil elements adjacent to the downstream wall of the key trench, again permitting the water to flow longitudinally until it found a convenient egress through open joints in the downstream rock. Erosion along the resulting flow path would ultimately lead to a piping failure of the embankment as discussed in a later section.

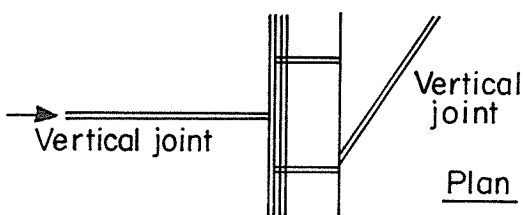


Section



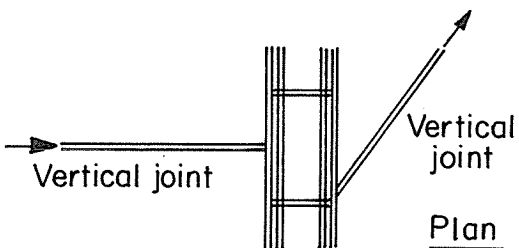
Plan

1. Flow in vertical upstream joint to face of key trench followed by longitudinal hydraulic fracturing of fill near face of trench allowing water to spread along face



Plan

2. Transverse hydraulic fracturing producing multiple fractures through soil in key trench and giving water access to downstream side of trench (may occur before Step 1)



Plan

3. Longitudinal hydraulic fracturing of soil near downstream face of key trench allowing water to spread along face and find egress through open downstream joints in rock

FIG. 1

SCHMATIC DIAGRAMS SHOWING DEVELOPMENT OF HYDRAULIC FRACTURING AND FLOW OF WATER THROUGH KEY TRENCHES

Analysis for Predicting the Possibility of Hydraulic Fracturing.

In many cases where hydraulic fracturing is believed to have occurred, its development resulted by accident during drilling or monitoring operations. In recent years experimental and analytical studies have been developed for investigating the possibility of its occurrence. Experimental studies include laboratory tests on large models, which clearly showed that high water pressures induced in vertical holes could produce observable extensive fracturing in earth materials, and field bore hole tests where high pressures induced by filling the hole with water led to fracturing at the bottom of the hole and an initially rapid loss of water from the hole. Analytical studies have involved studies of the stress conditions causing fracturing at the bottom of drill holes and the stress conditions in the shell and core materials in embankment dams. These latter studies, accomplished by means of the finite element method of analysis, have shown that this procedure has the capability to show where zones of low pressure will occur in the cores of embankment dams and thus where hydraulic fracturing might be anticipated. It has been used in design studies of such dams in the past few years.

It should be recognized that the use of finite element analyses to predict stress conditions in cores and key trench materials in this way requires the use of the most sophisticated analysis techniques and even then they should desirably be used in conjunction with some types of field test program to provide some check on the validity of the calculations. Furthermore, potential errors in the results would suggest that they are more useful as a guide to judgment than as an absolute indication of stress conditions.

The best method of stress analysis of this type is one which determines the stresses on the basis of a reasonable representation of the non-linear stress-strain relationships for the construction materials and follows a step-by-step sequence representative of the construction sequence for the embankment under consideration. Such features are embodied in the finite element computer program ISBILD which was developed at the University of California, Berkeley (Ozawa and Duncan, 1973). For some of the analyses described in this appendix, Bieber (1976) developed a computer program which employs the same analysis procedures and stress-strain relationships as ISBILD. Results from Bieber's program were compared with results from ISBILD for a simple problem to insure that the new program would produce results which conform to those from ISBILD in all essential respects.

The computer program ISBILD employs hyperbolic stress-strain relationships which model several important aspects of the stress-strain behavior of soils, including (1) nonlinearity, or decreasing modulus with increasing strain, (2) stress-dependency, or increasing stiffness and strength with increasing confining pressure, and (3) realistic variations of Poisson's ratio with strain and confining pressure. The parameters employed in the hyperbolic stress-strain relationships are listed in Table 1, together with descriptions of their physical significance and explanations of their roles in finite element analyses; a more complete description of these parameters is contained in a recent report by Wong and Duncan (1974).

Using this procedure, two types of analyses may be performed – a total stress analysis using undrained stress-strain parameters, or an effective stress analysis using drained stress-strain parameters. Both approaches have limitations. For example, an effective stress analysis may be used, incorporating drained stress-strain parameters, to evaluate the effective stress acting on any plane within the soil mass. Gradually increasing water pressures may be introduced by means of nodal point loads, representing buoyancy and seepage forces, and the resulting changes in effective stress may be calculated. If this type of analysis is performed using hyperbolic stress-strain and strength parameters determined from conventional laboratory tests conducted with positive (compressive) values of σ_3 , which are often used to represent the non-linear stress-strain properties of soils, the modulus of the soil will approach zero as the calculated value of σ_3 approaches zero simply due to the method of

stress-strain formulation. This is an inherent characteristic of the hyperbolic stress-strain relationship which employs the following approximation of the variation of initial tangent modulus with confining pressure:

$$E_i = K p_a \left(\frac{\sigma_3}{p_a} \right)^n$$

where E_i = initial tangent modulus
 K = modulus number
 p_a = atmospheric pressure
 σ_3 = minor principal effective stress
 n = modulus exponent

Because the modulus approaches zero as the effective stress is reduced, the soil tends to swell without limit and the calculated effective stress never reaches zero. The calculated effective stress therefore never becomes tensile, and the results of such analyses never indicate any likelihood for hydraulic fracturing, even for the most critical conditions where hydraulic fracturing would inevitably occur.

Alternatively a total stress analysis may be used to assess the possibility of hydraulic fracturing. Using this approach the total stresses acting on any plane within the soil mass are evaluated and hydraulic fracturing is presumed to occur whenever the water pressure exceeds the sum of the total normal stress and the tensile strength of the soil; alternatively the procedure may be visualized as one in which the effective stress on any plane is determined by subtracting the water pressure from the computed total stress. If the resulting effective stress is tensile and equal to or greater than the tensile strength of the soil, the inference is drawn that hydraulic fracturing would occur under the conditions analyzed. This total stress procedure is overly-conservative because it ignores the tendency of the soil to swell as the effective stresses on any plane are reduced; in effect the method assumes no tendency to swell during a reduction in stress equal to the water pressure. Furthermore the effects of creep movements in the soil under sustained loads are not considered. These limitations can be compensated for in the analysis by using a somewhat higher value of Poisson's ratio (expressed by the parameter G , see Table 1) than that which actually applies for the soil involved, and a range of other soil parameters. The best method to determine the appropriate value of G is to conduct field fracturing tests and compare the stresses required to cause fracturing with those computed using different values of G in the analysis. The value giving best agreement with field conditions is the value most likely to give the best assessment of the overall distribution of stresses and thereby the hydraulic fracturing potential from the analytical studies. Accordingly this procedure was selected for use in the present study.

An added complication in the case of Teton Dam arises from the possibility that the soil in some sections of the key trench may have become saturated by seepage. Stations of primary interest range from about 12+50 to 15+50 and while it seems reasonably clear that the key trench fill for stations at 12+70 and 13+70 would not have time to become saturated as the water level in the reservoir rose above the base of the trench at these locations, the same cannot be said for the key trench fill at Sta. 15+00. At this location the base of the trench is at El. 5105 and the water level stood in the reservoir at about El. 5160 for a period of 4 months prior to April 1, 1976. Thereafter it rose to El. 5300 in a further period of 2 months.

Whether or not these water head conditions would be sufficient to cause water to seep into and saturate the key trench fill depends on the permeability of the fill. Unfortunately data on this

Available Results from Tests on Teton Dam Materials

Parameter	Symbol	Role in Analysis	U-U Test	U-U Test	Drained Test	Drained Test	Drained Test
			Remoulded ML, at Compaction Water Content	Remoulded CL, at Compaction Water Content	Remoulded ML-CL at Compaction Water Content	Undisturbed at Field Water Content Boring 5255-D-4	Undisturbed Saturated Boring 5250-IR-U3
Moist unit weight	γ_m	Stress values are proportional to unit weight (moist, saturated or buoyant depending on zone)	114 lb/ft ³	120 lb/ft ³	120 lb/ft ³	119 lb/ft ³	120 lb/ft ³
Cohesion intercept	c	Together determine how strength varies with confining pressure	1630 psf	1670 psf	0	750 psf	900 psf
Friction angle	ϕ		32.5°	30°	35.4°	38.5°	29.5°
Modulus number	K	Together determine how initial tangent modulus varies with confining pressure	770	1200	250	530	430
Modulus exponent	n		0.32	-0.22	0.37	0.64	0.15
Failure ratio	R_f	Relates value of hyperbolic asymptote to compressive strength	0.77	0.81	0.65	0.81	0.77
Poisson's ratio at $\sigma_3 = 1$ atmosphere, and zero strain	G	Together determine how Poisson's ratio varies with σ_3 and strain	0.28	0.20	0.36	0.24	0.33
Reduction in Poisson's ratio for 10-fold increase in σ_3	F		0.11	0.10	0.17	0.12	0.25
Increase in Poisson's ratio for 100% strain	d		5.0	3.2	3.9	3.5	1.6
Value of σ_3 at which compression due to wetting begins	σ_{3t}	Together determine how much compression is caused by wetting	These parameters require special types of tests for their determination. The values used in the analyses were estimated on the basis of the amount of compression due to wetting of Teton Dam soils in tests done at Berkeley, and available data for other soils.				
Volumetric strain for change in σ_3 equal to 1 atmosphere	β						

D-5

TABLE I

STRENGTH AND STRESS-STRAIN PARAMETERS
USED IN FINITE ELEMENT ANALYSES

property of the Zone 1 fill are highly variable. Preconstruction values determined by the Bureau of Reclamation show an average value of 0.25×10^{-6} cm/sec and 146 tests on record samples taken during construction tend to confirm this result, showing values ranging from 0.02×10^{-6} to 3.6×10^{-6} cm/sec. On the other hand, horizontal permeability tests on 3 undisturbed samples taken during construction gave permeability coefficients ranging from 3×10^{-6} to 13×10^{-6} cm/sec while four similar tests at the University of California on samples taken from one block of soil from the key trench fill gave values ranging from 0.3×10^{-6} to 4.3×10^{-6} cm/sec.

It seems reasonable to conclude from these data that the coefficient of permeability of the in-situ Zone 1 fill varies mainly from about 0.1×10^{-6} to 5×10^{-6} cm/sec.

If the average permeability were 1×10^{-6} then a simple computation would show that for a head of 55 ft, such as would exist near the bottom of the trench at Sta. 15+00 from Jan. 1 to April 1, 1976, the water would flow horizontally into the fill for a distance of only about 6 or 7 ft. At higher elevations the water penetration would be even less.

On the other hand, if the coefficient of permeability of the fill were of the order of 5×10^{-6} cm/sec, as indicated by the undisturbed sample tests, the water would penetrate into the bottom of the fill a distance of 30 to 40 ft prior to April 1, suggesting, that by June 1, the major part of the key trench fill at Sta. 15+00 could have increased in degree of saturation. This raises the possibility that in this vicinity, arching of the soil over the key trench would occur not only due to the original differential compressibility of the soil and rock at the key trench elevation, but also due to some additional tendency of the fill in the key trench to settle slightly as a result of the wetting action. Although settlement due to wetting may be very small, it can never-the-less have a pronounced effect on the stress distribution in the key trench.

Because of the uncertainty regarding the extent of wetting in the key trenches at the deeper sections, analyses of stress distribution were made for both conditions and a determination of the most likely condition was made by comparing the computed stress distribution with the results of field tests to measure the in-situ stresses at which hydraulic fracturing developed. The secondary effect of settlement due to wetting can be taken into account in a finite element analysis of stress distribution using a computer program written by Nobari and Duncan (1972) and this program was used, together with measured values of compression of the Teton Dam Zone 1 material due to wetting, to compute the stress distribution at Sta. 15+00 for the wetted key trench condition, in addition to the stress distribution for the normal fill placement condition.

The purpose of the field test program was thus two-fold: (1) to investigate whether the soils in the vicinity of Station 15+00 showed any indication of having been saturated prior to the failure and (2) to investigate the appropriate value of Poisson's ratio or G , as used in the computations, to provide computed stresses in agreement with in-situ conditions.

The value of G determined by the field tests corresponds to a sudden or short-term application of the water pressure. In a dam, the rate of application of the water pressure by a rising reservoir is much slower. The effect of the difference in rate of loading, with respect to the value of G , has not been systematically investigated, and thus represents an element of uncertainty in predictions of the potential for fracturing.

Selection of Significant Soil Characteristics.

As previously noted, the computation of the stress distribution in an embankment using the program ISBILD requires the determination of nine different soil parameters. These parameters are readily

determined from triaxial compression tests and a number of such tests were conducted for this purpose. Since the primary interest in Teton Dam centers on the Zone 1 material, testing programs were limited to this material.

Tests were performed on both laboratory-compacted samples and on undisturbed samples cut from the key trench fill after the failure occurred. The results of these tests are summarized in Table 1.

As may be seen from the data presented in this table, the test data show considerable scatter for some of the parameters involved. However, in determining the stress distribution within the Zone 1 material, the most significant of the highly variable parameters are K (the modulus number), n (the modulus exponent) and G, the factor determining the relationship between major, minor, and intermediate principal stresses.

Because of the wide scatter in these values shown by the test data, it was decided to perform a parameter study to determine the effect of the values of K and n, within the range indicated by the data, on the values of the stresses computed to develop in the Zone 1 fill. Accordingly stress analyses were made for the conditions at Stations 15+00 for the following conditions

- | | |
|---------------|----------|
| (1) K = 250; | n = 0.07 |
| (2) K = 1000, | n = 0.07 |
| (3) K = 250 | n = 0.50 |

Other parameters were maintained constant at their most likely values (e.g. $G = 0.35$; $\gamma = 117 \text{ lb/ft}^3$; $c = 1650 \text{ psf}$, $\phi = 31^\circ$; etc.). The results of these studies are shown in Figs. 2 and 3. Fig. 2 shows computed values of the major principal stress and Fig. 3 shows computed values of the minor principal stress at a number of representative points both in the key trench and throughout the Zone 1 fill. It may be seen that, in spite of the wide variations in K and n, the values of the computed stresses do not change appreciably, indicating that the stress analysis procedure is insensitive to reasonable variations in these parameters. In view of this it was considered appropriate to use representative values, based on the test data and on experience with determinations of parameters for other soils. On this basis, the following parameters were selected for use in all further analyses:

γ	= 117 lb/ft ³
c	= 1650 psf
ϕ	= 31°
K	= 470
n	= 0.12
R_f	= 0.79
F	= 0.10
d	= 4.0

The value of G was left variable at this stage pending the completion of field tests to determine the stresses at which hydraulic fracturing occurred in the field. Three such tests were conducted in the embankment and key trench fill near the left abutment at Stations 26+00 and 27+00, where the key trench sections closely resemble those at Stations 15+00 and 13+70 on the right abutment respectively.

Field Tests for Hydraulic Fracturing

Several field tests were performed to measure the water pressures required at different points in the Zone 1 section in the unfailed portion of the dam to measure the water pressure required to cause

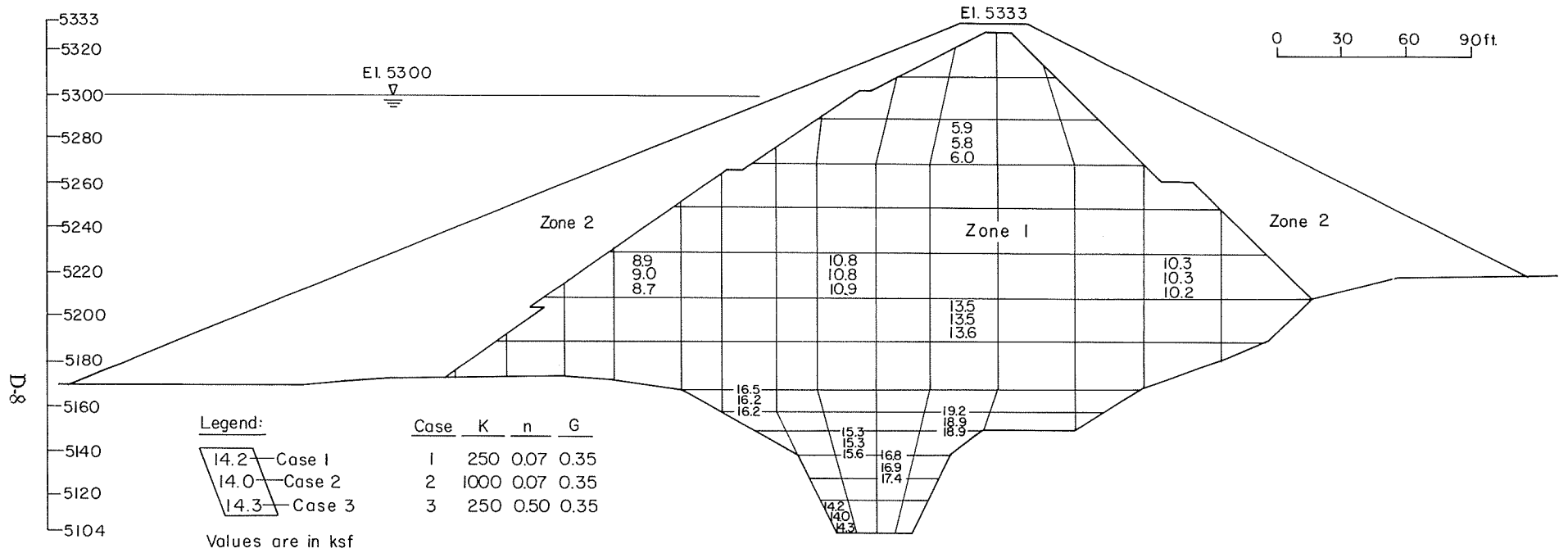


FIG. 2 EFFECT OF VARIATIONS IN PARAMETERS K AND n ON COMPUTED VALUES OF MAJOR PRINCIPAL STRESS AT STA. 15+00

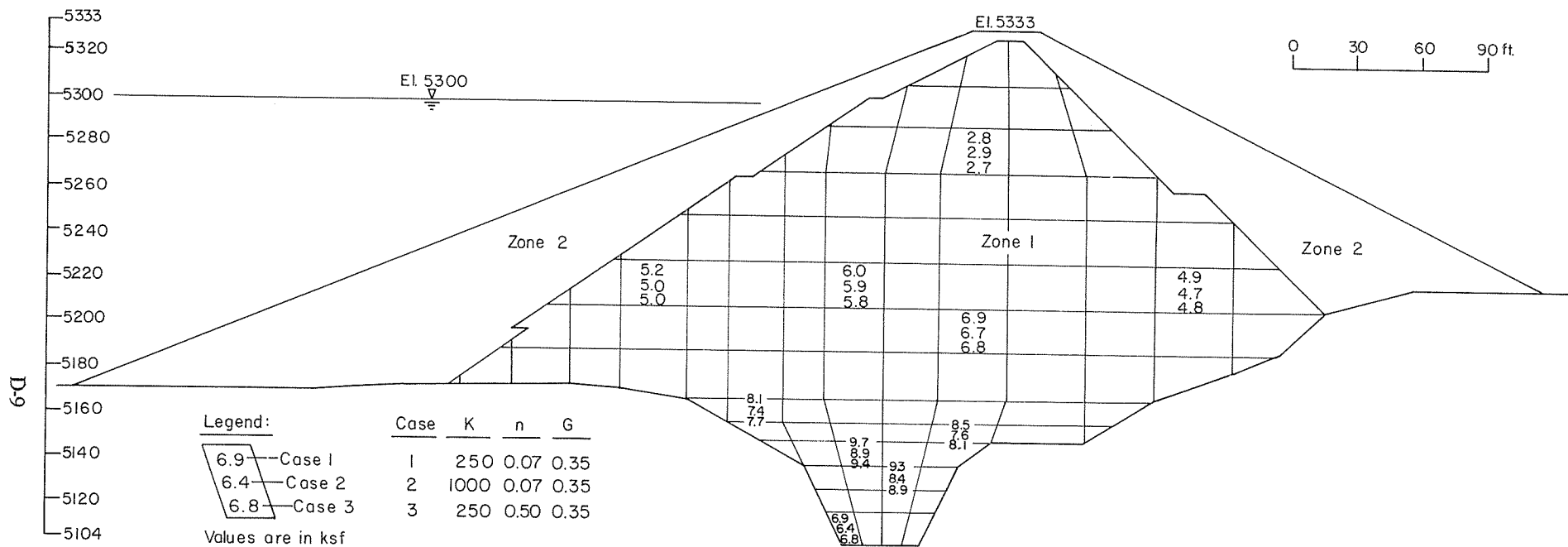


FIG.3 EFFECT OF VARIATIONS IN PARAMETERS K AND n ON COMPUTED VALUES OF MINOR PRINCIPAL STRESS IN TRANSVERSE PLANE AT STA. 15+00

hydraulic fracturing. Previous studies have shown that this water pressure should be closely equal to the sum of the minor principal stress at the point in question and the tensile strength of the soil at that point.

Sections chosen for study were Station 26+00, where the stress conditions were considered to be somewhat similar to those at Station 15+00 on the right abutment and Station 27+00, where conditions were similar to those at Station 13+70 on the right abutment.

Test No. 1 – Station 26+00

The first test was performed at Station 26+00 where fracturing was induced at El. 5210 under a water head of 101 ft, corresponding to a pressure of 6.3 ksf. The location of the test, superimposed on the cross-section at Station 15+00, is shown by point A in Figs. 4 and 5. Also shown in the figures are the computed stress conditions required to cause hydraulic fracturing in the vicinity of A for three different values of the parameter G and for the case where the key-trench fill is assumed to be unwetted (Fig. 4) and wetted (Fig. 5). It was estimated that the tensile strength of the Zone 1 fill was about 0.4 ksf for this purpose. It will be seen that, in Fig. 4 the computed stress required to induce fracturing at point A is about 6.5 ksf for $G = 0.35$, while in Fig. 5, the computed fracturing pressure is about 6.3 ksf for $G = 0.35$. Both of these computed results are in excellent agreement with the measured pressure causing fracturing in the field, but results for other values of G are significantly less favorable.

Test No. 2

The second test was performed at Station 26+00 in a depth range between Els. 5133 and 5161, and fracturing developed when the head of water acting on the soil reached an average elevation of 5293. As described in Chapter 3, it seems reasonable to believe that fracturing occurred at about El. 5147 so that the corresponding head causing fracturing would be 146 ft of water or a pressure of 9.1 ksf. The location of such a test position superimposed on the cross-section at Station 15+00 is shown by point B in Figs. 4 and 5. Also shown in the figures are the computed stress-conditions required to cause hydraulic fracturing in the vicinity of B for three different values of the parameter G and for the case where the key-trench fill is assumed to be unwetted (Fig. 4) and wetted (Fig. 5). It may be seen that, in this case also, reasonably good agreement is obtained between the measured pressure required to cause hydraulic fracturing (9.1 ksf) and the computed pressure for the case where $G = 0.35$ and the key-trench fill is assumed to be unwetted (8.8 ksf). Poorer agreement is obtained for higher and lower values of G. However the much lower values indicated for all values of G by an analysis performed for a wetted key-trench fill suggests that this type of analysis would not provide realistic results for the section under investigation, and indicates that the key-trench fill was probably not wetted before the failure. It might also be noted that the results of this test indicate the tensile strength of the fill to be of the order of 0.4 ksf (see Chapter 3).

Test No. 3

The third test was performed at Sta. 27+00 in a hole drilled to El. 5190. The hole was then filled with water to El. 5315 but no evidence of hydraulic fracturing was observed. The pressure at the bottom of the hole under this head was 7.8 ksf. The location of this test point superimposed on the cross-section at Station 13+70 is shown in Fig. 6. It may be noted that for $G = 0.35$, the corresponding value of the computed pressure required to cause hydraulic fracturing at this location is only 6.4 ksf. It seems likely, based on the results shown in Figs. 5 and 6, that a computed pressure in good agreement with the field test result might have been obtained if the analysis had been made for $G = 0.4$.

However in view of the good results obtained for Station 15+00 using $G = 0.35$ and an unwetted key-trench fill condition, together with the uncertainties necessarily introduced by other aspects of the

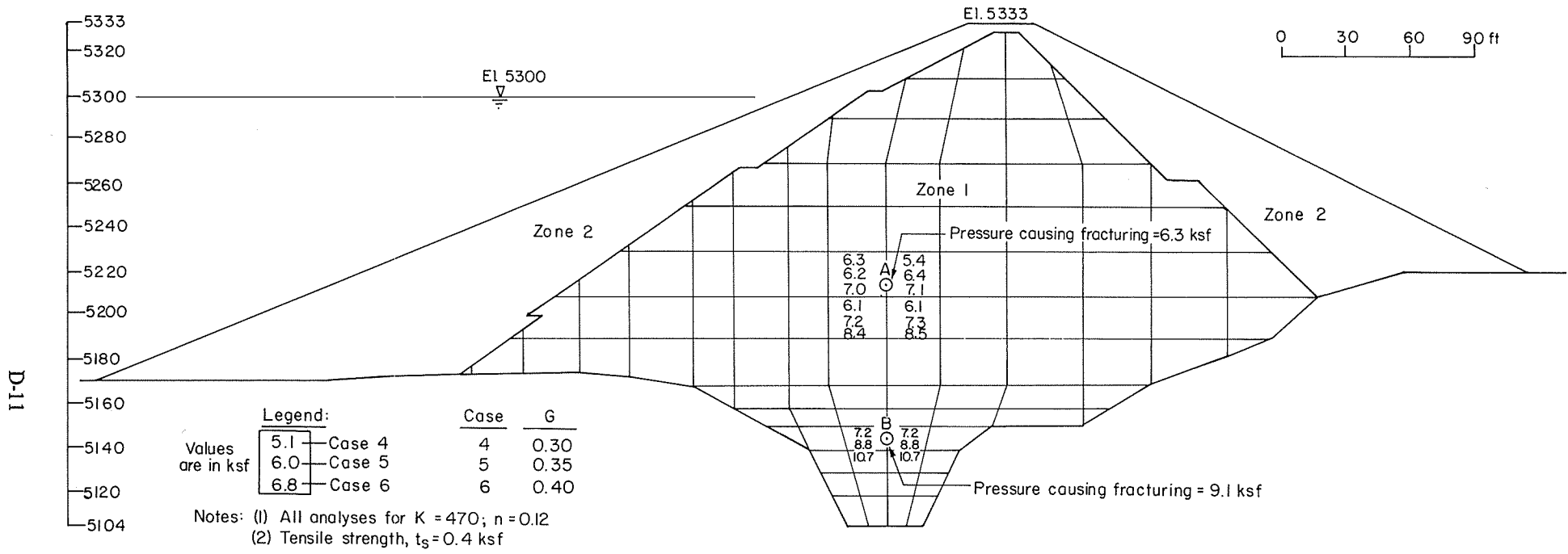


FIG. 4 EFFECT OF VARIATIONS IN PARAMETER G ON COMPUTED VALUES OF SUM OF MINOR PRINCIPAL STRESS AND TENSILE STRENGTH OF SOIL- STA.15+00 KEY TRENCH ASSUMED TO BE UNWETTED

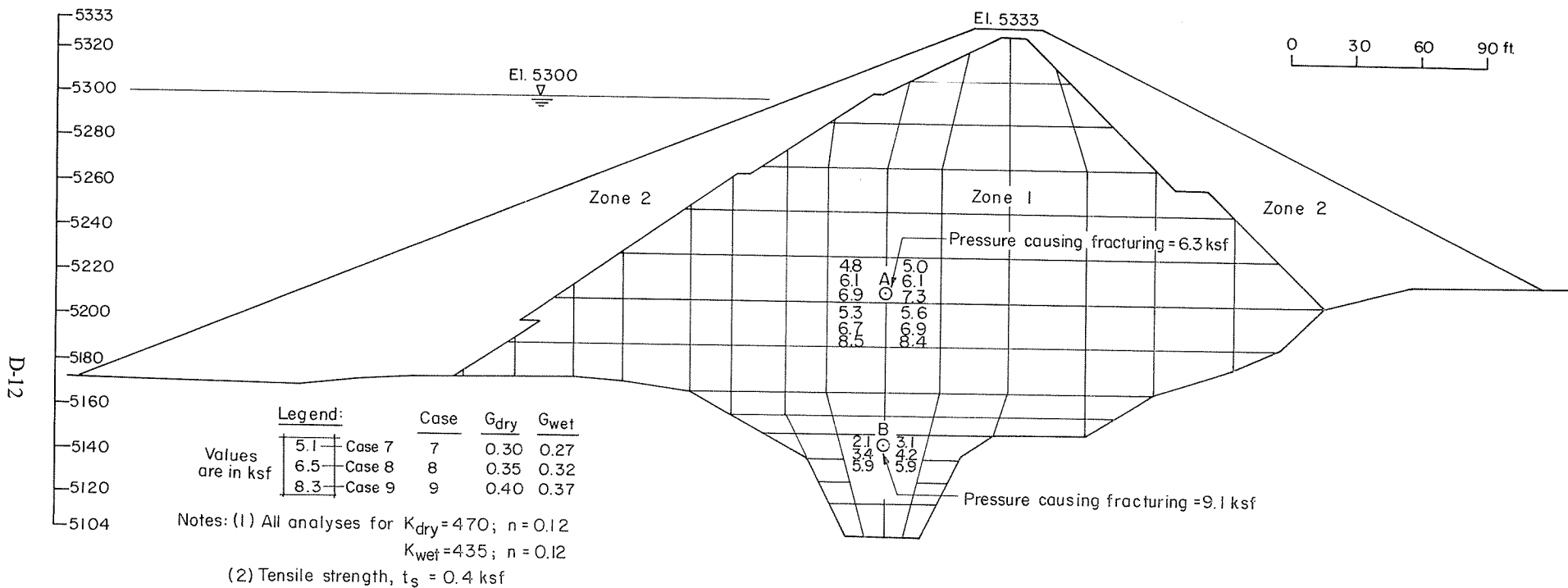


FIG. 5 EFFECT OF VARIATIONS IN PARAMETER G ON COMPUTED VALUES OF SUM OF MINOR PRINCIPAL STRESS AND TENSILE STRENGTH OF SOIL- STA. 15+00 KEY TRENCH ASSUMED TO BE WETTED

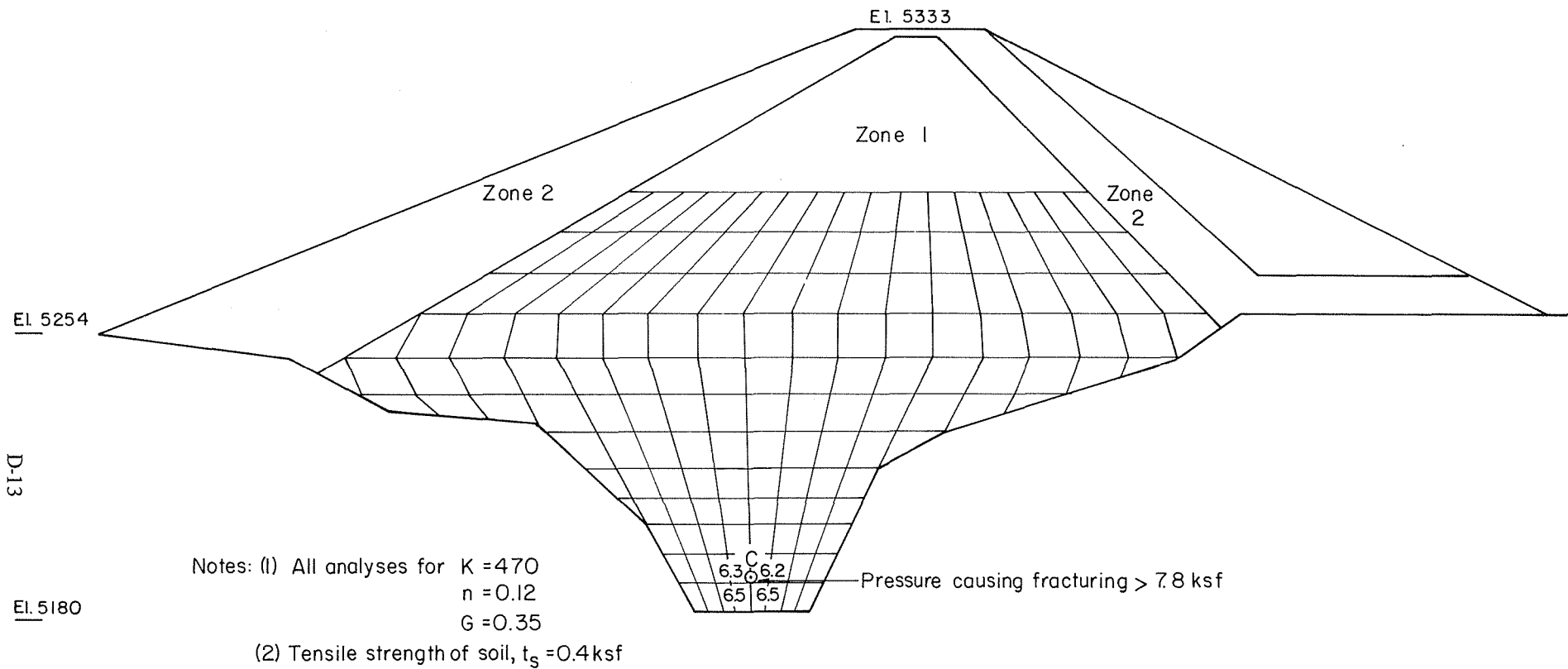


FIG.6 COMPUTED SUM OF MINOR PRINCIPAL STRESS AND TENSILE STRENGTH OF SOIL - STA.13 +70

analyses it was concluded that analyses based on these conditions would provide an adequate indication of the stress distributions in the embankment at sections of primary interest and a useful guide to the associated potential for hydraulic fracturing. Having thus established a reasonable set of analysis parameters and conditions, computations of stress conditions were then made for the embankment sections at Stas. 12+70, 13+70 and 15+00. The results of these analyses are described below.

Analysis of Section at Sta. 12+70

An idealized cross-section through the embankment at Sta. 12+70 is shown in Fig. 7. Before discussing the computed values of stresses developed throughout the embankment it is useful to note the stress conditions in a soil element adjacent to the upstream face of the key trench. Such an element is shown in Fig. 8 together with the orientations of the major and minor principal stresses. Since hydraulic fracturing is likely to occur first on the plane with the lowest value of normal stress it will always tend to be initiated on the minor principal plane, which for the element shown is inclined inwards at about 30° to the vertical. On the centerline of the trench the minor principal plane will be essentially vertical while on the downstream side of the face of the trench it will be inclined at the opposite direction to that shown in Fig. 8. Whether or not fracturing will occur on such planes depends on the relative values of the water pressure on the face of the trench and the minor principal stresses in soil elements adjacent to the wall of the trench.

A comparison of these stresses is shown in Fig. 7. Values of the minor principal stress developed in different elements of the finite element mesh are shown directly in the elements in ksf units and the hydrostatic water pressures assumed to develop in a highly jointed rock for a reservoir level of 5300 (the elevation at the time the dam failed) are shown adjacent to the elements in parentheses. It may be seen that for this section hydraulic fracturing of the type described above is only indicated for the outer rows of elements in the bottom part of the trench (shown shaded) and elements on the downstream side would only fracture if full hydrostatic pressure could develop in this area. In these elements and zones, however, the analyses would indicate the onset of hydraulic fracturing which could be expected to propagate from any point of initiation in a longitudinal direction, providing the possibility of full hydrostatic pressures developing over a substantial area near the lower part of the upstream face of the key trench. The resulting fractures are illustrated schematically in Fig. 9.

With regard to the possibility of hydraulic fracturing in the transverse direction it is necessary to compare the hydrostatic water pressures with the sum of the normal stress on the transverse section and the tensile strength of the soil as illustrated in Fig. 10. A comparison of the computed normal stress on the transverse plane with the full hydrostatic pressures is shown in Fig. 11. It may be seen that the analysis indicates that the stresses developed at all elevations in this section would be sufficient to preclude the possibility of transverse fracturing.

However with the reservoir level at El. 5300 the study would indicate that full hydrostatic water pressures could move through fractures along the upstream face, and along the downstream face, possibly finding egress through transverse fractures which might form at other sections of the embankment. This possibility is explored further below.

It is appropriate to point out at this stage that the walls of the key trench were not smooth as shown schematically in the sections used for analyses. Thus in addition to fracturing along the faces of the key trench, longitudinal movement of water might also be facilitated by zones of lower compaction underlying projections on the face, thereby compounding the conditions discussed above.

For simplicity in explanation, the grout curtain has been assumed to be fully or nearly fully impermeable. Under this condition, full reservoir pressure can reasonably be assumed to act on the

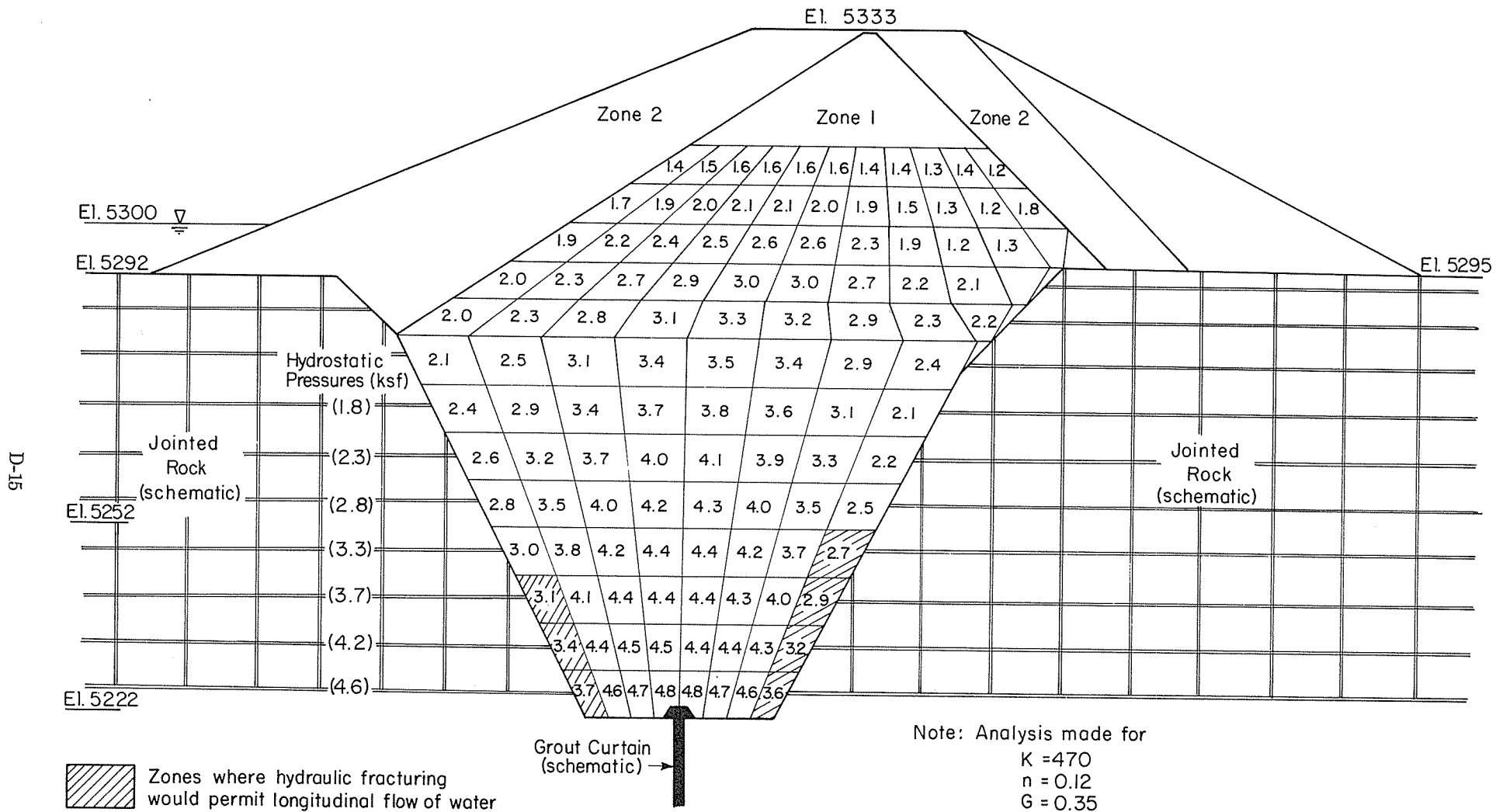


FIG. 7 COMPUTED VALUES OF MINOR PRINCIPAL STRESS
 IN PLANE OF SECTION ksf STA. 12+70

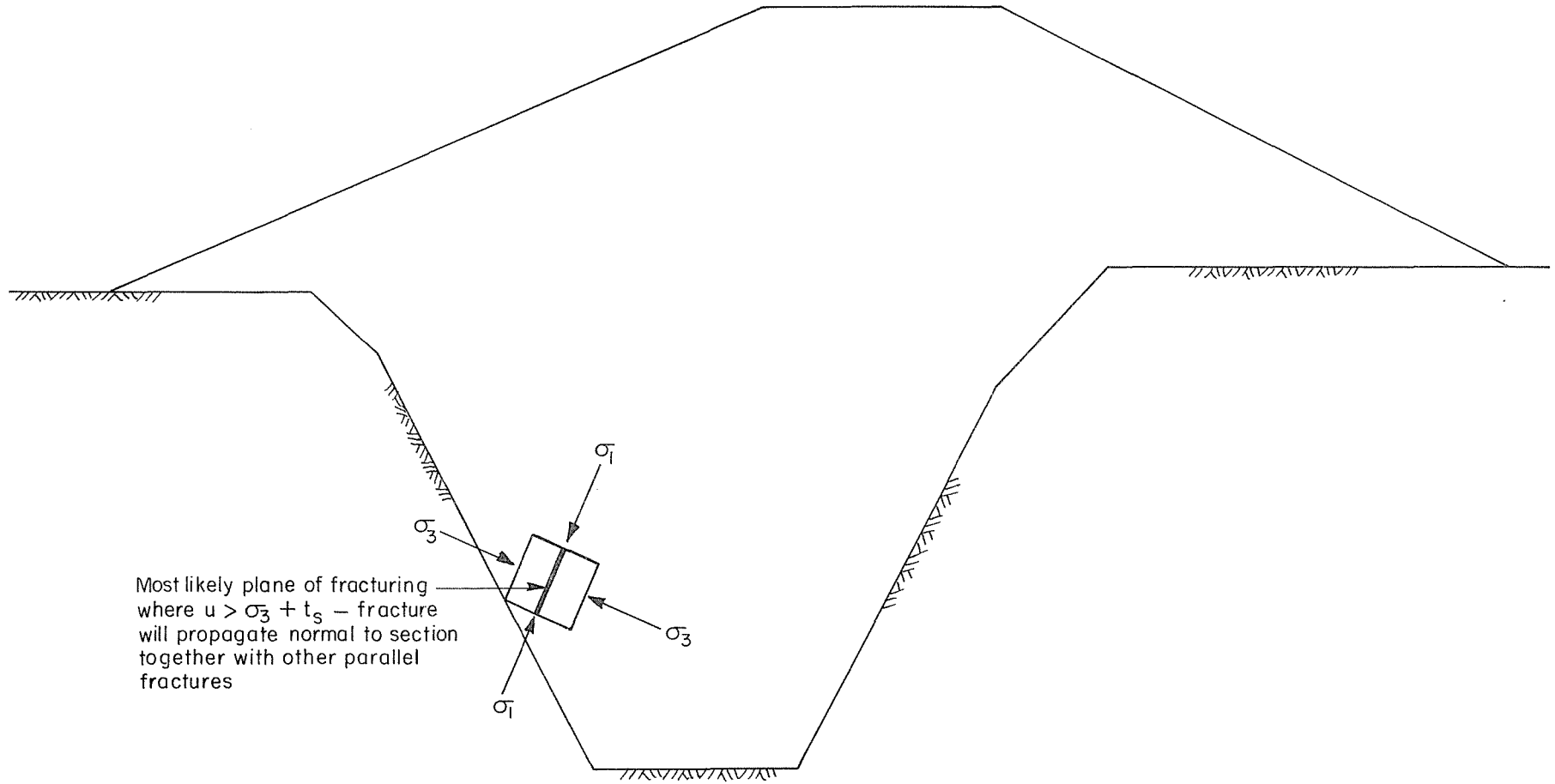


FIG. 8 DEVELOPMENT OF LONGITUDINAL FRACTURE PATTERN

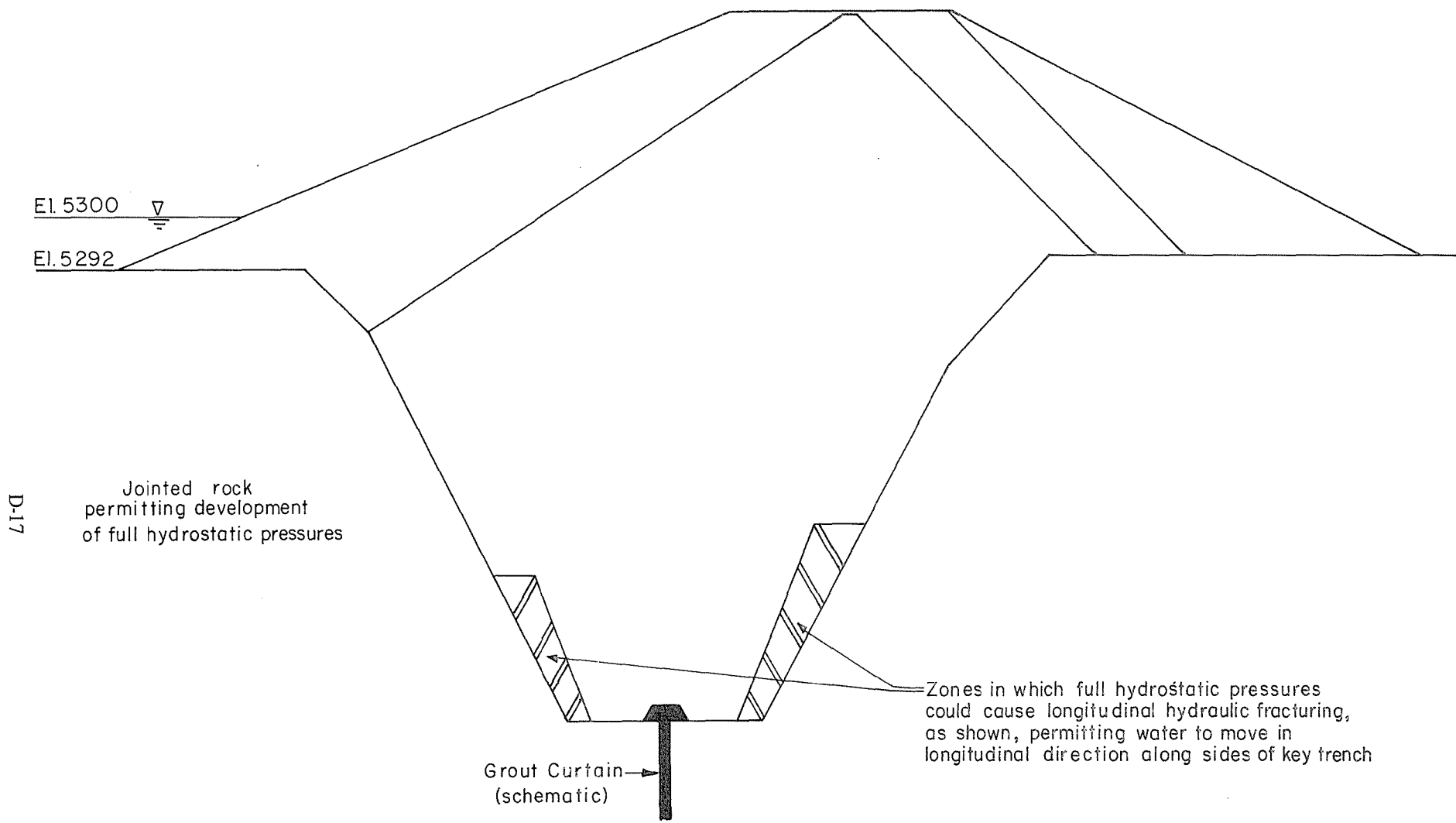
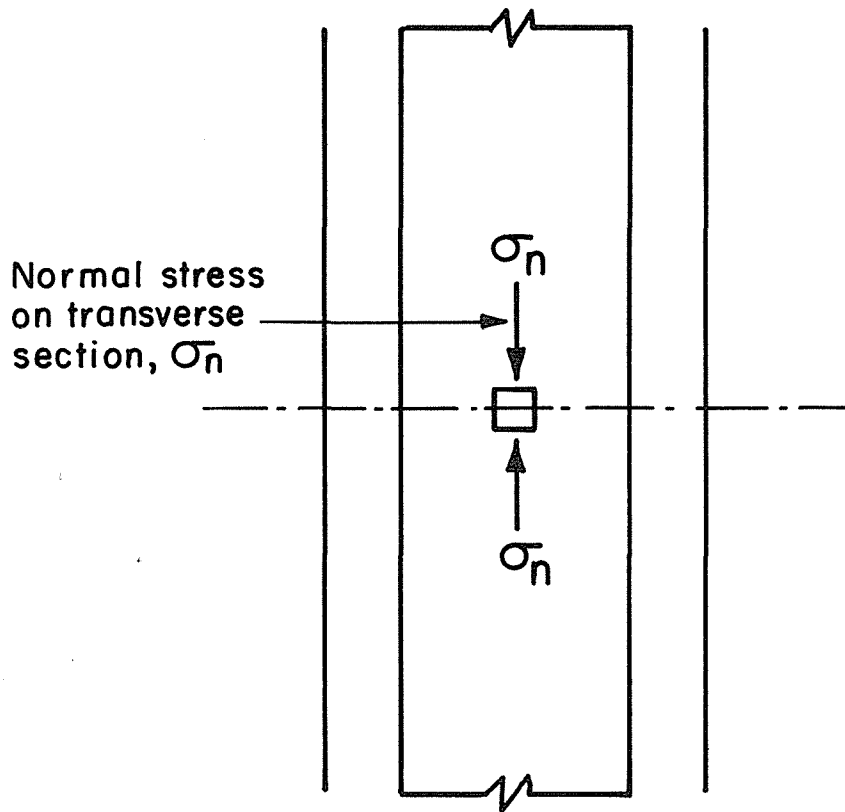
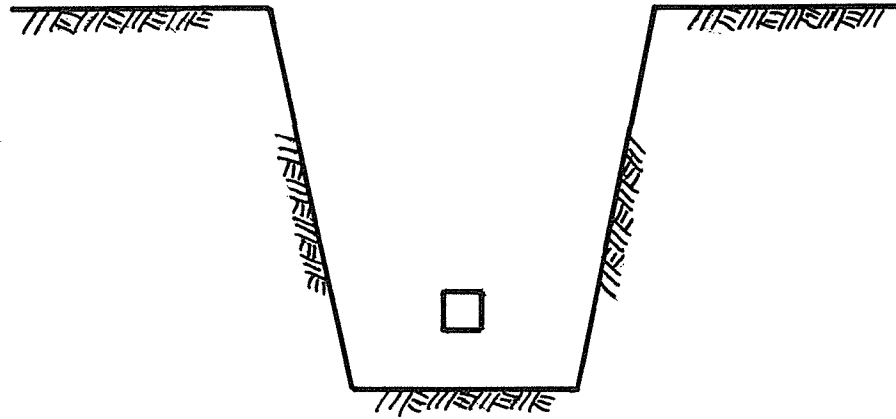


FIG. 9 ZONES OF POTENTIAL LONGITUDINAL FRACTURING AT STA. 12 + 70



Transverse fracturing occurs if $u > \sigma_n + t_s$

where: u = water pressure

t_s = tensile strength of soil

FIG. 10 MECHANISM FOR TRANSVERSE FRACTURING IN KEY TRENCH

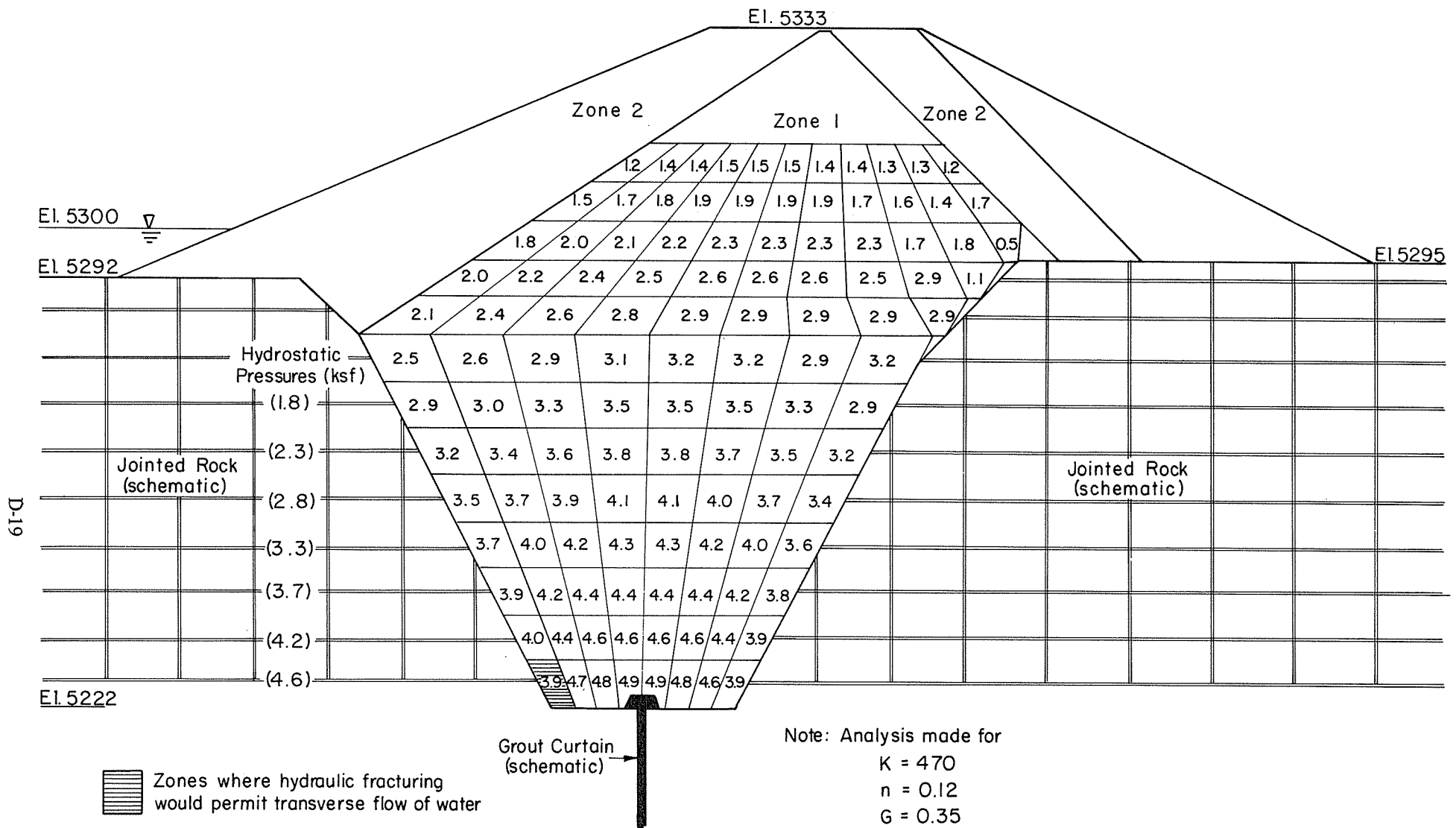


FIG. II COMPUTED VALUES OF NORMAL STRESS ON TRANSVERSE SECTION ksf STA. 12+70

upstream face of the key-trench fill. It is evident, however, that the calculated potential for hydraulic fracturing depends greatly on the actual water pressure. Since the efficiency of single-line grout curtains in rock, when determined by piezometric observations upstream and downstream of the curtains, has in reality turned out to be remarkably low, the actual water pressures are established by the conditions of flow through the foundation and curtain, and may be substantially less than full reservoir pressure. Therefore, the susceptibility to hydraulic fracturing determined by the foregoing calculations represents an upper limit.

Analysis of Section at Station 13+70

Analyses similar to those presented above, but for the embankment cross-section at Station 13+70, are shown in Figs. 12 and 13. Fig. 12 shows values of the minor principal stress at element locations throughout Zone 1, together with values of the hydrostatic water pressures in the upstream jointed rock for a reservoir level of 5300 (the level on the day of the failure). The shaded zone shows those parts of the key trench where the water pressure exceeds the sum of the minor principal stress and the estimated tensile strength of the key trench fill, and thus where inclined longitudinal fracturing as shown in Fig. 8 can be expected to occur. It may be seen that such fracturing could extend about 40 ft above the base of the key trench at this section and that longitudinal flow of water along fractures could occur all the way across the section.

Fig. 13 shows values of the normal stresses on the transverse section, together with values of the full hydrostatic pressure on the day of failure. Here it is apparent that transverse fracturing could occur to a height of about 20 ft above the base of the trench.

A combination of the two hydraulic fracture patterns shown for Sta. 13+70 would provide a continuous flow path for water from joints in the upstream rock to open joints in the downstream rock, providing a mechanism for erosion of the highly erodible Zone 1 fill.

The question might be raised whether, in fact, full hydrostatic pressures could be developed on the downstream side of the key trench fill. Until a continuous flow path developed, progressive fracturing could readily lead to the development of full hydrostatic pressures in all parts of the fracture system. Once the water found an outlet path, some loss of pressure would inevitably occur. If this loss of pressure was appreciable, the fracture might close, and if this happened flow would stop. Cessation of flow, however, would quickly lead to reestablishment of full hydrostatic pressure conditions, which would result in reopening of the crack. Thus, once a continuous seepage path had been established from upstream to downstream, it seems likely that flow would continue, perhaps on an intermittent basis in the early stages but on a continuing basis as progressive erosion developed in the key trench and later the embankment fill.

Analysis of Section at Sta. 15+00

Analysis of the stress conditions for the embankment sections at Station 15+00 are shown in Figs. 14 and 15. Fig. 14 shows values of the minor principal stress at element locations throughout Zone 1 together with values of the hydrostatic water pressures in the upstream jointed rock for a reservoir level of 5300. It is apparent that for these stress conditions, hydraulic fracturing in a longitudinal direction could at this stage extend through virtually the full area of the key trench, although very high pressures would prevent its development in the upper center part of the trench. Hydraulic fracturing would also be indicated in a substantial zone near the base of the Zone 1 material in the main body of the embankment.

Somewhat similar results are indicated in Fig. 15 which shows the distribution of normal stress on the transverse section at this station. Again the low values of lateral stress developed in the key trench

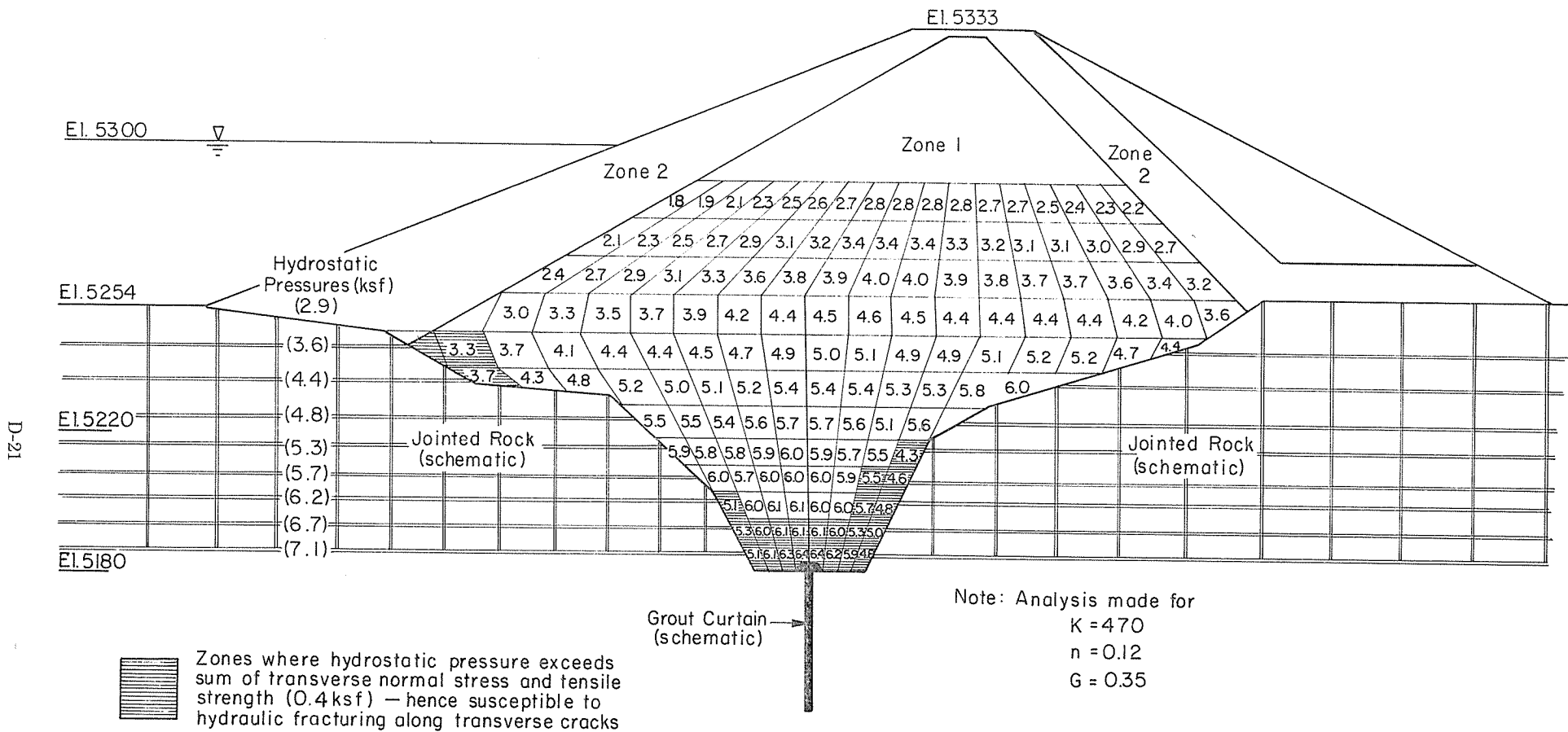


FIG. 12 COMPUTED VALUES OF MINOR PRINCIPAL STRESS IN PLANE OF SECTION STA. 13+70

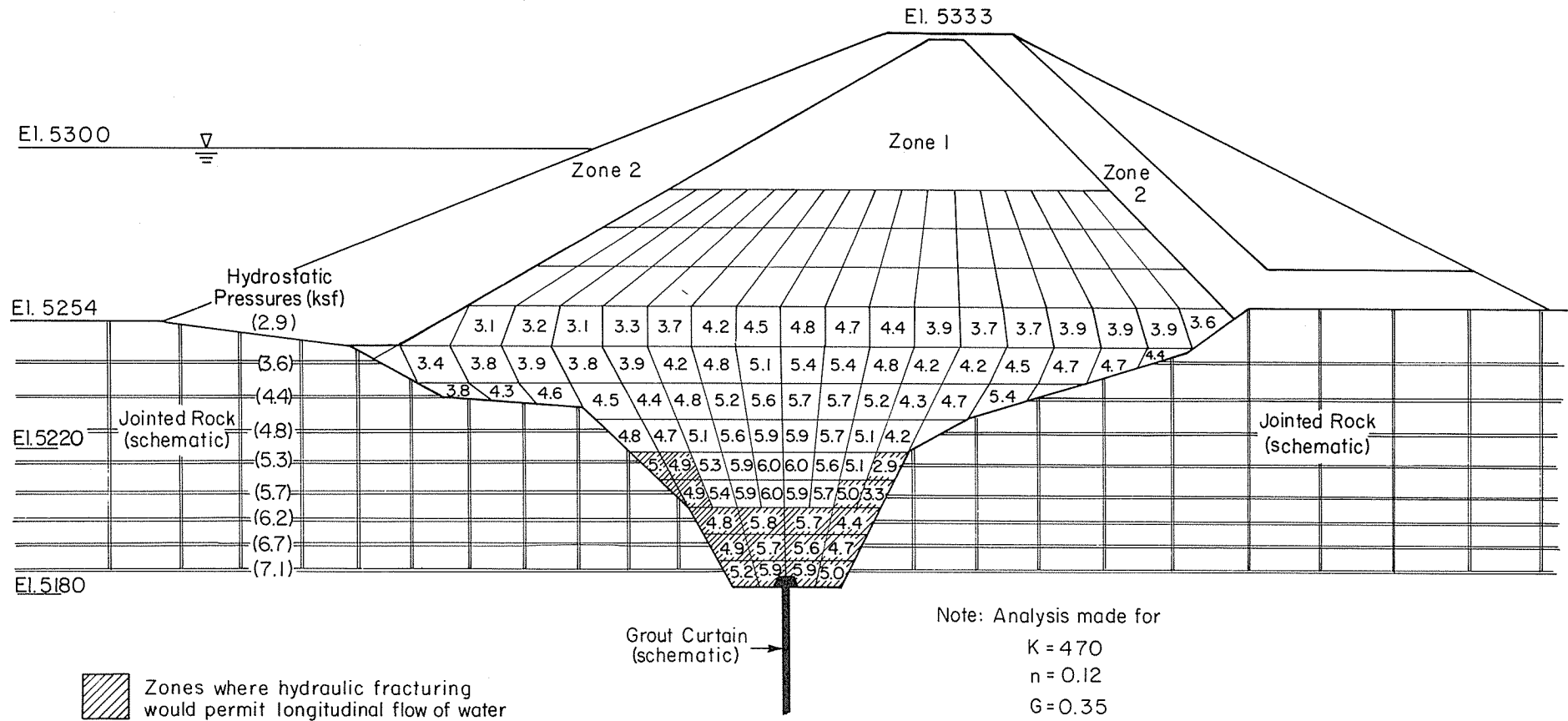


FIG.13 COMPUTED VALUES OF NORMAL STRESS ON TRANSVERSE SECTION IN Ksf STA. 13+70

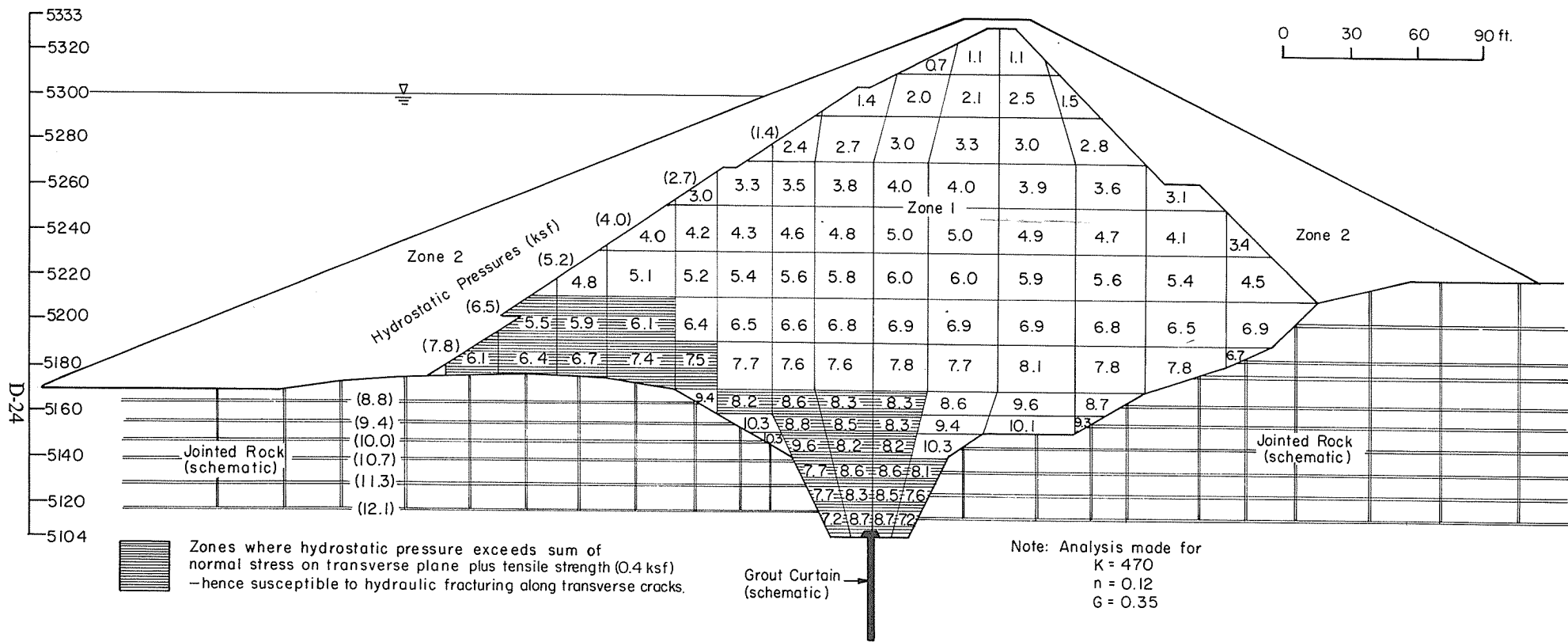


FIG. 15 COMPUTED VALUES OF NORMAL STRESS ON THE TRANSVERSE SECTION IN ksf STA. 15+00

would indicate that hydraulic fracturing could extend through the full depth of the trench except for a small zone in the upper part of the trench on the downstream side.

Summary of Results

In assessing the significance of the zones of potential hydraulic fracturing shown in Figs. 7 to 15, it should be noted that the determinations were made by comparing the stresses developed in the embankment and key trench fills with the full hydrostatic pressures in the adjacent rock on the day of failure when the reservoir elevation was 5300. On dates prior to this, the stress levels in the fill would be essentially the same, but the reservoir level and corresponding hydrostatic water pressures would be substantially lower so that the potential zones of hydraulic fracturing would be greatly reduced.

For example with the reservoir level at El. 5255 (as it was on May 20, 1976) the hydrostatic water pressures in the upstream jointed rock would only be sufficient to cause hydraulic fracturing in the bottom 10 ft of the key trench at Station 15+00 and none at all for Stas. 13+70 and 12+70. This condition is best illustrated by the longitudinal section drawn through the centerline of the key trench on the right abutment shown in Fig. 16. The analysis indicates only a very small zone in the vicinity of Sta. 15+00 where the water could move horizontally and vertically through hydraulically-induced fractures on this date, May 20, and for a reservoir level of 5255.

As the water level rose, the extent of the zone in which fracturing could occur naturally increased, but reference to Figs. 11 and 12 will show that even when the reservoir level rose to El. 5275 hydraulic fracturing would still not yet have developed at the bottom of the key trench at Station 13+70. This reservoir elevation was reached on May 25, 1976 and Fig. 17 shows the estimated extent of the zone of hydraulic fracturing in the key trench on this date.

Finally, by the time the reservoir reached El. 5300 on June 5, 1976, transverse hydraulic fracturing would become possible in the bottom section of the key trench at Station 13+70 and it would extend to a greater height at Sta. 15+00 as shown in Fig. 18. Note however that it is never likely to occur beyond about Sta. 16+00 because the key trench downslope of that station was either very shallow or non-existent, and it does not seem likely that it would develop upslope of about Station 13+20 because the stress conditions beyond that point are unfavorable to its development.

Figs. 16, 17 and 18 provide an excellent summary of the extent of the potential zone of hydraulic fracturing, as estimated from the results of the preceding analytical studies. It is interesting to note that they only indicate the development of a substantial zone of vulnerability due to this cause in the 10 days before failure actually occurred and that the location of the indicated zone of fracturing coincides closely with the zone in which piping finally developed (between about Stations 13+50 and 15+00).

While the potential for hydraulic fracturing to provide a flow path for water through the key trench is a significant aspect of any potential failure mechanism, it must be coupled with the possibility of erosion of soil and therefore the possibility of removal of eroded material through open joints in the downstream rock, at least in the early stages of failure development. Accordingly also plotted on the longitudinal sections shown in Figs. 16, 17 and 18 is the approximate location of the bottom of the open-jointed, highly pervious rhyolite in the vicinity of the key trench.

Consideration of the position of this material in conjunction with the estimated extent of the zones of hydraulic fracturing on May 20 (Fig. 16), May 25 (Fig. 17) and June 5 (Fig. 18) would seem to indicate that it was not until the reservoir elevation reached about El. 5290 on June 1 that the

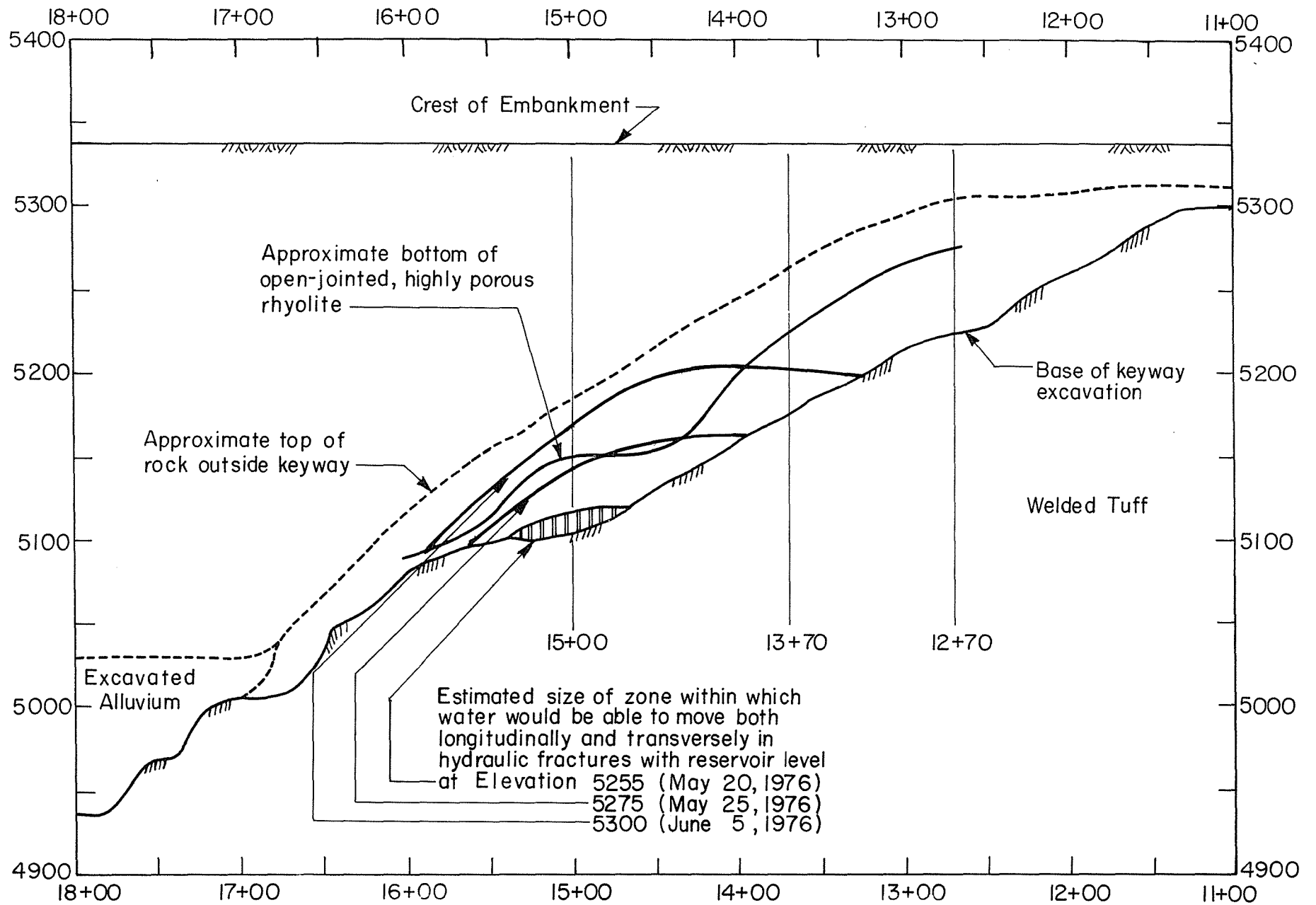


FIG. 16

LONGITUDINAL SECTION THROUGH CENTER LINE
CREST AND GROUT CAP

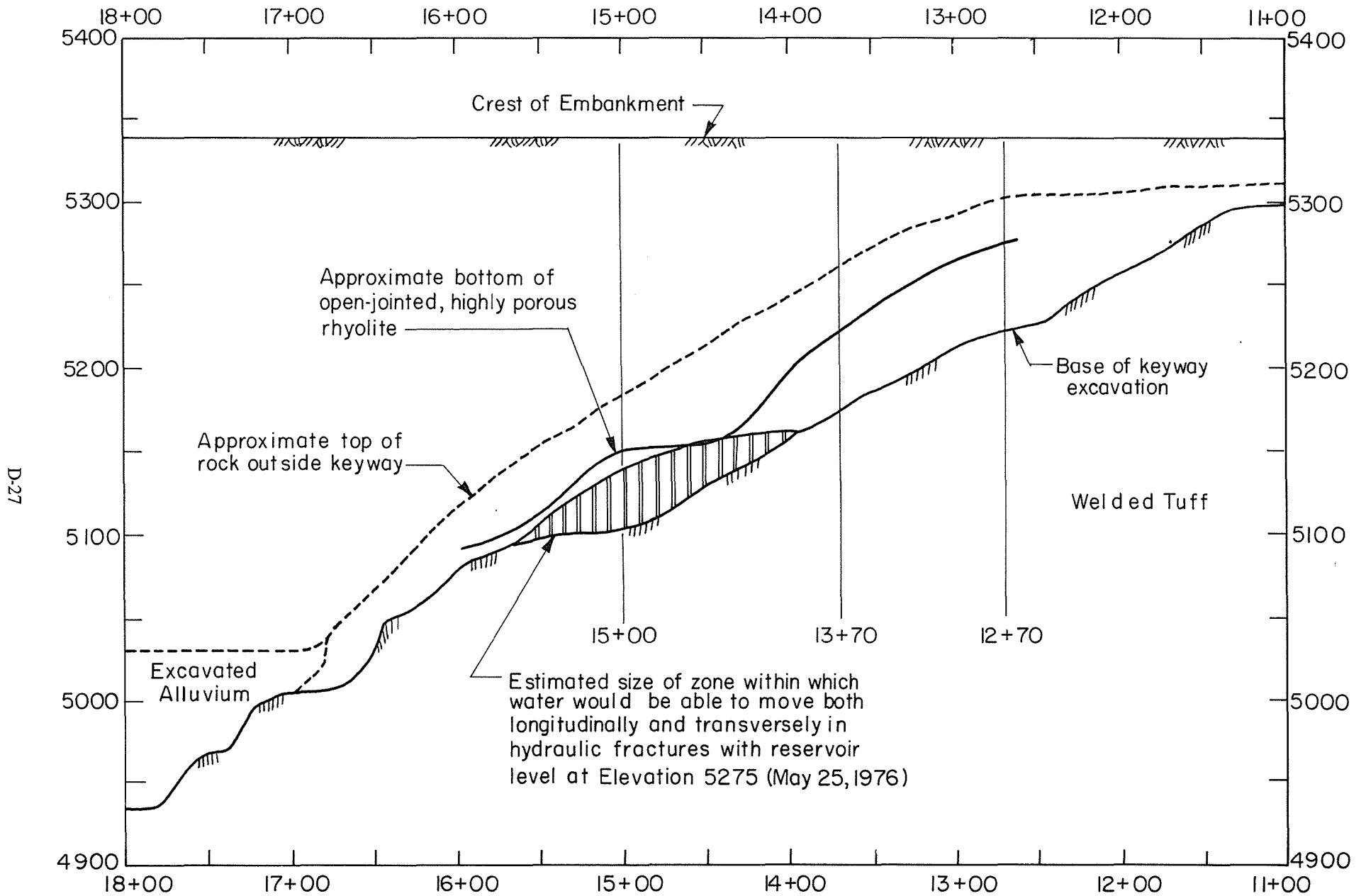


FIG.17 LONGITUDINAL SECTION THROUGH CENTER LINE
CREST AND GROUT CAP

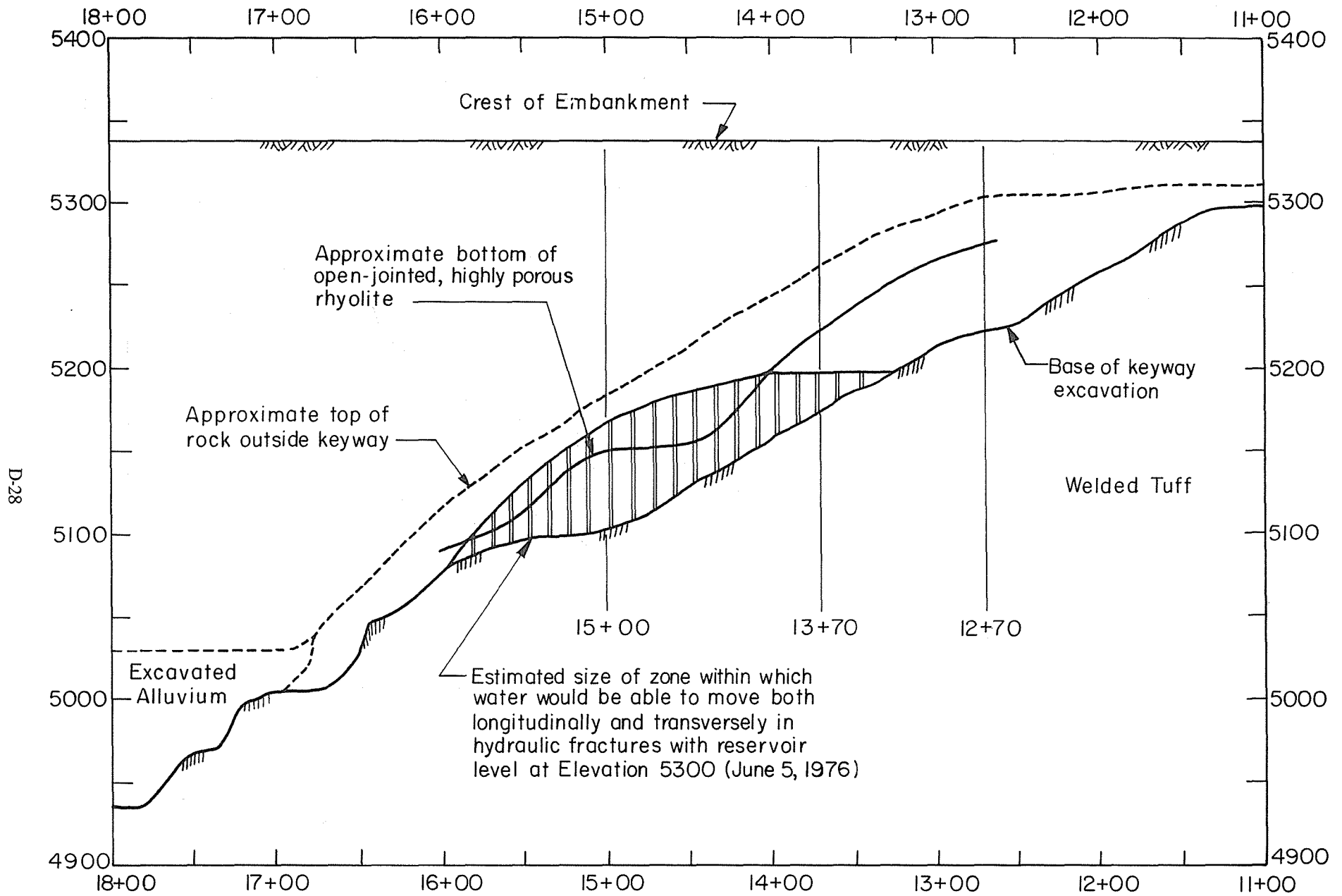


FIG.18 LONGITUDINAL SECTION THROUGH CENTER LINE
CREST AND GROUT CAP

complete flow path through highly pervious rock, through extensively fractured key trench fill and again into highly pervious rock would exist to permit the initiation of internal erosion and the mechanism which finally could lead to failure of the dam.

The remarkable coincidence of the critical zones for hydraulic fracturing, and the time at which it could develop, with the zone of failure and the time of failure would seem to lend considerable support to the hypothesis that hydraulic fracturing of the soil in the key trench may well have been a contributory cause to the failure of Teton Dam. However it should be noted that the potential zones of hydraulic fracturing would tend to be reduced if water pressures on the upstream face of the trench were substantially lowered as a result of leakage through the grout curtain. Thus the analysis presented above indicates an upper bound on the extent of hydraulic fracturing which might have occurred.

The hydraulic fracturing hypothesis presented above necessarily raises other questions concerning the dam failure. Foremost among these would have to be the question of why failure was initiated on the right abutment rather than the left. The key trench sections were remarkably similar on both sides and an analysis similar to that described in the preceding pages for stations on the left abutment of the dam would undoubtedly lead to somewhat similar results with regard to the potential for hydraulic fracturing.

In the final analysis therefore it must be considered that if hydraulic fracturing were responsible for the leakage through the key trench fill, initiation of failure on one side of the damsite rather than the other would be related to the question of minor geologic details and the fact that the joint system in the rhyolite was more extensively developed and adversely aligned to facilitate seepage and internal erosion on the right abutment than on the left. However the hypothesis would seem to indicate that if this mechanism of failure developed, given similar rock conditions in the left abutment, it would only have been a matter of time before seepage and internal erosion occurred on that side also.

Finally, it is worthy of note that, although it is assumed that hydraulic fracturing will occur in a fine-grained soil whenever the water pressure exceeds the sum of the minimum compressive stress and the tensile strength of the soil at a given point, the phenomenon is not yet fully understood and deserves research on a variety of materials under different boundary conditions and under controlled laboratory conditions. When a better physical understanding of the creation and propagation of cracks by water pressure has been achieved, the criteria for initiation of hydraulic fracturing utilized herein may require modification.

Significance of Key Trenches

The preceding discussion necessarily attaches considerable significance to the role of the key trenches in reducing the stresses in the key trench fill and thereby facilitating hydraulic fracturing and accompanying erosion. In order to further investigate the effects of the key trenches on the stress distribution and to provide a qualitative rather than a quantitative assessment of their significance, a series of studies was conducted for the conditions at Sta. 15+00 in which the vertical stresses developed in the embankment were expressed as a proportion of the total weight of overburden, for all points in the embankment. The results are expressed as contours showing the developed vertical stress as a fraction of the direct overburden pressure. Analyses were made for four conditions.

1. For the actual section at Sta 15+00 with no allowance for wetting of the Zone 1 fill in the key trench or the embankment.
2. For the section at Sta 15+00 if the key trench had not been constructed and with no allowance for wetting of the Zone 1 material.

3. For the actual section at Sta 15+00 with allowance for wetting of the Zone 1 fill to the extent indicated in Fig. 13.

4. For the sections at Sta. 15+00 if the key trench had not been constructed but the Zone 1 fill had been wetted to the extent indicated in Fig. 13.

The comparative results for analyses 1 and 2 above are shown in Fig. 19 and for analyses 3 and 4 above in Fig. 20. The effects of arching over the key trench and the considerable reduction in stresses in the key trench fill resulting from the presence of the key trench is readily apparent from these figures, confirming the fact that the use of key trenches on the sides of the abutments invited the development of arching, stress reduction and the accompanying onset of hydraulic fracturing and internal erosion.

Mechanism of Failure by Hydraulic Fracturing

The discussion presented in the preceding pages has shown clearly how the phenomenon of hydraulic fracturing could provide a continuous flow path through the key-trench fill in critical locations, if all features of the grout curtain had functioned adequately. The flow path in the early stages of its development would necessarily start in highly pervious rock, pass through fractures in the key-trench fill and then continue through highly pervious rock.

Whether the initial flow started by hydraulic fracturing or leakage in the rock just below the grout cap, 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 the 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 to speculate therefore on the manner in which this transition might have developed.

Playing a key role in this aspect of failure was undoubtedly the specific character of the joint systems in the rock in the vicinity of Station 14+00 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 plunging down to and below the base of the key trench on the upstream side of the key trench between Stas. 13+90 and 14+10. Similar but narrower joints could readily be identified at locations 10 to 20 ft on both sides of this zone.

Readily identifiable exit paths for water on the downstream side of the key trench in this vicinity could similarly be noted as follows:

(a) a limited number of open vertical joints in the relatively sound rhyolite below about El. 5200

(b) a maze of open horizontal and vertical joints in the highly fractured and jointed rhyolite between about Els. 5200 and 5240.

and (c) a 25 ft thick layer of highly pervious talus and slope wash between Els. 5240 and 5265.

Characteristically the primary open vertical joints in the downstream pervious rock angled in plan at about 45° from the dam axis towards the river, so that water entering this joint system would be expected to flow primarily in this direction until it encountered a more accessible outlet path near the face of the abutment rock, where joints were abundant in all directions.

Thus the general path of seepage and erosion, both as evidenced by the field and analytical studies and by the observed backward path of erosion towards the whirlpool during the failure itself would

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

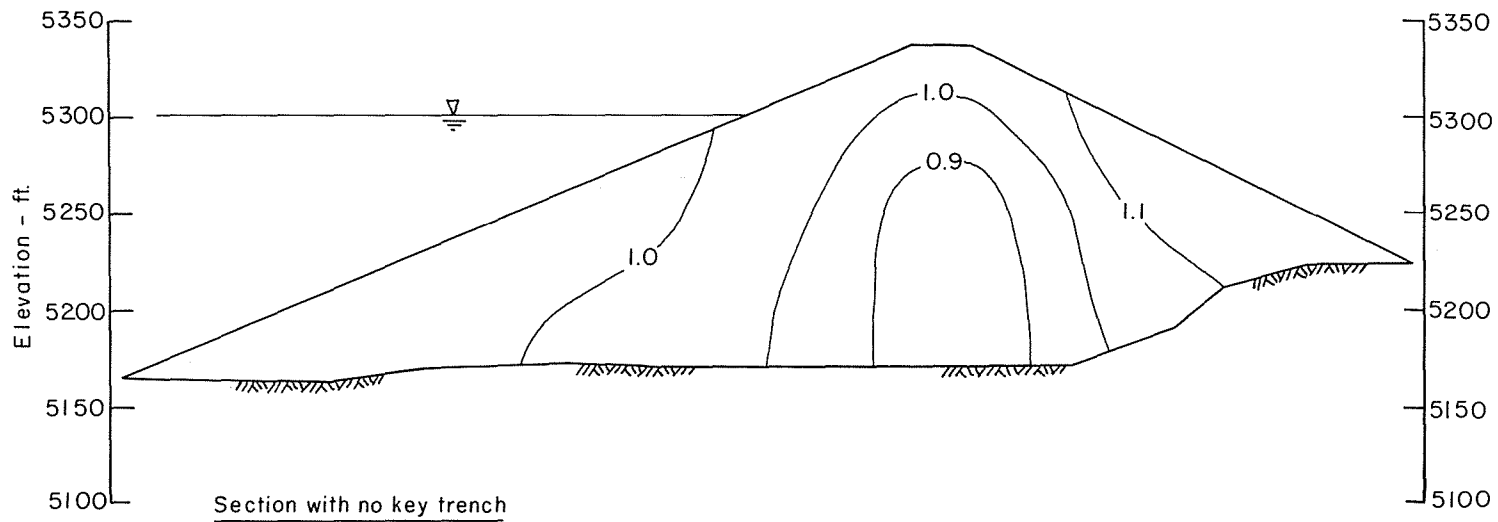
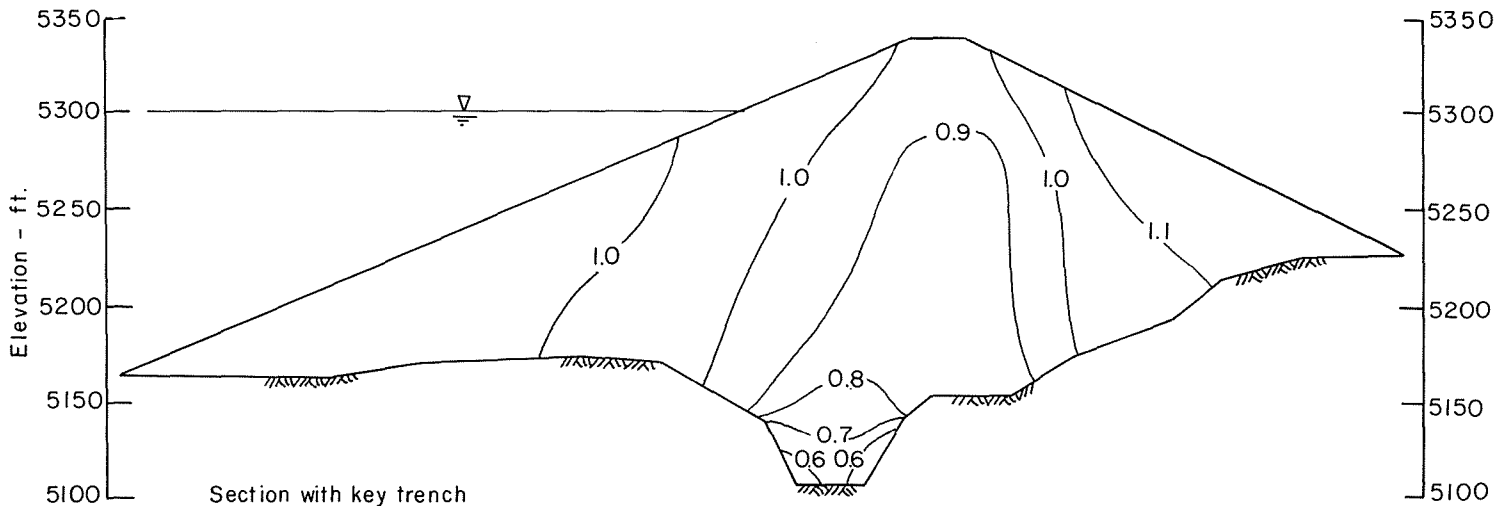


FIG. 19

CONTOURS SHOWING RATIO OF VERTICAL STRESS IN EMBANKMENT TO OVERBURDEN PRESSURE FOR CONDITIONS WITH AND WITHOUT KEY TRENCH AT STA. 15+00-BEFORE WETTING OF SOIL IN KEY TRENCH

Soil Properties

	γ (kcf)	ϕ	$\Delta\phi$	C(ksf)	K	K_{ur}	n	R_f	G	F	D	σ_{3T}	β
Before Wetting	0.117	31°	0	1.65	470	750	0.12	0.79	0.35	0.10	4.0	0	0.003
After Wetting	0.125	30°	0	150	435	650	0.12	0.75	0.32	0.10	4.0	0	0.003

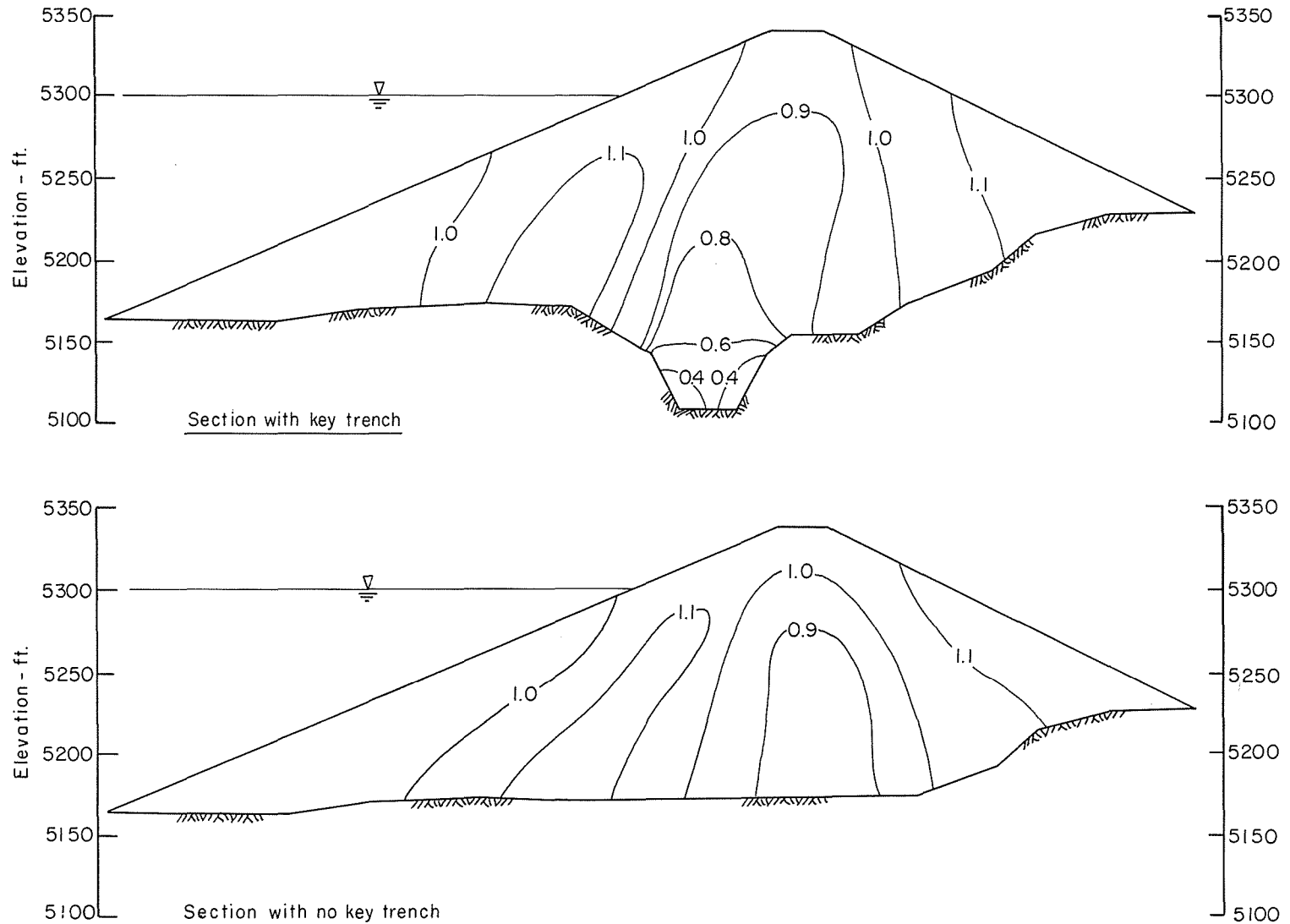


FIG.20 CONTOURS SHOWING RATIO OF VERTICAL STRESS IN EMBANKMENT TO OVERBURDEN PRESSURE FOR CONDITIONS WITH AND WITHOUT KEY TRENCH AT STA.15+00-AFTER WETTING OF SOIL IN KEY TRENCH

indicate that failure was probably initiated in the key trench in the vicinity of Sta. 14+00, and then progressed downstream approximately along the section ABC shown in Fig. 21. A cross-section through the embankment along section ABC is shown in Fig. 22.

The overall progression of piping leading to failure might thus be visualized as follows:

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 (see locations 1 and 2 in Fig. 21) 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 (Fig. 22).

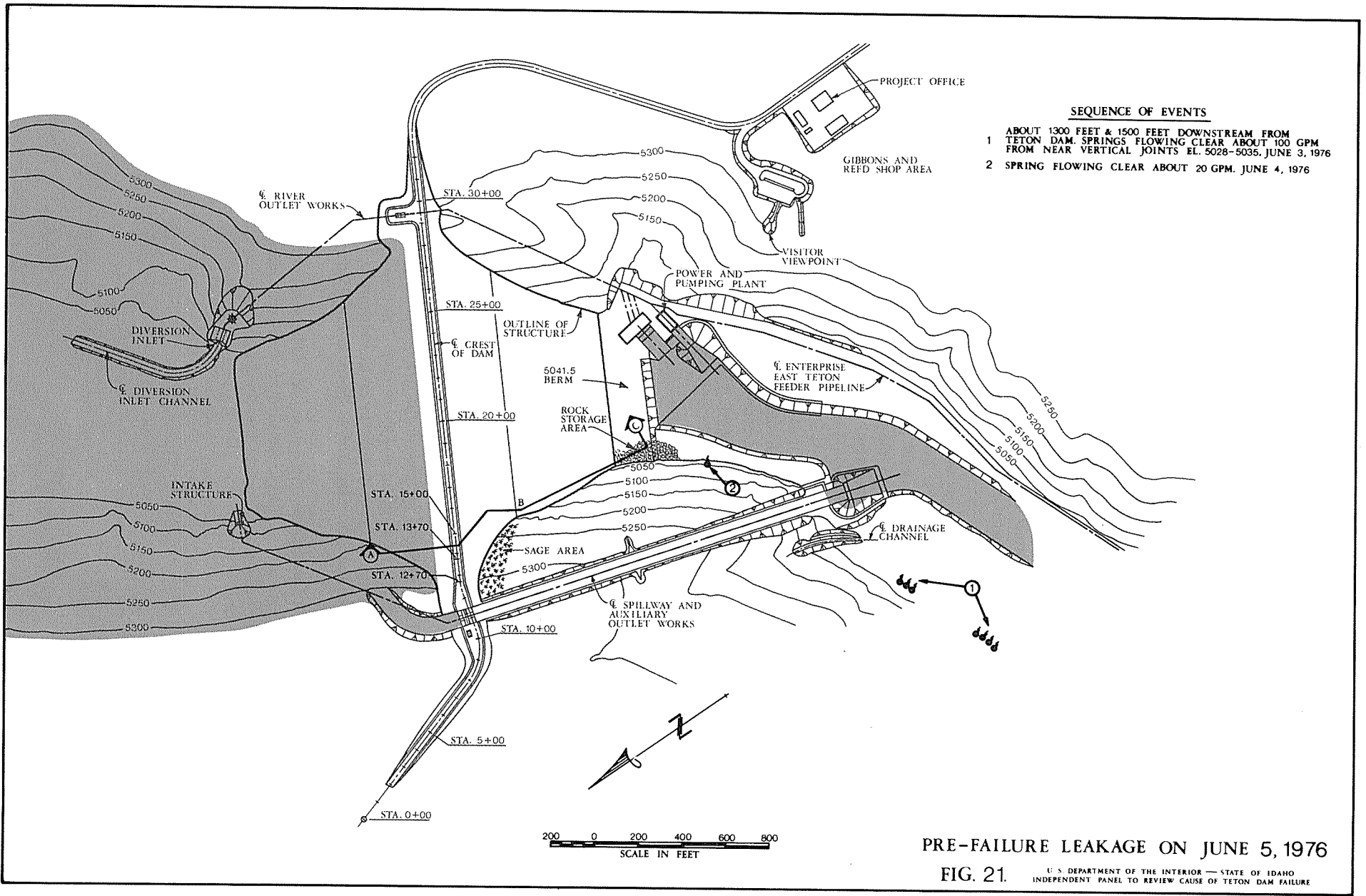
Thus the critical escape route for leakage was the multitude of partially filled void spaces in the loose slabby rock just beneath the Zone 1 fill downstream from the key trench. Significantly, materials partially filling void spaces in this zone of rock would be unaffected by overburden pressures from 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 Sta. 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 as shown in Fig. 23(a), 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 as shown in Fig. 23(b). At this stage water probably began to emerge at the contact of the embankment with the underlying rock at about El. 5190 to 5200.

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 as shown in Fig. 23(c), 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 as shown in Fig. 23(d).

Beyond this point the progressive formation of sinkholes, both upstream and downstream, as illustrated in Fig. 23(e), provided an ever-accelerating mechanism for internal erosion, finally leading to complete breaching of the dam as illustrated in Fig. 23(f).



D-35

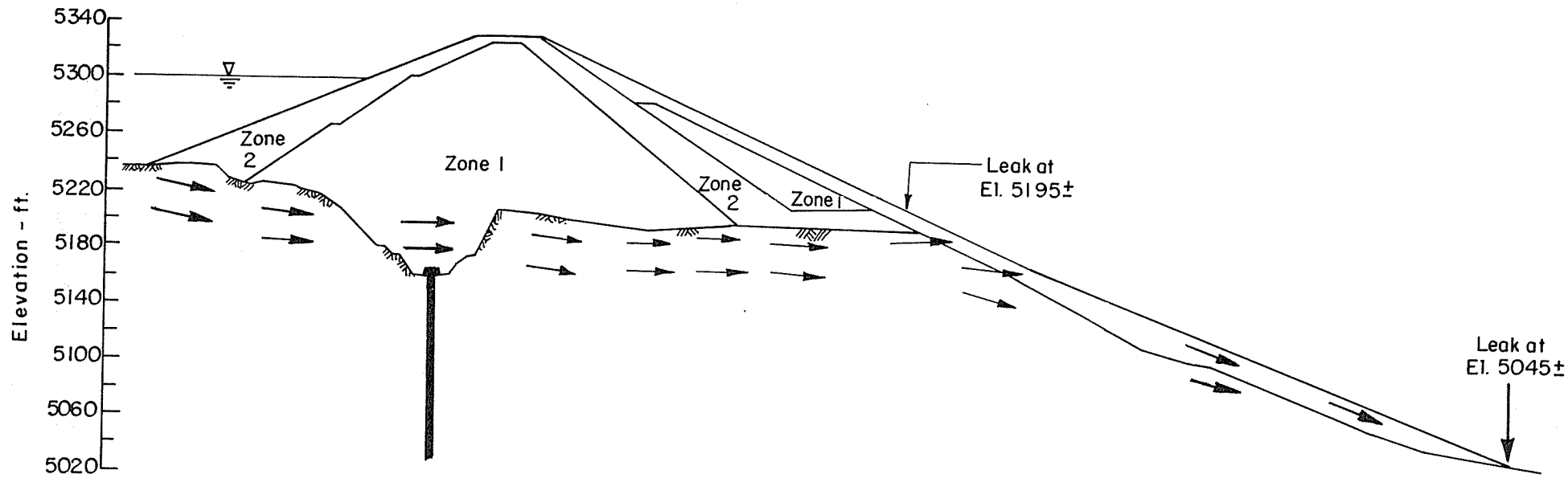
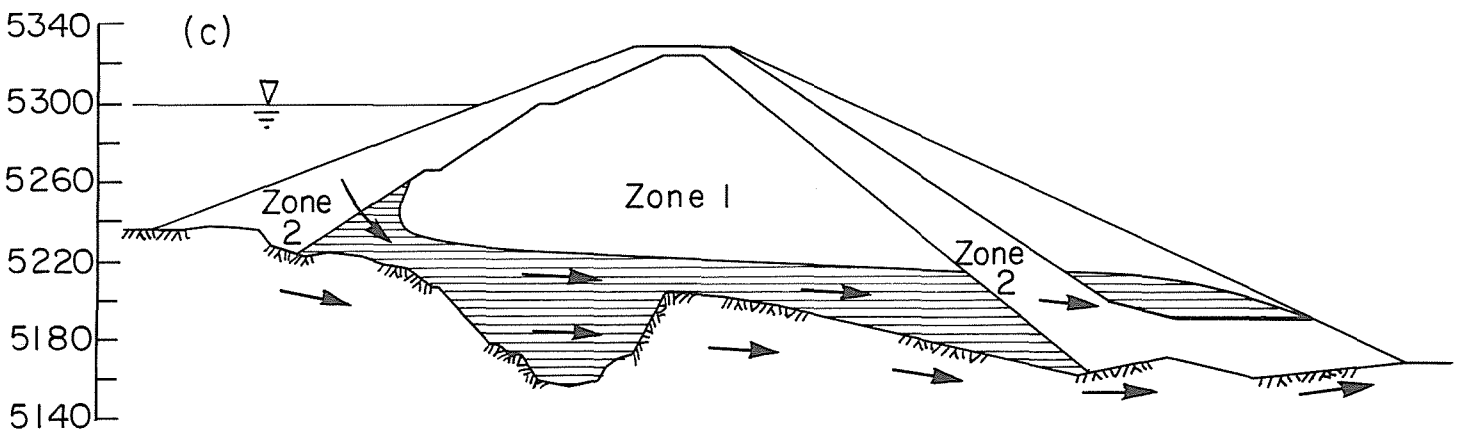
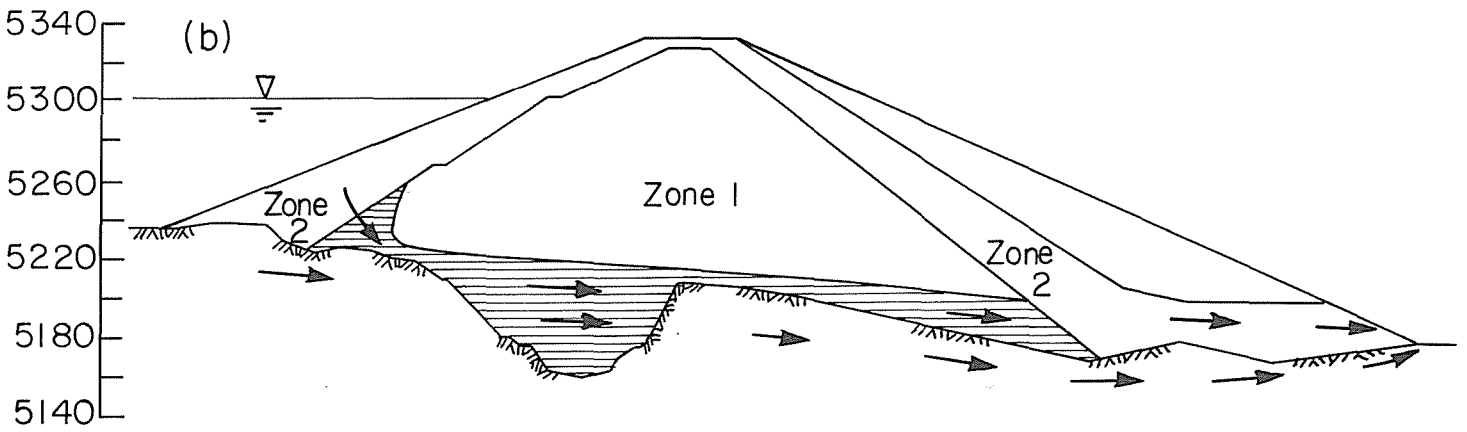
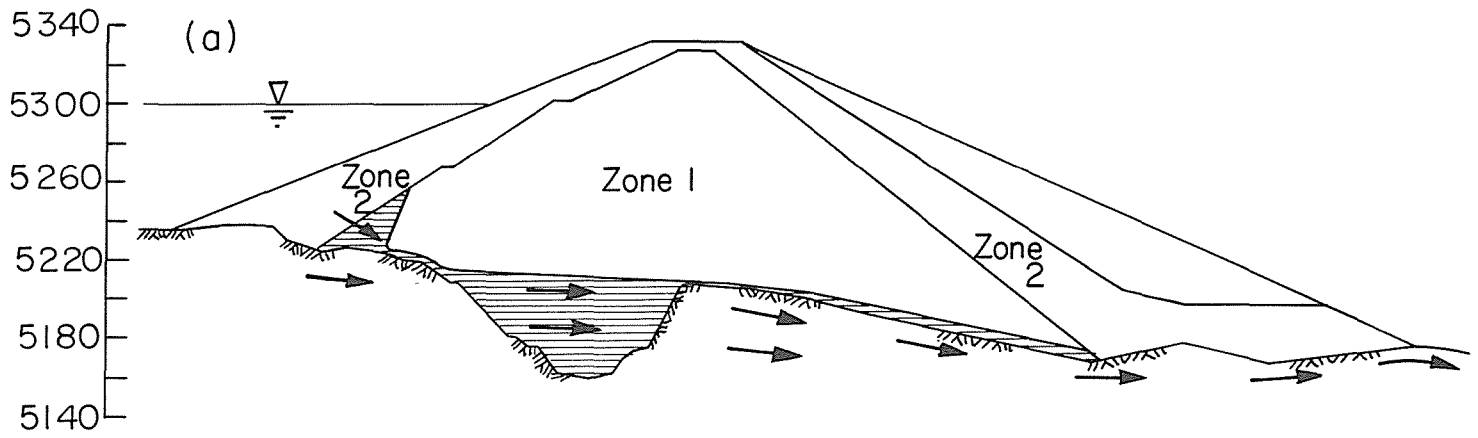
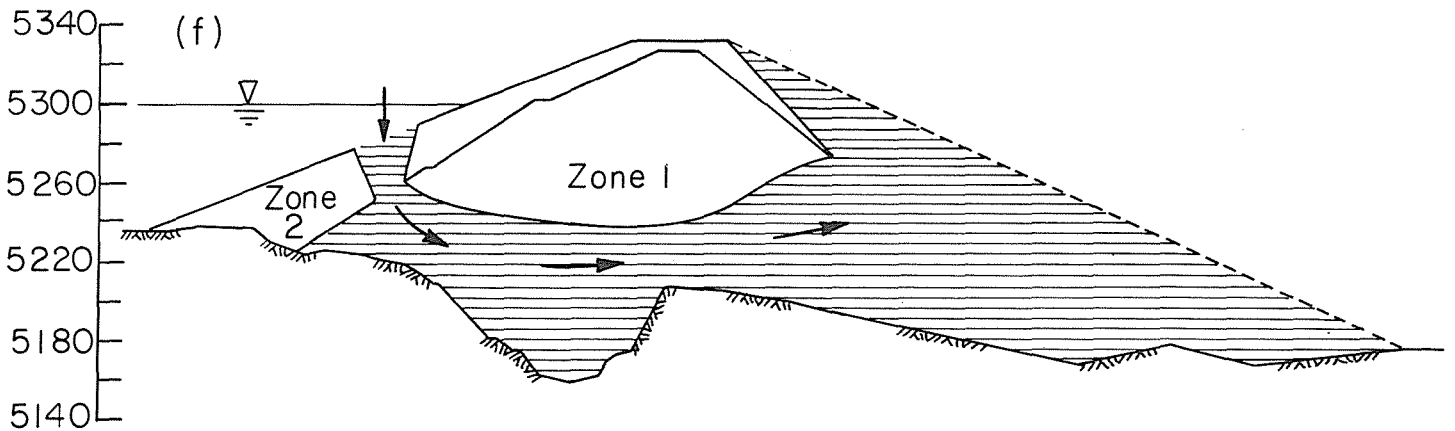
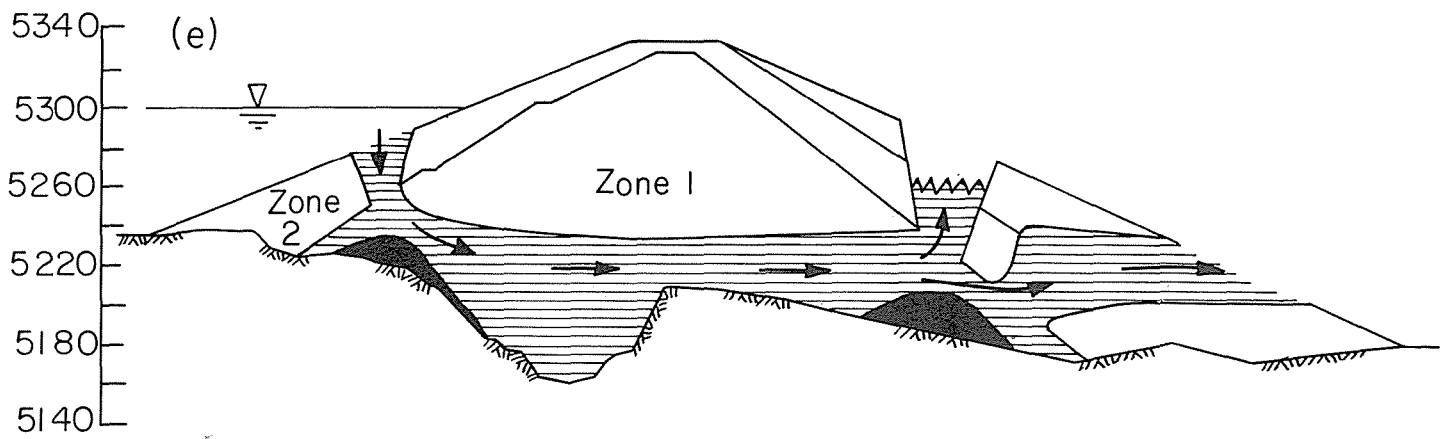
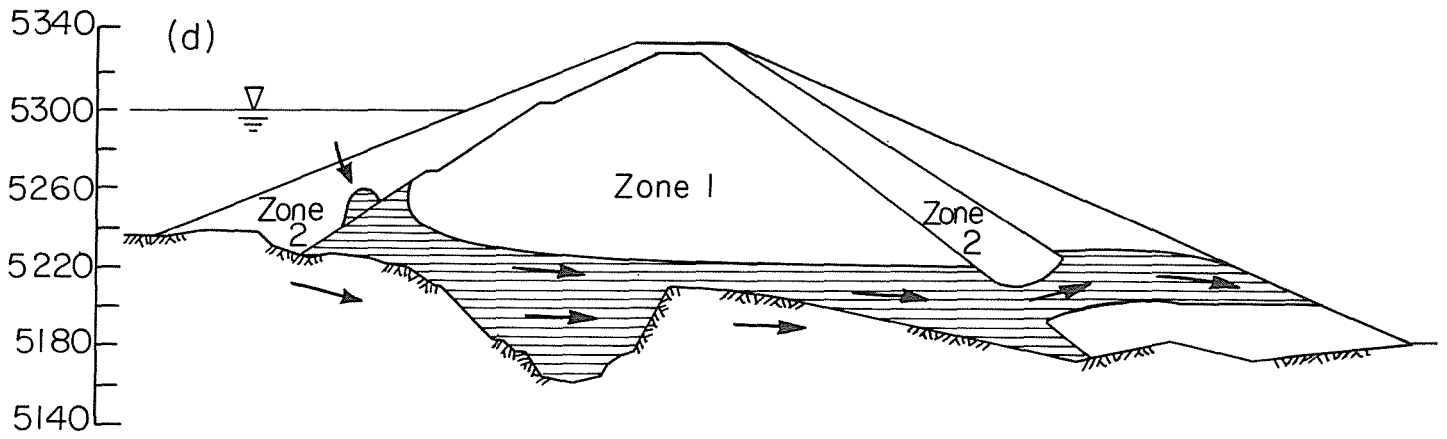


FIG. 22 PROBABLE PATH OF WATER IN EARLY STAGES OF LEAKAGE



(A, B, C, OF F) **FIG. 23** CONCEPTUAL MECHANISM OF PROGRESSIVE FAILURE ALONG SECTION A-B-C



(CONT. D, E, F)

FIG. 23 CONCEPTUAL MECHANISM OF PROGRESSIVE FAILURE ALONG SECTION A-B-C

This general concept of the mechanism appears to be consistent with the photographic record of the development of the failure.

It should be noted that even this rather detailed 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 were contributory 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 Station 14+00 was due to imperfect grouting of the rock just below the grout cap, or to hydraulic fracturing 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 lesson in the design and construction of future projects of this type.

REFERENCES

Bieber, Ray E. (1976) "TETON DAM: Finite Element Stress Analysis Study," Report DAC/1222 to Independent Panel to Review Cause of Teton Dam Failure, October 15, 1976.

Nobari, E.S. and Duncan, J.M. (1972) "Effect of Reservoir Filling on Stresses and Movements in Earth and Rockfill Dams," Geotechnical Engineering Report No. TE 72-1 to the Office, Chief of Engineers, U.S. Army Waterways Experiment Station, University of California, Berkeley, January.

Ozawa, Y. and Duncan, J.M. (1973) "ISBILD: A Computer Program for Analysis of Static Stresses and Movements in Embankments," Geotechnical Engineering, Report No. TE 73-4, University of California, Berkeley, December.

Wong, Kai S. and Duncan, J.M. (1974) "Hyperbolic Stress-Strain Parameters for Nonlinear Finite Element Analyses of Stresses and Movements in Soil Masses," Report No. TE 74-3 to National Science Foundation, Office of Research Services, University of California, Berkeley, July.

APPENDIX E

POST-FAILURE JOINT MAPPING

This appendix includes the following items:

1. GEOLOGIC EXPLANATION AND LOCATION MAP
A legend of geologic units and symbols and a map showing the locations and orientations of geologic sections in the right abutment.
2. STRIP MAPS OF THE RIGHT ABUTMENT KEYWAY FROM SPILLWAY TO RIVER CHANNEL showing joints 10 ft and longer.
3. GEOLOGIC SECTIONS
A-A¹ parallel to dam centerline 100 ft downstream
B-B¹ parallel to grout cap 10 ft downstream
C-C¹ parallel to grout cap 10 ft upstream
D-D¹ parallel to dam centerline 150 ft upstream
4. EXPLANATION OF FIELD OBSERVATIONS
5. TABULATION OF JOINT CHARACTERISTICS CROSS REFERENCED TO GEOLOGIC SECTIONS
6. FIGS. E-1 THROUGH E-24
Photos of joints in right abutment of Teton Dam
7. Memorandum from Project Construction Engineer, Newdale, Idaho, to U.S. Bureau of Reclamation, Director of Design and Construction, Denver, Colorado, dated March 14, 1976, Subject: Proposed Treatment of Fissures and Cavities in Right Abutment Key Trench

GEOLOGIC UNITS

WELDED ASH-FLOW TUFF (Tsv). Tertiary silicic volcanics of rhyolite composition. The tuff is variably welded, generally porphyritic with light colored feldspar phenocrysts up to 1/4 inch within a fine- to medium-grained tuff matrix. Very lightly to locally moderately vesicular with vesicles (up to 3/8" in size). Moderately to lightly jointed (joints spaced mostly from 1 to 6 feet) with intensely jointed zones (joints spaced less than 0.5 ft. apart). Joints are tight to open up to 1/4", but locally up to 4 inches. Many joints are stained with limonite, hematite, manganese and some calcite. Most of the flow is characterized by a faint to distinct foliation that is caused by flattened, wavy streaky, light colored pumice fragments (and some lapilli) and by some zones or areas of flattened or elongated vesicles. The rock generally is hard; hand size specimens break with a moderate hammer blow. The color is variable from light gray to medium gray with shades of red, brown and purple.

The welded ash-flow tuff formation exposed upstream of the grout curtain is divisible into 3 units: Tsv₁, Tsv₂, and Tsv₃. These units are difficult to trace or differentiate downstream of the grout curtain. The middle unit, Tsv₂, appears to pinch out or terminate downstream of the grout curtain and the upper platy unit, Tsv₁, appears to thicken downstream of the grout cap.

Tsv₁

Light gray-brown in upper part grading to light to medium gray in lower part and forms irregular ragged outcrops. Platy: Lenticular and tabular plates mostly 2 to 6 inches thick but some up to 18 and 24 inches thick. The plates dip at low angles and are about parallel to the faint foliation. Spaces between plates are 1/4 inch up to 2 inches open. Some plates are coated with calcite up to 1/4 inch; caliche and silt fill many of the openings in the upper 5 to 6 feet of the unit. The near vertical jointing strikes N 30° - 60° E and N 60° - 90° E, spaced from about 3 to 10 feet apart. Most of these joints are tight, planar and smooth and about half are stained with iron, manganese and calcite.

Gradational with upper platy unit and is marked at lower contact by a breccia zone. The unit, which is about 60 feet thick upstream of the grout cap, dips gently downstream (westward) and appears to pinch out or terminate against a strong NW trending joint about 15 feet downstream from the grout cap.

Tsv₂

Medium to dark gray. Forms many near vertical cliffs along the abutment upstream of the grout cap. Jointing, much of which is foliation jointing, is moderate to locally intense, (mostly 1-2 feet), mostly high angle and trends N 10° to 40° W. Most joints are tight, planar to wavy and smooth, but some joints are rough and open up to 4 inches. Calcite coats many of the joints and fills some joints up to 1/2 inch thick. Iron and manganese stains many of the joints. The foliation, which is faint to distinct, appears to dip at high angles in contrast to the low dipping foliation in the overlying and underlying units.

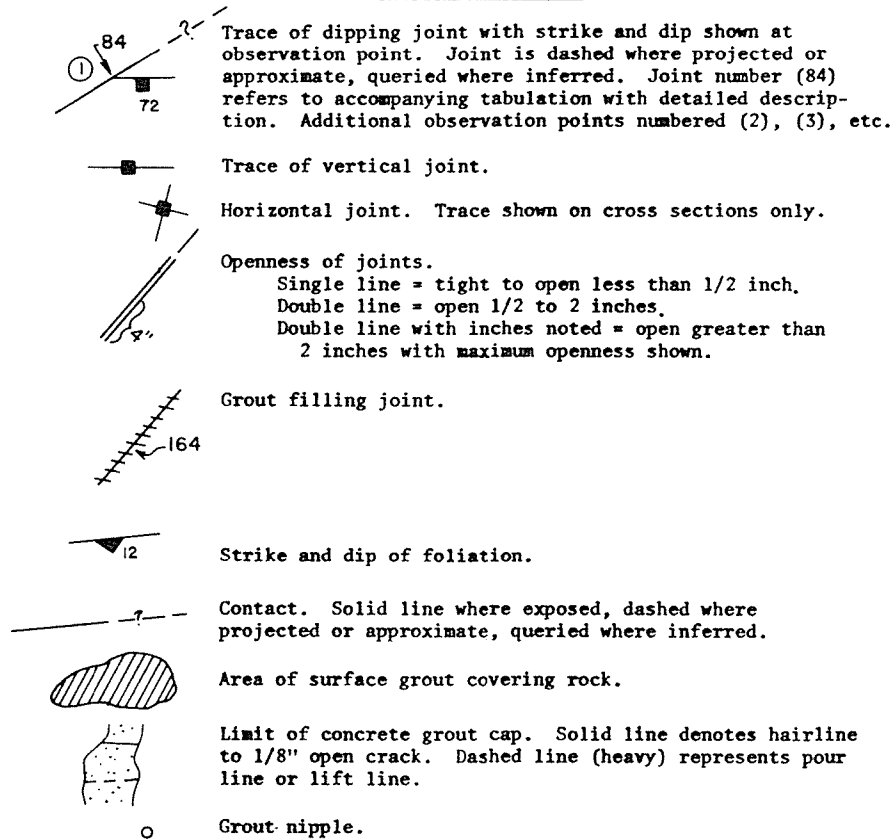
Breccia Zone - This zone forms the contact between Tsv₂ and Tsv₃ and is quite irregular and varies from a single zone about 2 inches thick up to several zones that occur over a thickness of up to about 10 feet. The zones consist of brecciated and crushed rock fragments, from less than 1/2 inch up to 12 inches thick, cemented together by a white, brown to dark gray calcite up to 1-1/2 inches thick. Openings in the breccia zone are irregularly shaped and vary from 1/4 inch up to 4 inches in size. The upper and lower contacts in some places are planar, but in other places are wavy and irregular. Through the grout cap area this zone occurs as a prominent low dipping, calcite filled joint about 1-1/2 inches thick.

Tsv₃

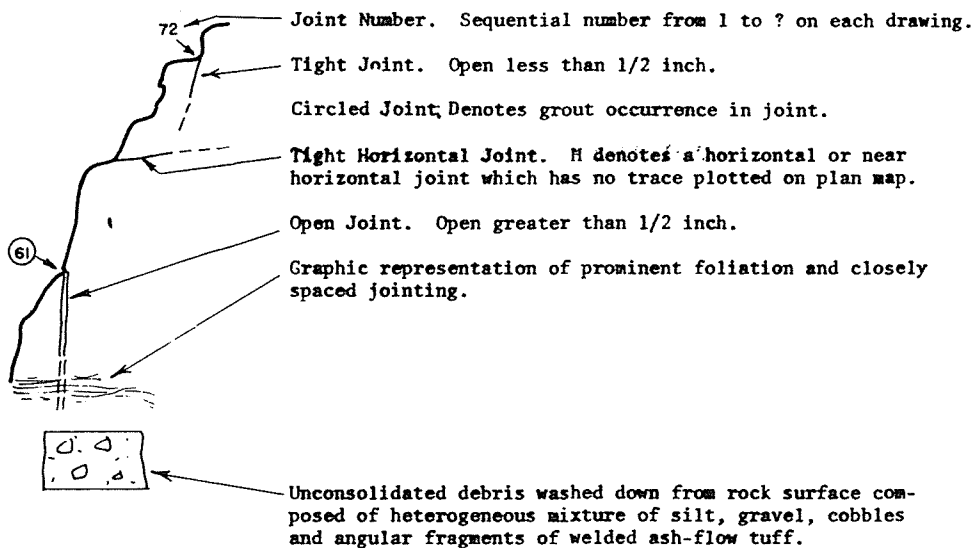
Characterized by the bold massive blocky outcrops and the prominent benches in the lower part of the abutment. The near vertical jointing is very prominent and many joints can be traced over 100 feet. The major joint trend is N 15° - 40° W. Spacing is mostly 5 to 10 feet with openness varying from tight to 1/4 inch, locally 1 to 4 inches. Most joints are smooth and are stained with iron and manganese oxides and some calcite. The foliation planes are near horizontal and are from a few inches to about 3 feet apart; some are tight, but others are open to 1/2 inch. Color is mostly medium gray.

SYMBOLS

Plan Geologic Maps

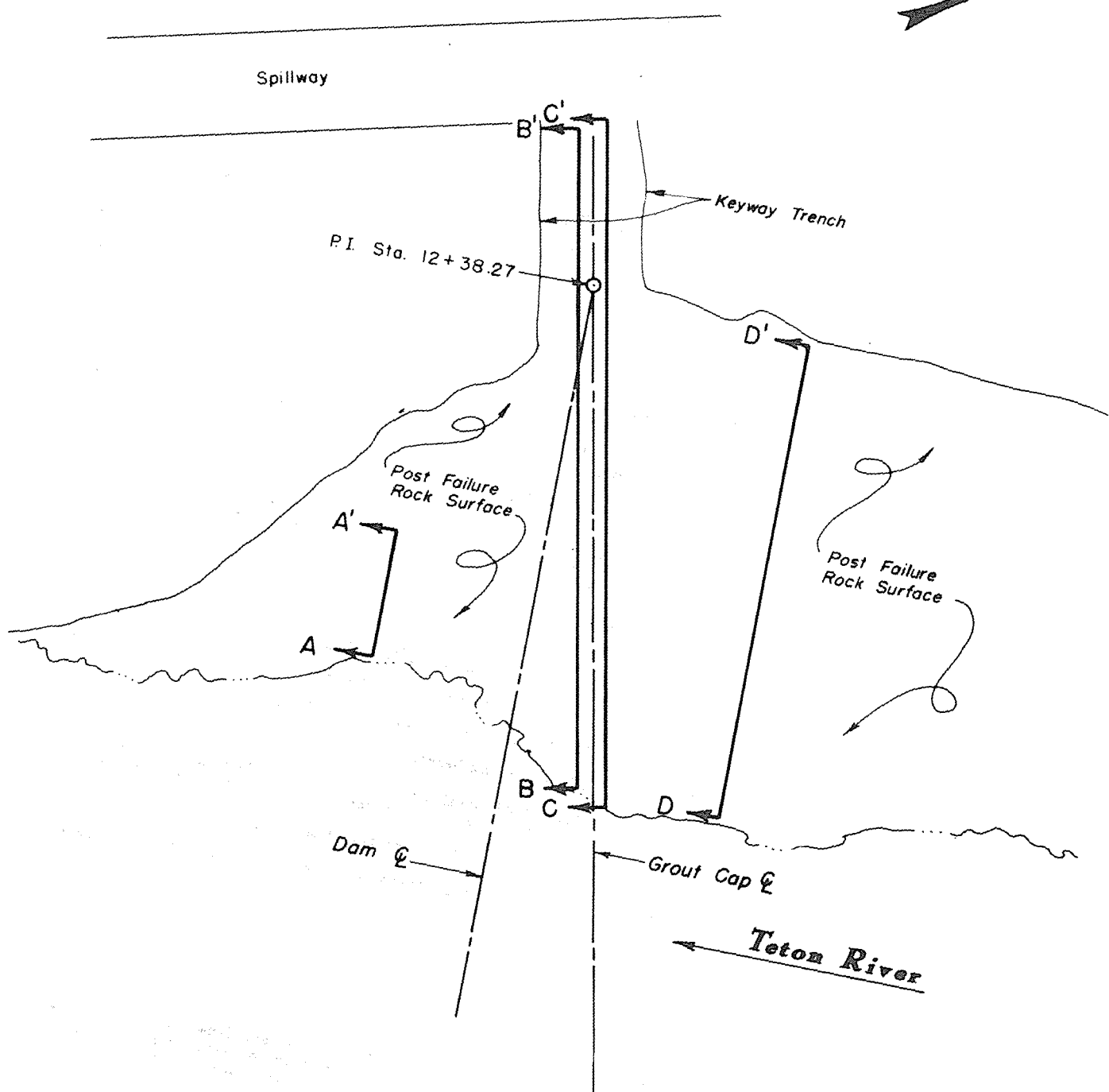
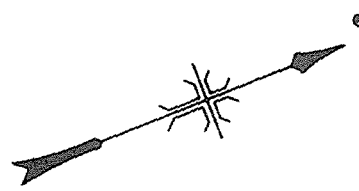


Geologic Cross Sections

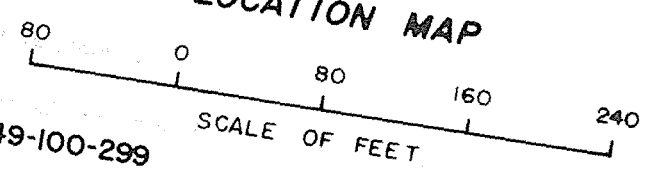


Notes:

1. For detailed description of each individual joint, refer to accompanying tabulation by Joint Number.
2. Length of joint trace in cross section portrays the continuity at depth.
3. All joint traces shown with apparent dips.

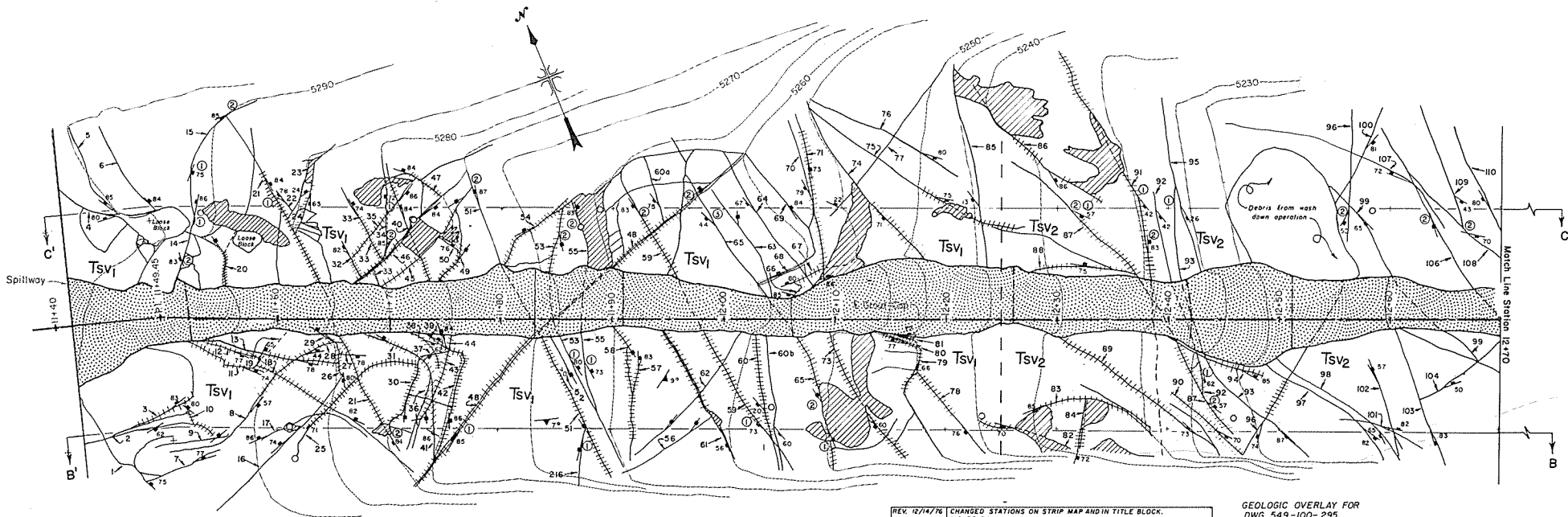


LOCATION MAP

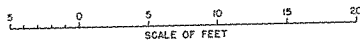


USBR DWG. NO. 549-100-299

E-5



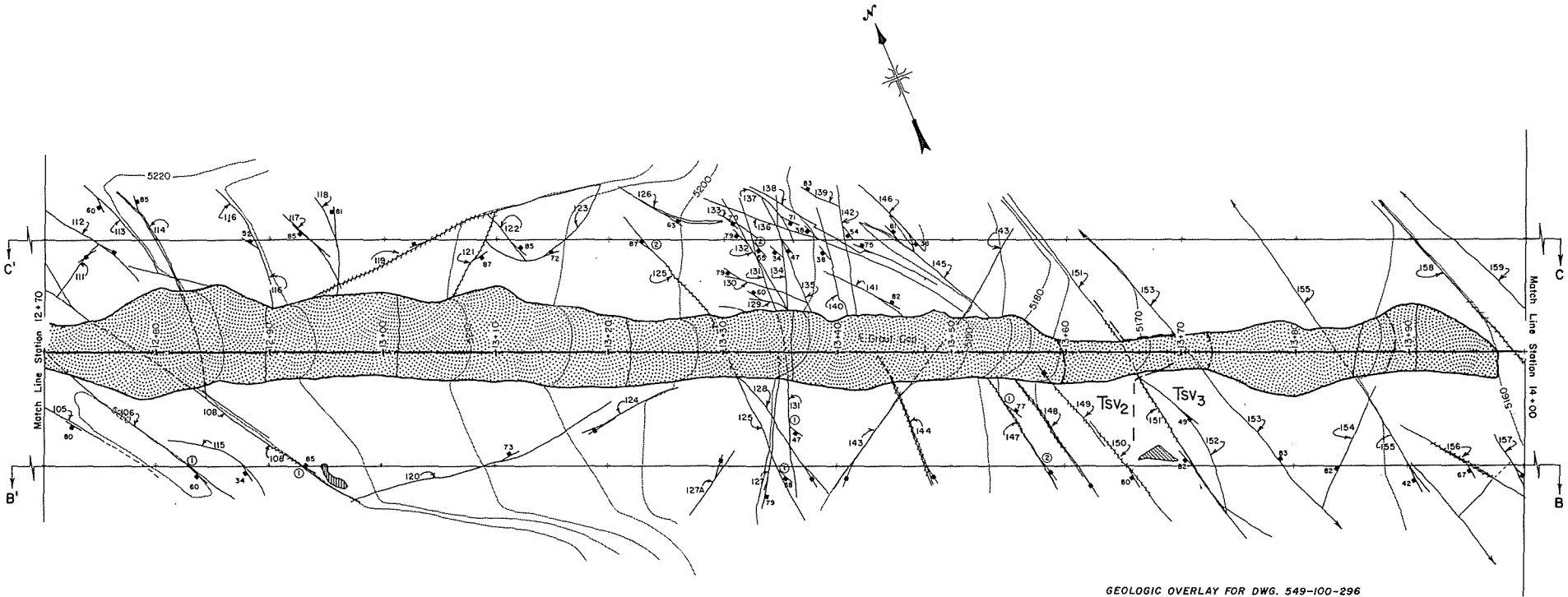
NOTES
 Contour sketched from Survey Control on 1-foot centers along Grout Cap C and both 10 feet upstream and downstream from C.
 Contour interval 5'



REV. 12/14/76 EDM 100	CHANGED STATIONS ON STRIP MAP AND IN TITLE BLOCK. MOVED P.L.
ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION TETON BASIN PROJECT LOWER TETON DIVISION - IDAHO TETON DAM STRIP TOPOGRAPHY ALONG GROUT CAP Sta. 11+40 to 12+70	
GEOLOGY <i>P.M.S.</i> DRAWN <i>EDM</i> CHECKED <i>M.P.S.</i>	SUBMITTED <i>John H. ...</i> RECOMMENDED <i>Paul D. ...</i> APPROVED <i>...</i> REGIONAL ENGINEER
BOISE, IDAHO SHEET 1 OF 4	NOVEMBER 1976 549-100-295

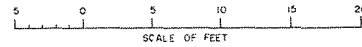
GEOLOGIC OVERLAY FOR
 DWG. 549-100-295

E-6



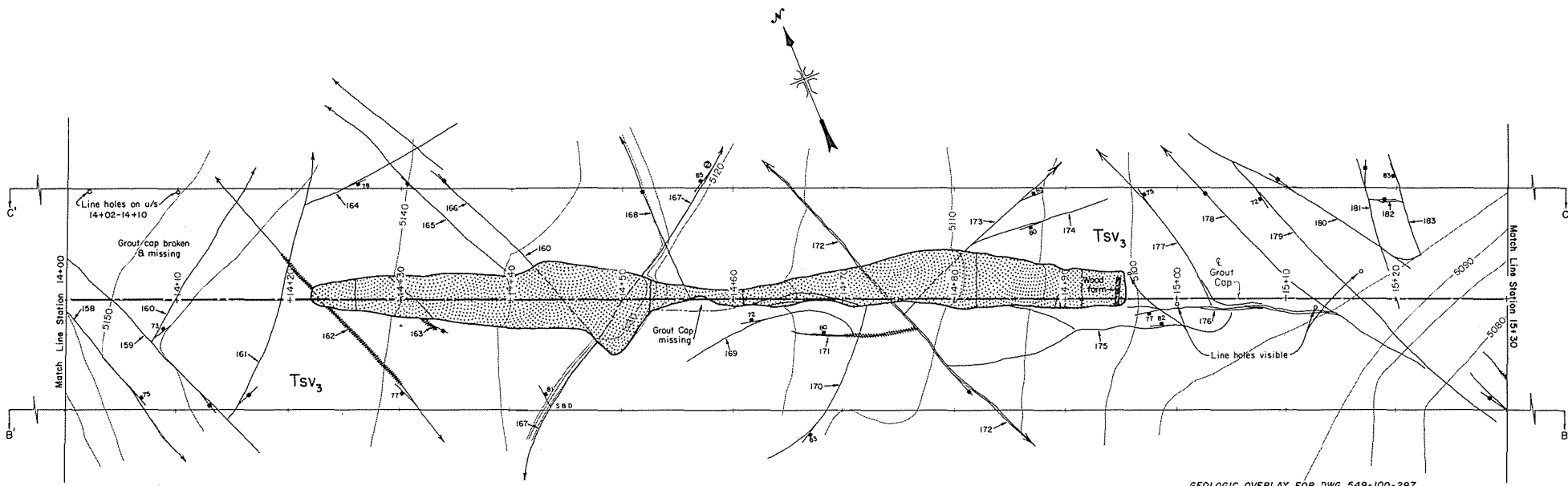
GEOLOGIC OVERLAY FOR DWG. 549-100-296

NOTES:
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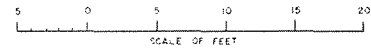
REV. 12-14-76 CHANGED STATIONS ALONG GROUT CAP AND IN TITLE BLOCK R. E. 100	
ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION TETON BASIN PROJECT LOWER TETON DIVISION - IDAHO	
TETON DAM STRIP TOPOGRAPHY ALONG GROUT CAP Sta. 12+70 to 14+00	
DESIGNED BY: <i>Ed M.</i>	SUBMITTED BY: <i>John L. ...</i>
DRAWN BY: <i>Ed M.</i>	RECOMMENDED BY: <i>John L. ...</i>
CHECKED BY: <i>Ed M.</i>	APPROVED BY: <i>John L. ...</i>
BOISE, IDAHO	NOVEMBER 1976
SHEET 3 OF 4	549-100-296

B-7

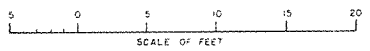
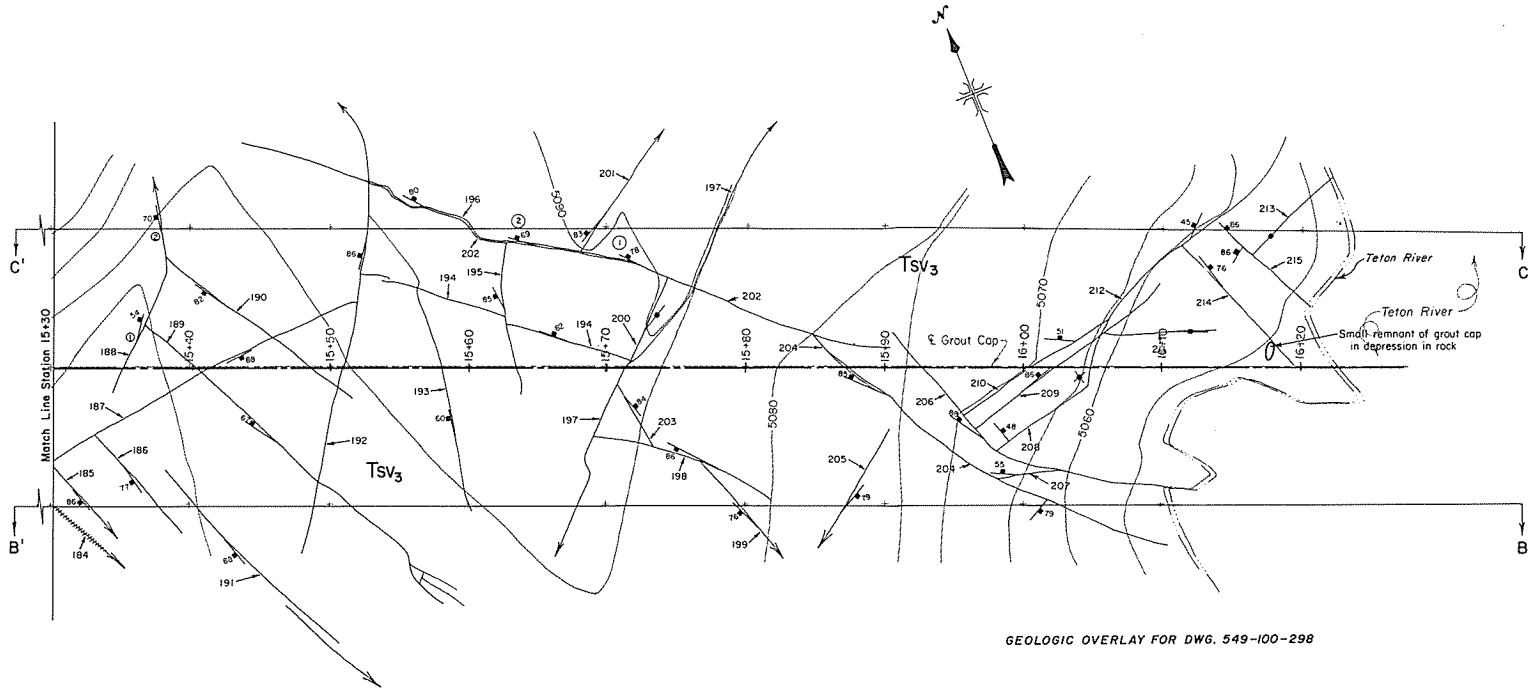


GEOLOGIC OVERLAY FOR DWG. 549-100-297

NOTES:
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 Contour interval 5'.



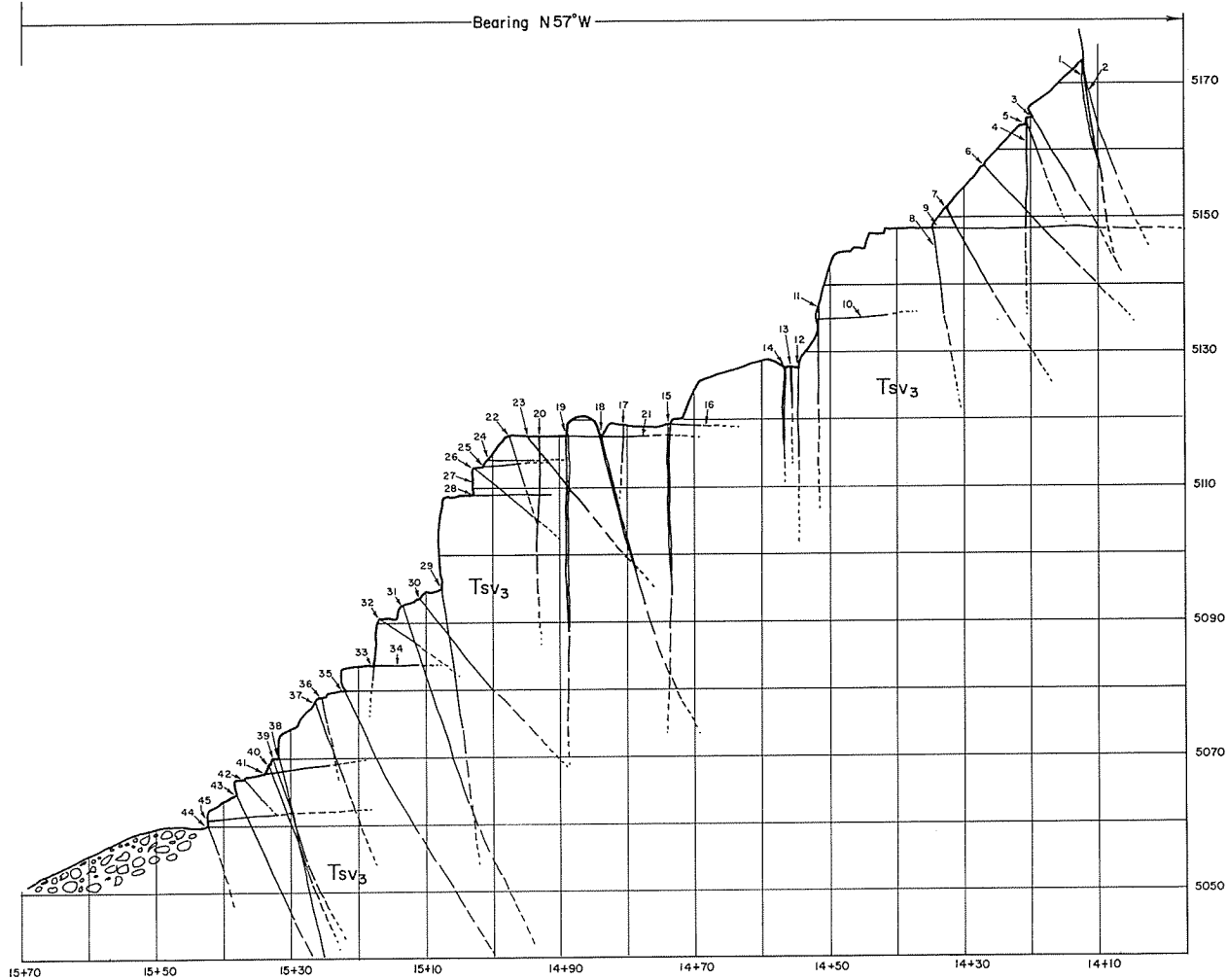
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ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION TETON BASIN PROJECT LOWER TETON DIVISION-IDAHO	
TETON DAM STRIP TOPOGRAPHY ALONG GROUT CAP Sta. 14+00 to 15+30	
GEOLOGY <i>J. M. S.</i>	SUBMITTED <i>J. M. S.</i>
DRAWN <i>E. D. M.</i>	RECOMMENDED <i>J. M. S.</i>
CHECKED <i>J. M. S.</i>	APPROVED <i>J. M. S.</i> REGIONAL ENGINEER
BOISE, IDAHO	NOVEMBER 1978
SHEET 3 OF 4	549-100-297



NOTES
 Contour sketched from Survey Control on 1-foot centers along Grout Cap \mathcal{E} and both 10 feet upstream and downstream from \mathcal{E} .
 Contour interval 5'.

REV. 12-14-76 ERC 100	CHANGED STATIONS ON PROFILE AND IN TITLE BLOCK
ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION TETON BASIN PROJECT LOWER TETON DIVISION-IDAHO	
TETON DAM STRIP TOPOGRAPHY ALONG GROUT CAP Sta. 15+30 to 16+20	
GEOL. BY <i>M.S.</i>	SUBMITTED BY <i>M.S.</i>
DRAWN BY <i>EDM</i>	RECOMMENDED BY <i>W.C.</i>
CHECKED BY <i>M.</i>	APPROVED BY <i>W.C.</i>
BOISE, IDAHO	NOVEMBER 1976
SHEET 4 OF 4	549-100-298

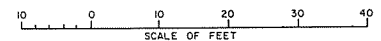
E-9



ELEVATIONS

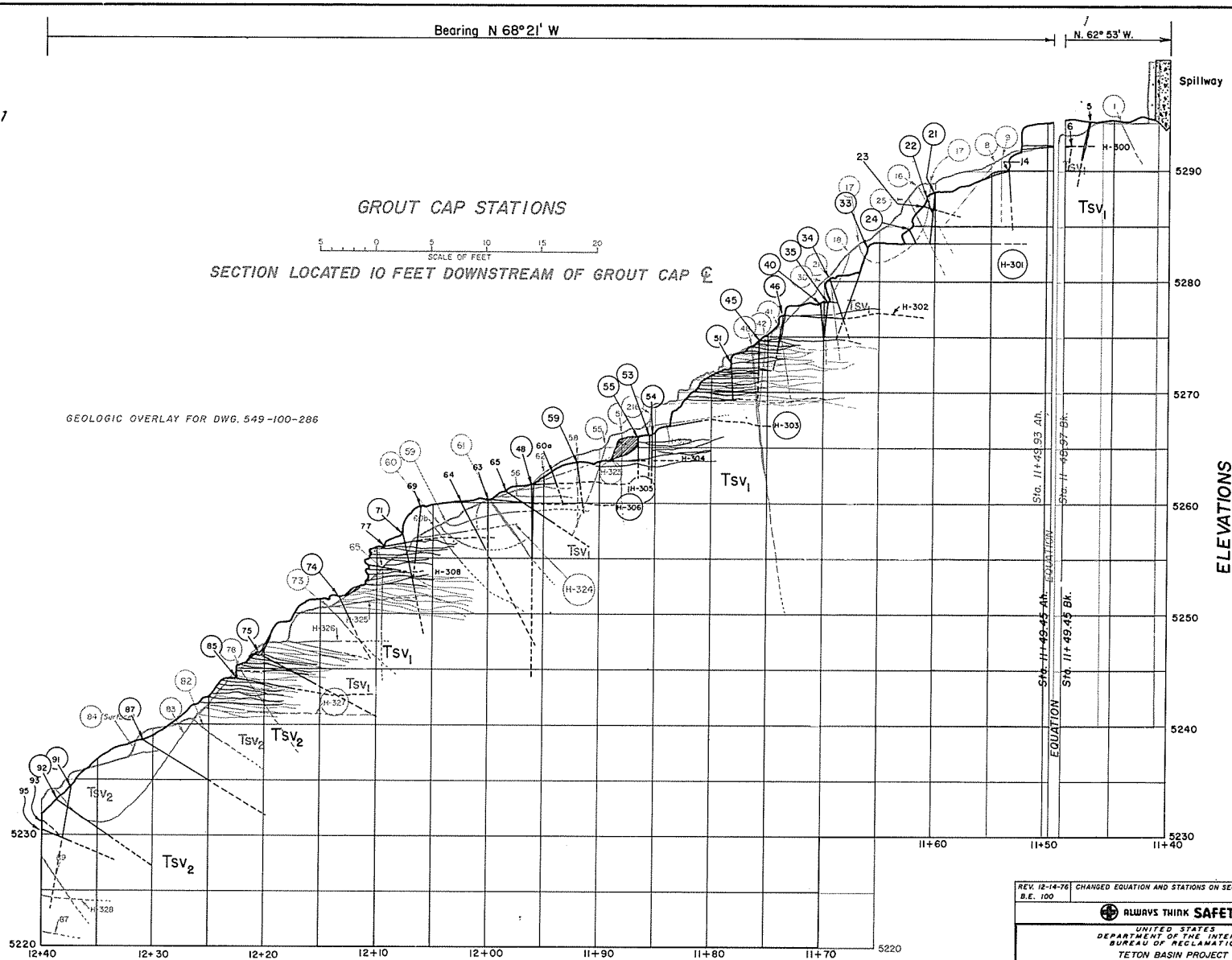
NOTES:
For Explanation of Geologic Symbols
see Dwg. 549-100-299.
For location of Section see Dwg.
549-100-299.
Rock surface and geology
sketched in field from limited
horizontal and vertical survey
control.

STATIONS (referenced to Dam \odot)



SECTION LOCATED 100 FEET DOWNSTREAM OF DAM \odot

REV 12/14/76 ERC 100	CHANGED STATIONS ON SECTIONS AND IN TITLE BLOCK
ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION TETON BASIN PROJECT LOWER TETON DIVISION-IDAHO TETON DAM GEOLOGIC SECTION A-A' Sta. 13+97 - 15+70	
GEOLOGY <i>J. E. ...</i> D.S.	SUBMITTED <i>J. E. ...</i>
DRAWN <i>G. F. ...</i>	RECOMMENDED <i>J. E. ...</i>
CHECKED <i>G. F. ...</i>	APPROVED <i>J. E. ...</i> REGIONAL ENGINEER
BOISE, IDAHO	NOVEMBER 1976
SHEET 1 OF 1	549-100-280



GROUT CAP STATIONS
 SECTION LOCATED 10 FEET DOWNSTREAM OF GROUT CAP C

GEOLOGIC OVERLAY FOR DWG. 549-100-286

GROUT CAP STATIONS

SECTION LOCATED 10 FEET UPSTREAM OF GROUT CAP C

NOTES:
 For explanation of Geologic Symbols, see Dwg. 549-100-299.
 For location of Section, see Dwg. 549-100-299.
 Rock surface and geology sketched in field from horizontal and vertical survey control on 1-foot centers.

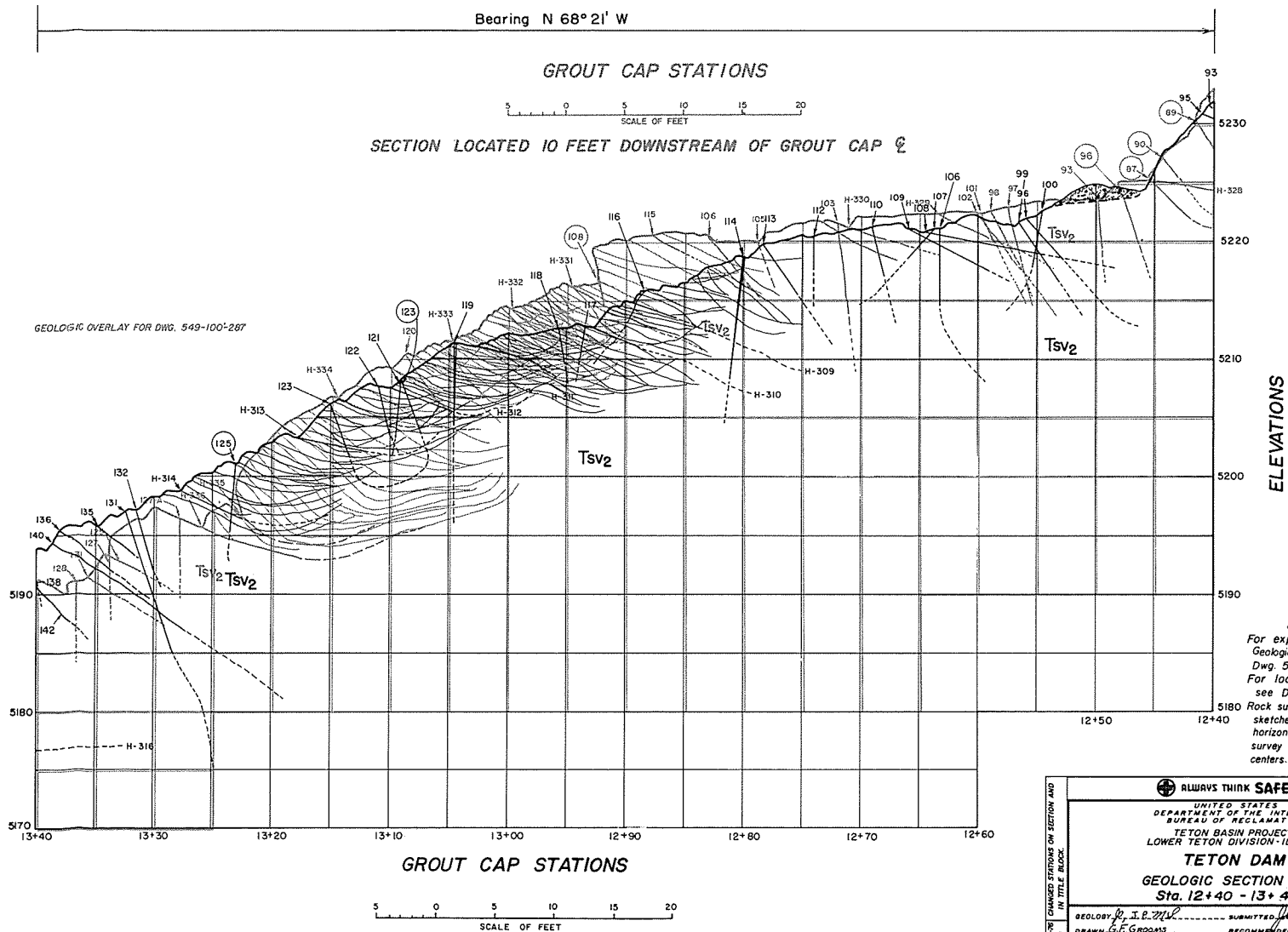
REV. 12-14-76 CHANGED EQUATION AND STATIONS ON SECTIONS AND TITLE BLOCK.
 B.E. 100

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UNITED STATES
 DEPARTMENT OF THE INTERIOR
 BUREAU OF RECLAMATION
 TETON BASIN PROJECT
 LOWER TETON DIVISION-IDAHO

TETON DAM
 GEOLOGIC SECTION C-C'
 Sta. 11+40 - 12+40

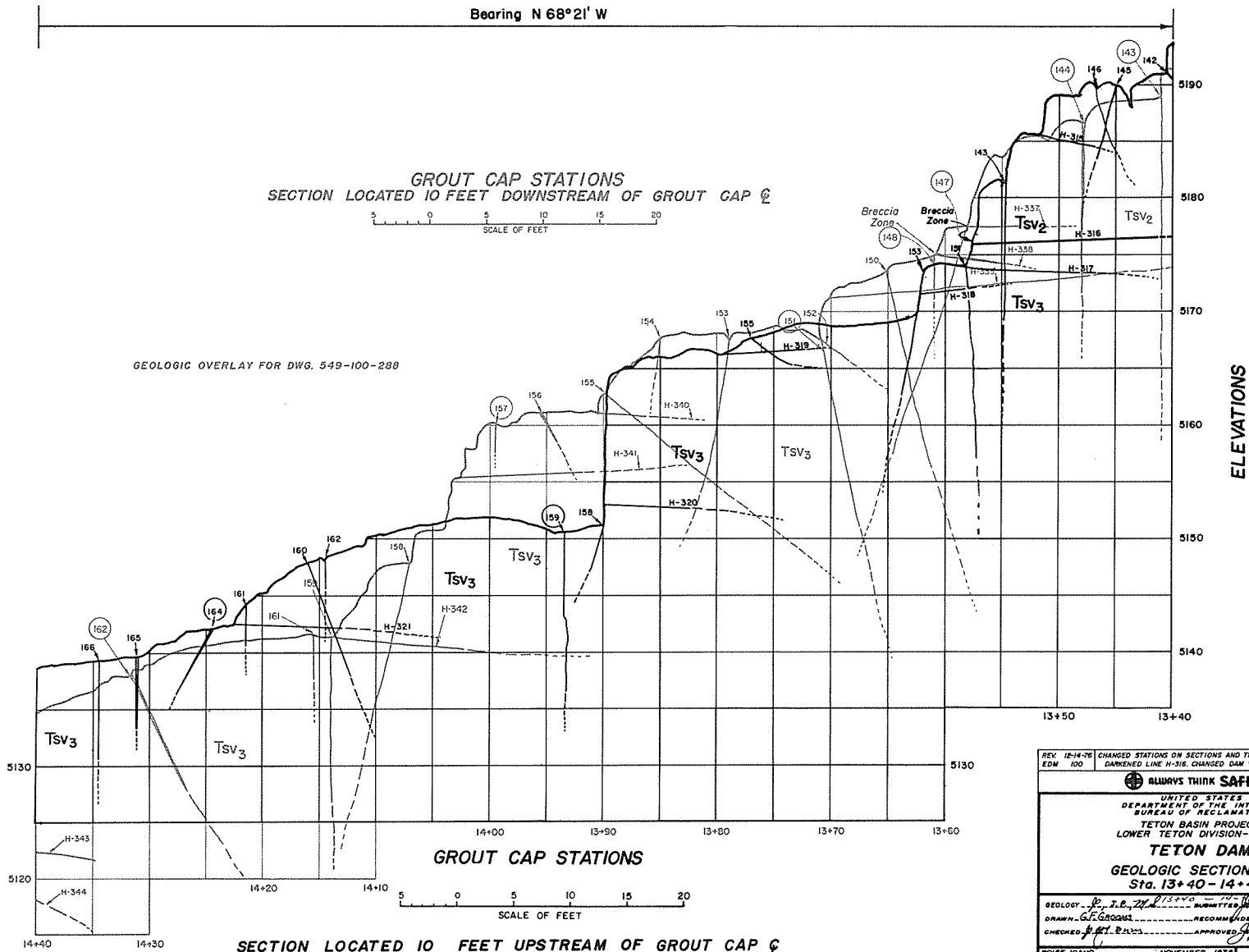
GEOLOGY *J.P.W.* SUBMITTED *[Signature]*
 DRAWN *G.E. GROOMS* RECOMMENDED *[Signature]*
 CHECKED *[Signature]* APPROVED *[Signature]*
 BOISE, IDAHO NOVEMBER 1976 SHEET 1 OF 5 549-100-286



NOTES:
 For explanation of Geologic Symbols, see Dwg. 549-100-299.
 For location of Section, see Dwg. 549-100-299.
 Rock surface and geology sketched in field from horizontal and vertical survey control on 1-foot centers.

ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION TETON BASIN PROJECT LOWER TETON DIVISION - IDAHO	
TETON DAM GEOLOGIC SECTION C-C' Sta. 12+40 - 13+40	
DESIGNED BY <i>J. E. M. W.</i>	SUBMITTED BY <i>[Signature]</i>
DRAWN BY <i>G. F. GEORAS</i>	RECOMMENDED BY <i>[Signature]</i>
CHECKED BY <i>[Signature]</i>	APPROVED BY <i>[Signature]</i>
RESIDENT ENGINEER	
BOISE, IDAHO NOVEMBER 1978 549-100-287	
SHEET 2 OF 3	

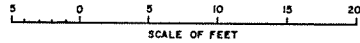
NOTES:
 For explanation of Geologic Symbols, see Dwg. 549-100-299.
 For location of Section see Dwg. 549-100-299.
 Rock surface and geology sketched in field from horizontal and vertical survey control on 1-foot centers.



GEOLOGIC OVERLAY FOR DWG. 549-100-288

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ALWAYS THINK SAFETY		
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION TETON BASIN PROJECT LOWER TETON DIVISION-IDAHO TETON DAM GEOLOGIC SECTION C-C' Sta. 13+40 - 14+40		
GEOLOGY <i>R. J. E. [Signature]</i>		SUBMITTED <i>[Signature]</i>
DRAWN <i>C. R. [Signature]</i>		RECOMMENDED <i>[Signature]</i>
CHECKED <i>[Signature]</i>		APPROVED <i>[Signature]</i> REGIONAL ENGINEER
BOISE, IDAHO		NOVEMBER 1976 SHEET 3 OF 5
		549-100-288

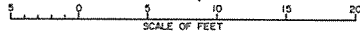
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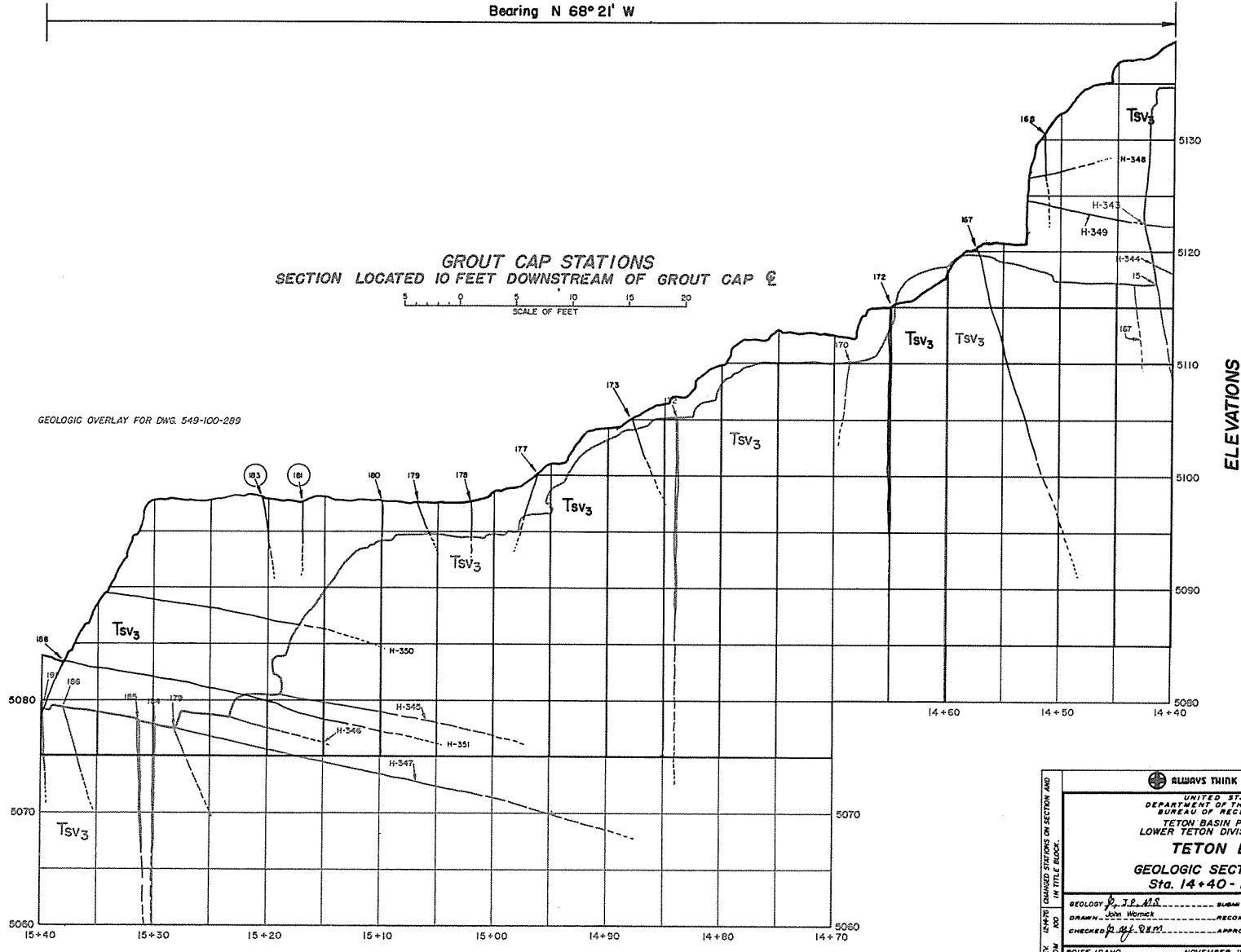
Bearing N 68° 21' W

NOTES:
For explanation of Geologic Symbols,
see Dwg. 549-100-299
For location of Section, see Dwg. 549-100-299
Rock surface and geology sketched in field from
horizontal and vertical survey control on 1-foot
centers.

GROUT CAP STATIONS SECTION LOCATED 10 FEET DOWNSTREAM OF GROUT CAP C



GEOLOGIC OVERLAY FOR DWG. 549-100-289



ALWAYS THINK SAFETY

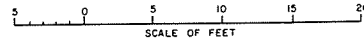
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TETON BASIN PROJECT
LOWER TETON DIVISION - IDAHO
TETON DAM
GEOLOGIC SECTION C-C'
Sta. 14+40 - 15+40

REV. (ENR) CHANGED STATIONS ON SECTION AND
ADDED IN TITLE BLOCK.

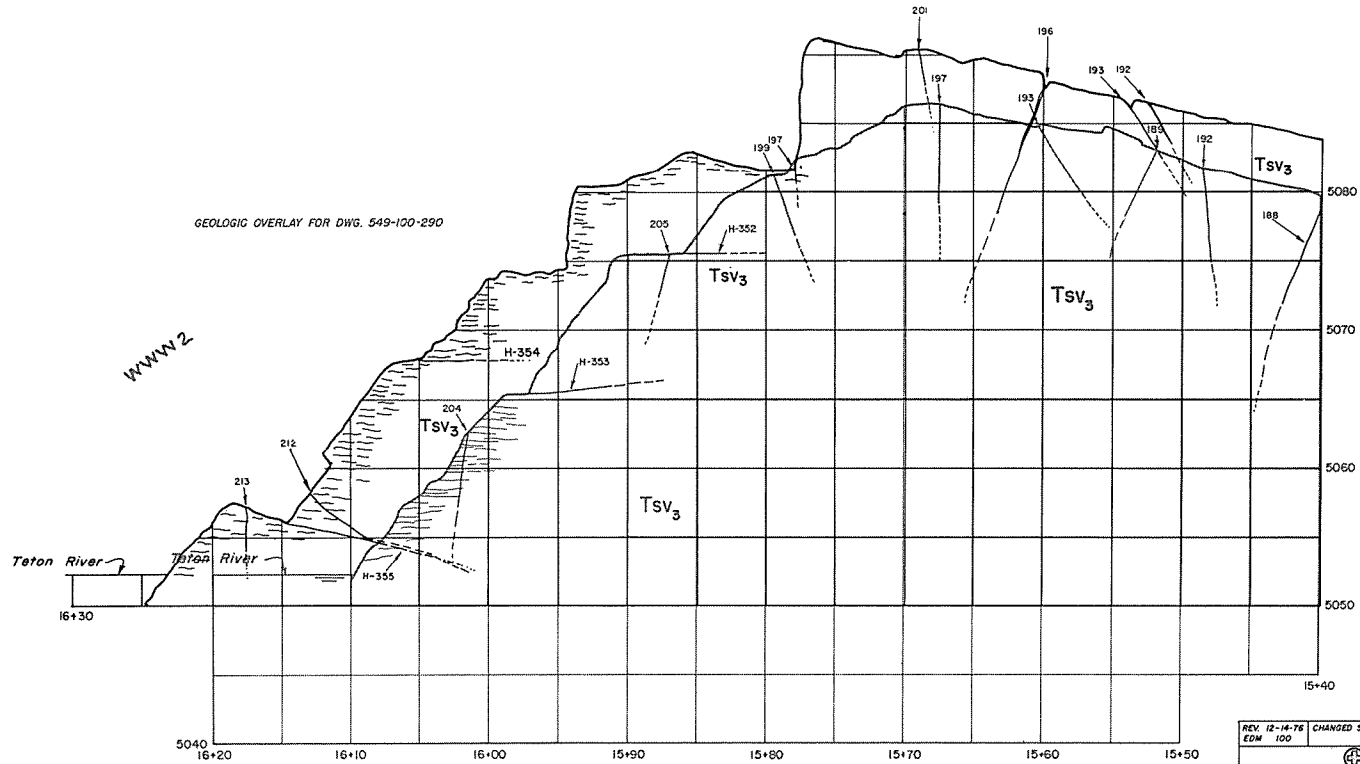
GEOLOGY: J.E. M.S. SUBMITTED: *[Signature]*
DRAWN: JOHN WENZEL RECOMMENDED: *[Signature]*
CHECKED: *[Signature]* APPROVED: *[Signature]*
REGIONAL ENGINEER

BOISE, IDAHO NOVEMBER 1976 549-100-289
SHEET 4 OF 8

GROUT CAP STATIONS
SECTION LOCATED 10 FEET UPSTREAM OF GROUT CAP C



Bearing N 68° 21' W



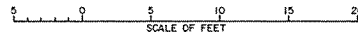
ELEVATIONS

NOTES:
For explanation of Geologic Symbols, see Dwg. 549-100-299.
For location of Section, see Dwg. 549-100-299.
Rock surface and geology sketched in field from horizontal and vertical survey control on 1-foot centers.

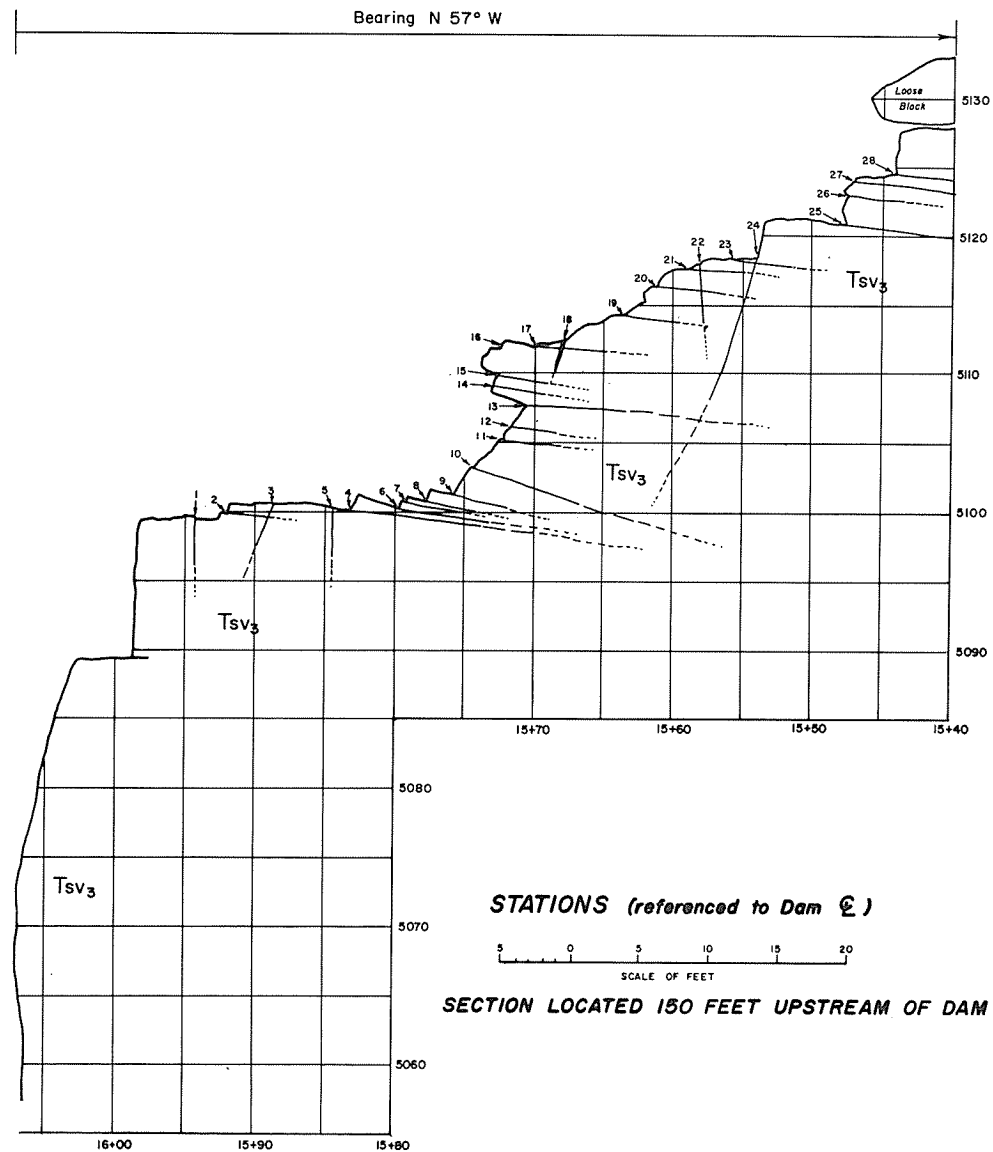
B-14

GROUT CAP STATIONS

SECTION LOCATED 10 FEET DOWNSTREAM OF GROUT CAP C



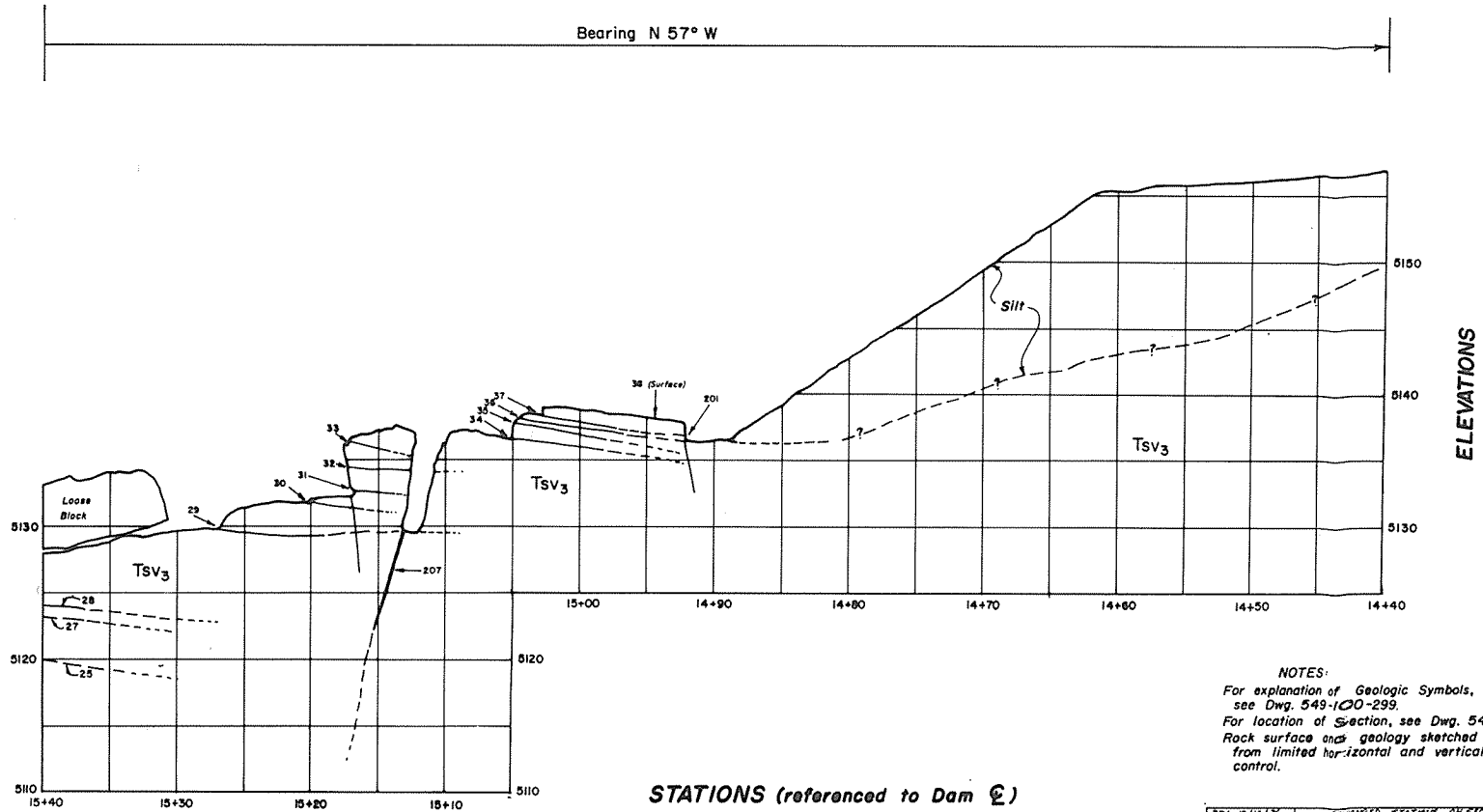
REV. 12-14-76 EDM 100	CHANGED STATIONS ON SECTION AND IN TITLE BLOCK.
ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION TETON BASIN PROJECT LOWER TETON DIVISION - IDAHO TETON DAM GEOLOGIC SECTION C-C' Sta. 15+40 - 16+30	
GEOLOGY BY <i>J.P.</i>	SUBMITTED BY <i>J.P.</i>
DRAWN BY <i>John Ward</i>	RECOMMENDED BY <i>John Ward</i>
CHECKED BY <i>J.P.</i>	APPROVED BY <i>Jack D. [Signature]</i> REGIONAL ENGINEER
BOISE, IDAHO	NOVEMBER 1976
SHEET 5 OF 5	
549-100-290	



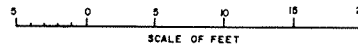
ELEVATIONS

NOTES:
 For explanation of Geologic Symbols, see Dwg. 549-100-299.
 For location of Section, see Dwg. 549-100-299.
 Rock surface and geology sketched in field from limited horizontal and vertical survey control.

REV. 12/1/74 (CHANGED STATIONS ON SECTION) AND IN TITLE BLOCK. EPM 100	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION TETON BASIN PROJECT LOWER TETON DIVISION - IDAHO TETON DAM GEOLOGIC SECTION D-D' Sta. 15+40 - 16+077	
GEOLOGY <i>J. M. E. J.E.</i> DRAWN <i>JOHN WELCH</i> CHECKED <i>J. M. E.</i>	SUBMITTED <i>[Signature]</i> RECOMMENDED <i>[Signature]</i> APPROVED <i>[Signature]</i> SPECIAL ENGINEER
BDISE, IDAHO SHEET 1 OF 4	NOVEMBER 1976 549-100-291



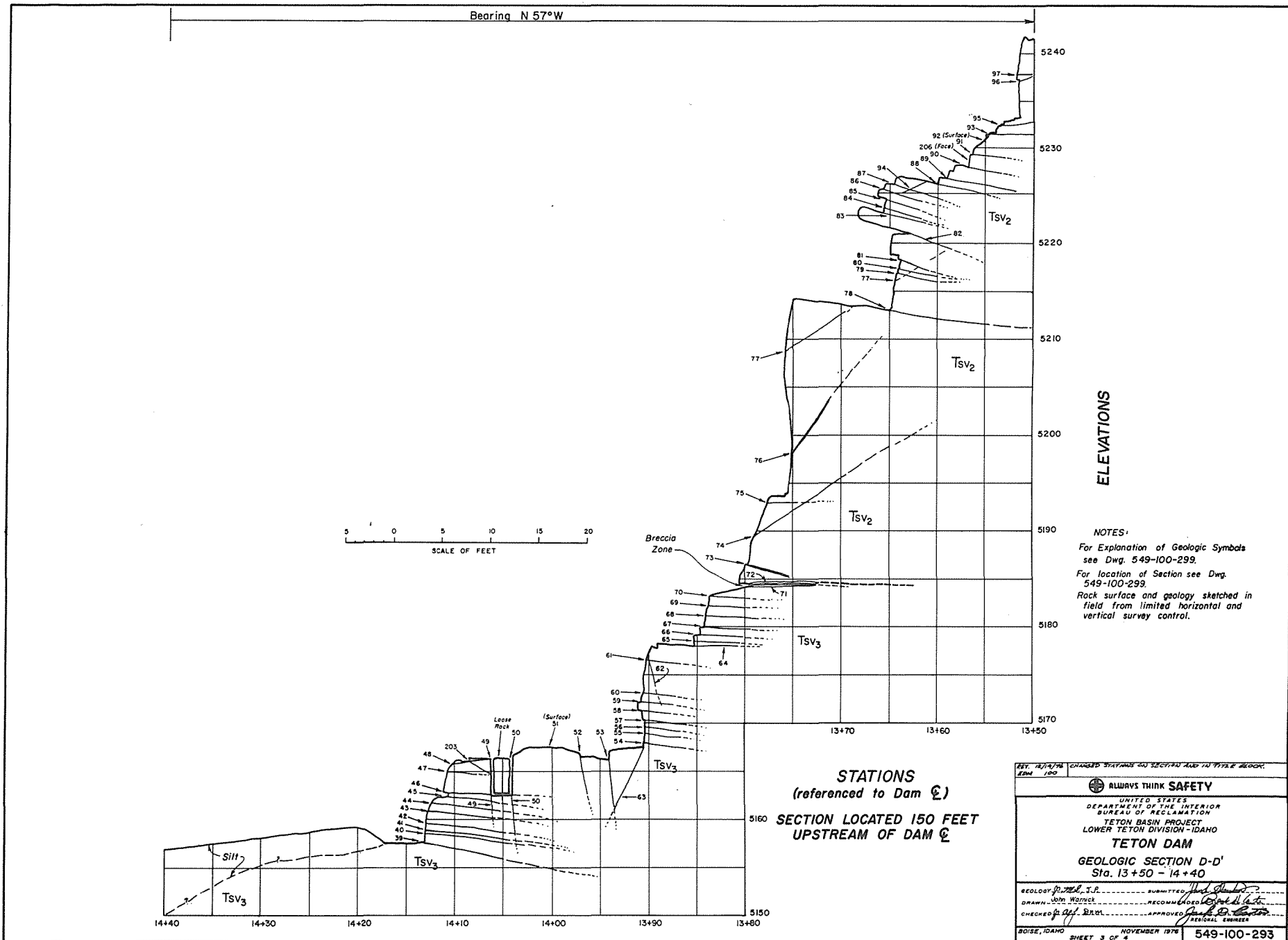
STATIONS (referenced to Dam C)



SECTION LOCATED 150 FEET UPSTREAM OF DAM C

NOTES:
 For explanation of Geologic Symbols,
 see Dwg. 549-100-299.
 For location of Section, see Dwg. 549-100-299.
 Rock surface and geology sketched in field
 from limited horizontal and vertical survey
 control.

REV 12/16/74 ERC 100	CHANGED STATIONS ON SECTIONS AND 1st TITLE BLOCK
ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION TETON BASIN PROJECT LOWER TETON DIVISION-IDAHO TETON DAM GEOLOGIC SECTION D-D' Sta. 14+40 - 15+40	
GEOLOGIST: <i>P. H. J. R.</i> DRAWN: <i>ERC MS/SEL</i> CHECKED: <i>ERC P.J.M.</i>	SUBMITTED: <i>[Signature]</i> RECOMMENDED: <i>[Signature]</i> APPROVED: <i>[Signature]</i> REGIONAL ENGINEER
BOISE, IDAHO	NOVEMBER 1974
SHEET 2 OF 4	549-100-292

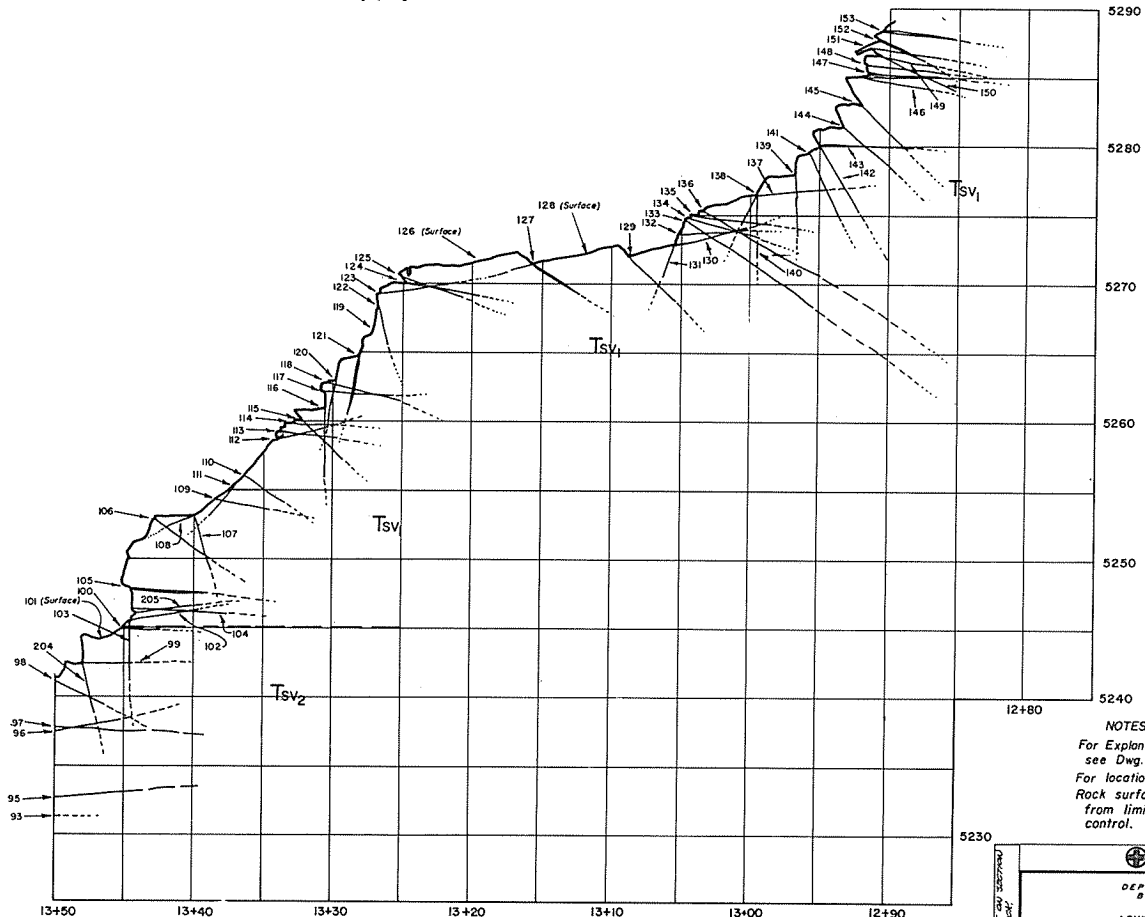


REV. 12/17/76	CHANGED STATIONS AND SECTION ALSO IN TITLE BLOCK
DATE 1/20	
ALWAYS THINK SAFETY	
UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION TETON BASIN PROJECT LOWER TETON DIVISION - IDAHO TETON DAM GEOLOGIC SECTION D-D' Sta. 13+50 - 14+40	
GEOLOGY BY <i>J. E. L. J.S.</i>	SUBMITTED <i>John E. L. J.S.</i>
DRAWN BY <i>John W. White</i>	RECOMMENDED BY <i>John W. White</i>
CHECKED BY <i>J. E. L. J.S.</i>	APPROVED <i>John W. White</i> REGIONAL ENGINEER
BOISE, IDAHO	NOVEMBER 1976
SHEET 3 OF 4	
549-100-293	

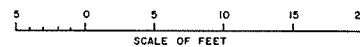
E-18

Bearing N 57° W

The joints shown from Sta. 12+80 to about Sta. 13+30 are the major continuous joints in Tsv₁. Not depicted are the numerous closely spaced, discontinuous and lenticular, near horizontal joints that characterize the unit as notably platy.



STATIONS (referenced to Dam ☉)



SECTION LOCATED 150 FEET UPSTREAM OF DAM ☉

NOTES:

For Explanation of Geologic Symbols, see Dwg. 549-100-299.
For location of Section, see Dwg. 549-100-299.
Rock surface and geology sketched in field from limited horizontal and vertical survey control.

<p>ALWAYS THINK SAFETY</p> <p>UNITED STATES DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION TETON BASIN PROJECT LOWER TETON DIVISION-IDAHO TETON DAM GEOLOGIC SECTION D-D' Sta. 12+75 - 13+50</p>	
<p>GEOLOGY: <i>[Signature]</i>, I.P. DRAWN: John W. Brown CHECKED: <i>[Signature]</i>, I.P.</p>	<p>SUBMITTED: <i>[Signature]</i> RECOMMENDED: <i>[Signature]</i> APPROVED: <i>[Signature]</i>, REGIONAL ENGINEER</p>
<p>BOISE, IDAHO SHEET 4 OF 4 NOVEMBER 1976 549-100-294</p>	

TETON DAM
POST FAILURE ROCK SURFACE JOINT SURVEY

EXPLANATION

ITEM	DEFINITIONS
1. Joint Number (and observation points 1, 2, 3, etc. if needed)	Sequential number from 1 to ? on each photograph, section or plan map.
2. Strike	Strike taken at observation point.
3. Dip	Measured maximum dip at observation point.
4. Continuity (feet)	Measured on surface trace.
D	Discontinuous; < 50'
MC	Moderately Continuous; 50-100'.
C	Continuous; > 100'
	Note: Joints < 10' in length not mapped unless unusual condition existed.
5. Openness	
T	Tight; < 1/2"
O	Open; 1/2-2"
EO	Excessively Open; > 2" (maximum openness noted).
6. Surface Planarity	
P	Planar
I	Irregular
C	Curved
W	Wavy
F	Offset (Range of offset in inches or feet).
7. Roughness	
S	Smooth; deviation from plane < 1/4"
R	Rough; deviation from plane > 1/4"

8. Filling, coating, or stain

A. Natural

IO
MO

CT
CP
CV
SL

B. Grout

GT
GV

9. Remarks

Pertinent observation not covered in above items.

Estimated % of joint surface by each type present.

Iron Oxide (red, yellow, rust).
Manganese oxide (black stain or dendrites).

CaCO₃; < 1/16"

CaCO₃; 1/16-1"

CaCO₃; > 1"

Silt

< 1/4"

> 1/4"

Hydrothermal alteration, evidence of waterflows or seeps, occurrence of unusually thick CaCO₃. Foliation or lineation attitudes.

TETON DAM POST FAILURE JOINT SURVEY

SHEET 1 OF 2

LOCATION Geologic Section A-A' (100' d/s of dam)

DRAWING NO. 549-100280

MAPPED BY J.P., J.S., M.S.

DATE 11-2-76

JT. NO.	STRIKE	DIP	CONT.	OP	PL	RGH	COATINGS OR FILLING (%)						REMARKS									
							IO	MO	CT	CP	CV	SL		GROUT								
1	N27W	86NE	MC	0	P	S	10		40													
2	N14W	75SW	MC	T	P	S	80		15												Open 3/4" - 1"	
3	N27W	74SW	MC	T	P	S	95															
4	N38W	Vert.	MC	T	P	S	95		15													
5	N50E	76SE	D	T	P	R			50													
6	N30W	68SW	MC	T	P	S	40	10	10													
7	N20W	73SW	MC	T	P	S	10		30													
8	N15W	85SW	MC	T	P	R																
9	Horiz	-	C	T	P	S															70% clay in joint	
10	N10W	7NE	D	T	P	S	10														70% clay in joint	
11	N45W	85NE	MC	T	P	S	80															
12	N12W	Vert.	MC	T	P	S	80															
13	Due N	Vert.	D	T-0	I	R							100									
14	N17W	Vert.	D	0 (1/2)	I	R							100									
15	N17W	Vert.	C	0	I	S	70						100								Open to 2" where #13 intersects #14	
16	N47E	9SE	D	T	P	S	10														Open to 2"	
17	N73E	87SE	D	T	P	R	30	10														
18	N27W	76SW	C	0	P	S	70	10					100									Open to 2"
19	N27W	Vert.	C	T-0	P	S	70	10														
20	N17W	Vert.	MC	T	P	S	70															
21	Horiz.			T	P	S	10															
22	N20W	81SW	MC	T	P	S	70	10														

E-21

TETON DAM POST FAILURE JOINT SURVEY

SHEET 1 OF 11

LOCATION Geologic Sections B-B', C-C', and Geologic Overlays

DRAWING NO. 549-100-281 thru 290, 295 thru 298 MAPPED BY J.P., M.S., J.S.

DATE 10-27-76 thru 11-4-76

JT. NO.	STRIKE	DIP	CONT.	OP	PL	RGH	COATINGS OR FILLING (%)						REMARKS	
							IO	MO	CT	CP	CV	SL		GROUT
1	N23W	75NE	D	T	C	S		5						
2	N84E	62SE	D	T	P-C	S								
3	N75E	83NW	D	T	P-C	S							GT 90	
4	N88W	80NE	D	T-0	C	S							GT 60	
5	N27W	85NE	D	T-0	C	S								
6	N20W	84NE	D	T	C	S	10							
7	N83E	77NW	D	T	P-C	S	15							
8	N60E	57SE	D	T	P	S							GT 95	Chalky grout on face.
9	N77E	Vert.	D	T	P	S								
10	N27W	80NE	D	T	P-C	S-R							GT 90	
11	N46W	74NE	D	T	P-W,F	S							GT 95	F = 4"
12	N45W	77SW	D	T	P,F	R							GT 95	F = 1"
13	N54W	78SW	D	T	P	S							GT 95, GV 5	
14-1	N37E	86NW	D	T-0	C	S								
14-2	N7E	83SE	D	T-0	C	S		5						
15-1	N45E	75SE	MC	T	C	S	60							
15-2	E-W	85SW	MC	T	C	S	60							
16	N68E	74NW	D	T	P,F	S							GT 90	F = 1"
17	E-W	86N	D	T	P-C	S							GV - GT	Face 90% grout coated, joint filled.
18	N50W	82NE	D	T	P	S							GT 95	
19	N48E	65SE	D	T	P	S							GT 95	
20	N10W	Vert.	D	T	C	S							GT 1	
21-1	N8W	84NE	C	T	P-W	S	20						GT 50, GV 50	Joint opening filled with grout.
21-2	N9W	84SW	D	T	P	R							GT 95	
22	N18W	78SW	D	T	P-W	R		10					GT 90	
23	N78E	24NW	D	T	P	S								
24	N22E	65NW	MC	T	P	S							GT 90	
25	N57E	71NW	D	T	P-C	S							GV	
26	N18E	80NW	D	T	P-C	R							GV 90	Sandy grout filling 1/2" opening.

E-23

TETON DAM POST FAILURE JOINT SURVEY

SHEET 3 OF 11

LOCATION Geologic Sections B-B', C-C', and Geologic Overlays

DRAWING NO. 549-100-281 thru 290, 295 thru 298 MAPPED BY J.P., M.S., J.S.

DATE 10-27-76 thru 11-4-76

JT. NO.	STRIKE	DIP	CONT.	OP	PL	RGH	COATINGS OR FILLING (%)						REMARKS		
							IO	MO	GT	CP	CV	SL		GROUT	
50	N34W	76NE	D	T	P	S								GV	Grout spotty on face.
51-1	N7W	Vert.	MC	T	P-C	S	10							GT 60, GV 20	
51-2	N6E	87NW	D	T	P-W,F	R								GV	Spotty.
52	N5W	40NE	D	T	C	S	5							GT 10	
53-1	N12W	80NE	D	O	P	S								GV 10	
53-2	N8E	Vert.	D	T	P	S		60						GT 90	Face covered with grout.
54	N69E	Vert.	D	T	P-C	R		60						GT	Spotty grout.
55-1	N6W	73NE	D	T	P-C	S	40								
55-2	N9E	Vert.	D	T	P	S		60						GT 90	
56	N60E	Vert.	D	T	P	S	10								
57	N23E	83SE	D	T	P-C	R								GT 20	F = 4"
58	N10W	Vert.	D	T	P-C	S									
59-1	N20W	73SW	MC	T-0	P	S								GT 70, GV 30	Open to 1"
59-2	N9W	83SW	MC	T	P	S								GT 80	
60	N17W	20SW	MC	T	P	S								GT 70	
60a	N15W	75SW	D	T	P	S			60	20					
60b	N13W	60SW	MC	T	P	S									
61	N-S	56W	MC	T-0	P	R								GV 20, GT 60	Open to 1/2"
62	N65E	Vert.	D	T	P	S									
63	N19E	Vert.	D	T	P	S			80						
64	N14W	67SW	D	T	P	S		10	70						
65-1	N11W	Vert.	MC	T	I	R								GT 95	
65-2	N5E	Vert.	MC	T	I	R								GT 60	
65-3	N25W	44SW	D	T	P	S		5	60	10					
66	N55W	85SW	D	T	P	S								GP 90	
67	N86W	80NE	D	T	P-I	S			10					GP 50	
68	N87W	Vert.	D	T-0	P	S			50						
69	N21W	84NE	D	T	P	S			80						
70	N2W	79SW	D	T	P	S	5	5	60					GP 20	

E-25

TETON DAM POST FAILURE JOINT SURVEY

SHEET 4 OF 11

LOCATION Geologic Sections B-B', C-C', and Geologic Overlays

DRAWING NO. 549-100-281 thru 290, 295 thru 298 MAPPED BY J.P., M.S., J.S.

DATE 10-27-76 thru 11-4-76

JT. NO.	STRIKE	DIP	CONT.	OP	PL	RGH	COATINGS OR FILLING (%)						REMARKS	
							IO	MO	CT	CP	CV	SL		GROUT
71	N13E	73NW	D	T	P-C	S			20				GP 50	
72	N80E	Vert.	D	T	P	S							GP 95	
73	N22W	60SW	MC	T	P-C	S							GT 95	
74	N25W	71SW	D	T	P	S			10				GP 30	
75	N60W	75SW	D	T	P	S							GV 10, GT 70	
76	N52W	80SW	D	T	P	S								
77	N45W	22NE	D	T	P	S	5		5					
78	N15W	76NE	MC	T	P-C	R			30				GT 15	
79	N50E	66SE	D	T	P-C	S							GT 70	
80	N56W	77NE	D	O	P-C	S							GT 70, GV 15	Open to 1"
81	N62W	72NE	D	T	P-C	S							GT 95	
82	N53W	70SW	D	T	P-C	S	5						GT 60	
83	E-W	85SW	D	T	C	S							GT 50	
84	N42E	72SE	D	T	P	R							GT 70	
85	N15W	13SW	MC	T	P	S	5	15						
86	N10W	86NE	D	T	P-C	S			30				GT 5	
*87-1	N45W	57SW	D	T	P-C	S			20				GT 90	F = 2"
88	N62W	75NE	D	T	P, F	S								
89	N25W	73SW	MC	T	P	S							GV 70	
90	N37W	70SW	D	T	P-C	S							GV 30, GT 5	
91-1	N17W	42SW	C	T	I	R	10						GT 80	
91-2	N17E	83SE	C	T	I	R	10						GT 80	
*92-1	N17W	42SW	MC	T	P-C	S								Three subparallel joints.
93	N22W	87SW	D	T	P	S	30		20					
94	N37W	85NE	D	T	C	S							GT 95	
95	N10W	26SW	MC	T	P	S	50							
96-1	N42E	74NW	MC	T	C	S			15				GT 2	
96-2	N50E	50SE	C	T	P-C	S	60							
97	N30W	82SW	MC	T	P	S								
*87-2	N25W	57NE	D	T	P-C	S							Grout	
*92-2	N5W	62SW	C	T	I	R							Grout	

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TETON DAM POST FAILURE JOINT SURVEY

SHEET 5 OF 11

LOCATION Geologic Sections B-B', C-C', and Geologic Overlays

DRAWING NO. 549-100-281 thru 290, 295 thru 298 MAPPED BY J.P., M.S., J.S.

DATE 10-27-76 thru 11-4-76

JT. NO.	STRIKE	DIP	CONT.	OP	PL	RGH	COATINGS OR FILLING (%)							REMARKS	
							IO	MO	CT	CP	CV	SL	GROUT		
98	N-S	65W	MC	T	P	S									
99	N23W	65SW	D	T	P-C	S	70								
100	N60E	81SE	D	T	P-C	S	60		30						
101	N53W	82NE	D	T	P	S									
102	N7W	57NE	D	T	P	S	20	40							
103	N3E	83NW	MC	T	P	S	20	40							
104	E-W	50N	D	T	P	S									
105	N38W	80SW	D	T	I	S-R	20	40							
106-1	N27W	60SW	D	T	P-C	S	10		40						
106-2	N3E	Vert.	D	T	P-C	S			20						
107	N47W	72NE	D	T	P-C	S	70								
108-1	N37W	85NE	MC	T	P	S	10	5	5	20			GT 60		
108-2	N57W	70SW	MC	T	P-C	S	40		30						
109	N37W	43SW	D	T	P	S	20								
110	N10W	80SW	D	T	P-C	S	20								
111	N75E	Vert.	D	T	P	S	30								
112	N40W	Vert.	D	T	P	S									
113	N3W	60SW	D	T	P	S									
114	N5E	85NW	D	O	P	S	10	5		20				Open to 3/4"	
115	N10W	34SW	D	T	P-C	S	20	40	20						
116	N35W	52NE	D	T	P-C	S	20	40							
117	N27W	85SW	D	T	P-C	S	20	40							
118	N15E	81NW	D	T	P-C	S	10								
119	E-W	Vert.	MC	T-0	I	R							GV 15, GT 40	Open to 1/2"	

E-27

TETON DAM POST FAILURE JOINT SURVEY

SHEET 7 OF 11

LOCATION Geological Sections B-B', C-C', and Geological Overlays

DRAWING NO. 549-100-281 thru 290, 295 thru 298 MAPPED BY J.S., J.P., M.S.

DATE 10-31-76

JT. NO.	STRIKE	DIP	CONT.	OP	PL	RGH	COATINGS OR FILLING (%)						REMARKS		
							IO	MO	CT	CP	CV	SL		GROUT	
(1) 147	N16W	77NE	C	T	P	S	30	30						GV=10	
(2) 147	N30W	Vert	C	T	P	S	30	30							
	148	N12W	Vert	D	T-0	P	S	10	20					GV=40	Open to 1/2" and filled with grout, on c, zone open.
	149	N15W	Vert.	D	T	P	S	30	30					GV=10	Open to 1/2" grout.
	150	N9W	80SW	C	T	P	S	30	30						Up to 1/8" grout.
	151	N19W	82SW	C	T	P	S	30	30	10				GV=5 to 1/4"	
	152	N28W	49SW	D	T	P-C	S	10	30						
	153	N13W	83NE	MC	T	P	S								
	154	N47E	82NW	D	T	P	R		30						
	155	N7W	42SW	C	T	P-C	S	10	30	10					
	156	N10W	67SW	D	T-0	P-C	S	20	40						Open to 1/2" & filled with grout.
	157	N23W	Vert.	D	T	P-C	S	10	5	25					
	158	N20W	75NE	C	T	P	S	40							
	159	N17W	Vert.	C	T	P	S		60						
	160	N75E	73NW	D	T	P-C, F	S	15	20						F=2", Trace. GT (chalky)
	161	N67E	Vert.	D	T	P-C	R								
	162	N23W	77SW	C	T-0	P	S								Open to 1/2" and filled with grout.
	163	N40W	Vert.	D	T-0	I	R								Open to 4" & partially filled with grout.
	164	E-W	87S	MC	T	C	S								
	165	N25W	Vert.	MC	T	P	S	40							
	166	N22W	Vert.	MC	T	P, F	S	10							F=12"
(1) 167	N7W	81NE	D	O	I	R									
(2) 167	N57E	85NW	C	T	P, F	S									
	168	N-S	Vert.	D	T	P	S	5	10						
	169	N80W	72NE	D	T	P	S								
	170	N58E	83SE	D	T	P-C	S								
	171	N65W	80NE	D	T-0	P	S								Open 1/2" partially filled with grout.
	172	N20W	Vert.	C	O	P	S								Open to 1/2" filled with silt.

E-29

TETON DAM POST FAILURE JOINT SURVEY

SHEET 8 OF 11

LOCATION Geological Sections B-B', C-C' and Geological Overlays

DRAWING NO. 549-100-281 thru 290, 295 thru 298 MAPPED BY J.S., J.P., M.S.

DATE 10-31-76

JT. NO. 1	STRIKE 2	DIP 3	CONT. 4	OP 5	PL 6	RGH 7	COATINGS OR FILLING (%) 8							REMARKS 9	
							IO	MO	CT	CP	CV	SL	GROUT		
173	E-W	83N	D	T	P	S									
174	E-W	80S	D	T	I	R									
175	N72W	82NE	D	T	P-C	S									
176	N80W	77SW	D	T	P-C	R									
177	N12W	75NE	D	T	I	R									
178	N25W	Vert.	D	T	P	S									
179	N10W	72SW	D	T	P	S									
180	N33W	Vert.	D	T	P	S									
181	N23E	Vert.	D	T	P	S									Open to 1/2", partially filled with silt.
182	N70W	Vert.	D	T											Open to 1/2", partially filled with silt.
183	N-S	83W	D	T	P-C	S									Open to 1/2", partially filled with silt.
184	N34W	Vert.	C	T-0	P-I	S-R									Open to 1" filled with silt, rubble and grout.
185	N20W	86SW	C	T-0	P-C	S	5						20 GT		Open to 3/4" filled with silt and rubble.
186	N13W	77SW	D	T	P	S	10								
187	N85E	88SE	D	T	P	S	10	15							
(1) 188	N34E	54NW	C	T	P-C	S	15								
(2) 188	N12E	70NW	C	T	P-C	S	15								
189	N35W	67NE	D	T	P	S	10	10							
190	N33W	82SW	D	T	P	S	10								
191	N18W	88SW	D	T	P	S	15								
192	N35E	86NW	D	T	P,F	S	10	20							F=6"
193	N10E	60NW	D	T	C-I	S-R	10	5							
194	N50W	82NE	D	T	P,F	S									F=18"
195	N6W	85SW	D	T	P	S									
196	N41W	80NE	MC	T-0	C-I		5	10							Open to 1" filled with rubble
197	N50F	Vert.	D	T	P-C	S									
198	N42W	86SW	D	T	P-C	S	10								
199	N23W	76SW	D	T	P	S									

E-30

TETON DAM POST FAILURE JOINT SURVEY

SHEET 10 OF 11

LOCATION Geological Sections B-B', C-C', and Geological Overlays

DRAWING NO. 549-100-281 thru 290, 295 & 298

MAPPED BY J.P., M.S., J.S., D.H.

DATE _____

JT. NO. 1	STRIKE 2	DIP 3	CONT. 4	OP 5	PL 6	RGH 7	COATINGS OR FILLING (%) 8						REMARKS 9	
							IO	MO	CT	CP	CV	SL		GROUT
H300	Horiz.		D	T	I	R								
H301	Horiz.		D	T	P F	S							GV	F = 1/4"
H302	N70E	7NW	D	O	I	R							GT 90	Open to 1" filled with burlap & grout
H303	N20W	19NE	D	T	C	S							GT 80.	
H304	Horiz.		D	T	P	S-R		20						
H305	N5W	7NE	D	T	P	S							GT 20	
H306	N80E	7SE	D	T	C	S							GT 60.	
H307	N52E	6NW	D	T	P-C	S								
H308	N30E	10NW	MC	T	P	S		20						
H309	N15W	23SW	D	T	P	S		20	40					
H310	N17W	28SW	D	T	P	S		20	40					
H311	N-S	20W	D	T	P	S		20	40					
H312	N11E	21NW	MC	T	P	S		20	40					
H313	N23W	18SW	MC	T	P	S		20	40					
H314	N23W	35SW	MC	T	P	S		20	40					
H315	N20W	17SW	D	T	P-C	S			95					
H316	N53W	6NE	C	T	I	R		20	15		30			
H317	N16W	2SW	MC	T	P	S		20	30					
H318	N40E	6SE	D	T	P-C	S		20	30					
H319	N70E	5SE	D	T	P	S			10					
H320	N16W	2SW	MC	T	P	S		20						
H321	N40W	4SW	MC	T	P	S								
H322	N-S	10E	MC	T	P	S		20	30				GT 10	
H323	N23E	20SE	D	T	P	S								
H324	N35E	10SE	D	T	P	S		10	20				GT 15	
H325	N73E	20SE	D	T	P-C	S		10		5				
H326	Horiz.		D	T	P-C	S		25	20					
H327	N33W	23SW	C	T	P	S							GV 80	
H328	N35E	10NW	D	T	P	S		40						

E-32

TETON DAM POST FAILURE JOINT SURVEY

SHEET 11 OF 11

LOCATION Geological Sections B-B', C-C', and Geological Overlays

DRAWING NO. 549-100-281 thru 290, 295 & 299 MAPPED BY J.P., M.S., J.S., D.H.

DATE _____

JT. NO.	STRIKE	DIP	CONT.	OP	PL	RGH	COATINGS OR FILLING (%)						REMARKS	
							IO	MO	CT	CP	CV	SL		GROUT
H329	N20W	33SW	D	T	P-C	S	20	40						
H330	N20W	23SW	D	T	P	S	20	40						
H331	N5E	30NW	D	T	P-C	S	20	40						
H332	N10W	28SW	D	T	P-C	S	20	40						
H333	N33W	20SW	D	T	P-C	S	5	20						
H334	N12E	36NW	MC	T	P-C	S	10	20						
H335	N24W	52NE	D	T	P-C	S	10	20						
H336	N10W	19NE	D	T	P-C	S	20	40						
H337	Horiz.		D	T	W	I	15	20						
H338	N50E	9NW	D	T-0	W-C	I	40		20	20				
H339	N50E	6SE	C	T	C	S	40							
H340	N40E	4NW	D	T	C	S	30	10						
H341	N85E	3SE	MC	T	P	S	30							
H342	N12W	6SW	MC	T	P	S		10	5					
H343	N25W	7SW	MC	T	P	S								
H344	N30E	28NW	D	T	P-C	S								
H345	N14W	12SW	MC	T	P	S								
H346	N20E	13NW	D	T	P	S	5	5						
H347	N13W	13SW	C	T	P,F	S	10	5						
H348	E-W	3S	D	T	P-C	S								
H349	N12W	13SW	MC	T	P-I	S-R	20	30						
H350	N34W	13SW	MC	T	P-I	S-R								
H351	N17W	12SW	C	T	P,F	S	10	5						F = 4
H352	N15W	16SW	MC	T	P	S								
H353	N47W	11NE	D	T	P	S								

E-33

TETON DAM POST FAILURE JOINT SURVEY

SHEET 1 OF 6

LOCATION Geologic Section D-D' (150' w/s E & dam)

M.S., J.S., J.P., D.S.,

DRAWING NO. 549-100-291,292

MAPPED BY D.H., A.L., W.C.W., P.A.H. DATE 11-3-76

JT. NO.	STRIKE	DIP	CONT.	OP	PL	RGH	COATINGS OR FILLING (%)							REMARKS	
							IO	MO	CT	CP	CV	SL	GROUT		
1	N30E	90	D	T	P	R									
2	N86E	8NW	D	T	P	S									Foliation surface.
3	N48W	71NE	D	T	P	S									
4	N21W	9SW	C	T	P	S									
5	N38E	90	D	T	P	S									
6	N14W	14SW	MC	T	P	S									
7	N7W	13SW	D	T	P	S									
8	N6E	14NW	D	T	P	S									
9	N6W	18SW	D	T	P	S									
10	N2E	20NW	MC	T	I	R									Foliation surface.
11	N6W	8SW	D	T	P	R									
12	N4W	11SW	D	T	P	R									
13	N50W	16SW	MC	T	P	S									
14	N5W	12SW	P	T	P	S									
15	N5W	12SW	D	T	P	S									
16	N60W	64NE	MC	T	P	S									
17	N5W	5SW	D	T	P	S									
18	N55E	72SE	D	O	P	S									Down dropped, open to 2".
19	N5E	9NW	D	T	P	S									
20	N12E	7NW	D	T	P	S									
21	N6E	4NW	D	T	P	R									
22	N30E	85NW	D	T	P	R									
23	N4E	8NW	D	T	P	R									
24	N36W	84NE	C	T	P	S									
25	N4E	9NW	MC	T	I	R									
26	N5E	7NW	D	T	I	R									
27	N5E	7NW	MC	T	I	R									Foliation surface.
28	N5E	7NW	MC	T	I	R									Foliation surface.
29	N5E	5SW	MC	T	I	R									Foliation surface.

E-34

TETON DAM POST FAILURE JOINT SURVEY

SHEET 2 OF 6

LOCATION Geologic Section D-D'
DRAWING NO. 549-100-292,293

M.S., J.S., J.P., D.S.,
MAPPED BY D.H., A.L., W.C.W., P.A.H. DATE 11-3-76

JT. NO. 1	STRIKE 2	DIP 3	CONT. 4	OP 5	PL 6	RGH 7	COATINGS OR FILLING (%) 8						REMARKS 9	
							IO	MO	CT	CP	CV	SL		GROUT
30	N4E	8NW	D	T	I	R								
31	N20E	4NW	D	T	I	R								
32	N20E	4NW	MC	T	I	R								
33	N14W	20SW	D	T	I	R	20							Foliation.
34	N12W	9SW	MC	T	I	R								
35	N7E	11NW	MC	T	I	S								
36	N7E	11NW	MC	T	I	S								
37	N7E	11NW	MC	T	I	S								
38	N10E	9NW	D	T	P	S								
39	N5W	14SW	MC	T	I	R								Foliation.
40	N15E	6NW	MC	T	I	R								Foliation.
41	N15E	6NW	MC	T	I	R								Foliation.
42	N15E	6NW	MC	T	I	R								Foliation.
43	N15E	6NW	MC	T	I	R								Foliation.
44	N15E	6NW	MC	T	I	R								Foliation.
45	N15E	6NW	MC	T	I	R								Foliation.
46	N15E	6NW	MC	T	I	R								Foliation.
47	N17W	7SW	D	T	I	R								
48	N54E	8SE	D	T	I	R								
49	N29W	77NE	MC	T-0	P	S								
50	N6W	86SW	C	T-0	P	S								
51	N45W	5SW	D	T	I	R								
52	N25W	84SW	D	T	P	S								
53	N75E	86NW	D	T	P	S								
54	N35E	6NW	D	T	P	S				35				
55	N35E	6NW	D	T	P	S				35				
56	N35E	6NW	D	T	P	S				35				
57	N35E	6NW	D	T	P	S				35				
58	N35E	6NW	D	T	P	S				35				

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TETON DAM POST FAILURE JOINT SURVEY

SHEET 3 OF 6

LOCATION Geologic Section D-D'
DRAWING NO. 549-100-293,294

M.S., J.S., J.P., D.S.,
MAPPED BY D.H., A.L., W.C.W., P.A.H. DATE 11-3-76

JT. NO. ₁	STRIKE ₂	DIP ₃	CONT. ₄	OP ₅	PL ₆	RGH ₇	COATINGS OR FILLING (%) ₈							REMARKS ₉	
							IO	MO	CT	CP	CV	SL	GROUT		
59	N15W	8SW	D	T	P	S			35						
60	N35E	6NW	D	T	P	S			35						
61	E-W	N12	D	T	P	S									
62	E-W	82N	D	T	P	S		50	40						
63	N80W	80NE	D	T	P	S	80								
64	N60W	5SW	D	T	P	S	80								
65	N82W	8SW	D	T	P	S		40	10						Foliations.
66	N82W	8SW	D	T	P	S		40	10						Foliations.
67	N82W	8SW	D	T	P	S									Foliations.
68	N82W	8SW	D	T	P	S		40	10						Foliations.
69	N82W	8SW	D	T	P	S		40	10						Foliations.
70	N82W	8SW	D	T	P	S		40	10						Foliations.
71	N56E	12SE	C	O	W	I					50				Lower contact, Breccia Zone.
72	N18E	8NW	C	O	W	I					50				Upper contact, Breccia Zone.
73	N4E	18NW	C	O	W	I					40				0 = 1"
74	N21E	36SE	C	T	P	S	80								
75	N60W	5NW	D	T	P	S				50					
76	N72W	80NE	MC	O	W	I									0 = 1"
77	N70W	70NE	MC	T	P	I									Vertical foliation.
78	N20W	15SW	C	T	W	I	20	40							
79	N30E	12NW	D	T	W	S	50	20							
80	N30E	12NW	D	T	W	S	50	20							Smooth concoidal jointing.
81	N9W	26SW	D	T	W	S	50	20							
82	N9W	26SW	D	T	W	S									
83	N24W	24SW	D	T	W	S		70							
84	N3W	21SW	D	T	W	S									
85	N4E	21NW	D	T	W	S									
86	N15W	25SW	D	T	W	S									
87	N6W	22SW	D	T	W	S									

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TETON DAM POST FAILURE JOINT SURVEY

SHEET 4 OF 6

LOCATION Geologic Section D-D'
DRAWING NO. 549-100-293,294

M.S., J.S., J.P., D.S.,
D.H., A.L., W.C.W., P.A.H. MAPPED BY
DATE 11-3-76

JT. NO. 1	STRIKE 2	DIP 3	CONT. 4	OP 5	PL 6	RGH 7	COATINGS OR FILLING (%) 8						REMARKS 9	
							IO	MO	CT	CP	CV	SL		GROUT
88	N22W	22SW	D	T	P	S	10	90						
89	N14W	13SW	MC	T	P	S		95						
90	N14W	15SW	D	T	P	S		90						
91	N39W	16SW	D	T	P	S		95						
92	N88E	45SE	D	T	P	S		95						
93	N44W	14SW	D	T	P	S		95						
94	N70W	68NE	D	O	P	S		90						0 = 1"
95	N33W	9NE	MC	T	P	R	60	5						
96	N18W	18NE	D	T	P	R	45	50						
97	N10E	3NW	MC	T	P	R	90	10						
98	N40W	66SW		T	W	S	90	10						
99	N45W	4SW		T	W	S	90	10						
100		0		T	W	S	80							
101	E-W	35N		T	W	S	60							
102	N45E	10SE		T	W	S	70							
103	N10W	90		T	P	S	40	20	10					
104	N15W	5SW		T	W	S								
105	N18W	8SW		O	W	S			40					0 = 1"
106	N85E	54NW	D	T	P	S			75					
107	N84E	79NW	D	T-0	P	S			75					0 = 1/4" - 1/2"
108	N87E	37SE	D	T	P	S			15					
109	N24W	20SW	D	T-0	C	S								0 = 1/4"
110	N35W	60SW	D	T	C	S				70				

E-37

TETON DAM POST FAILURE JOINT SURVEY

SHEET 5 OF 6

LOCATION Geologic Section D-D'
DRAWING NO. 549-100-294

M.S., J.S., J.P., D.S.,
MAPPED BY D.H., A.L., W.C.W., P.A.H. DATE 11-3-76

JT. NO.	STRIKE	DIP	CONT.	OP	PL	RGH	COATINGS OR FILLING (%)							REMARKS	
							IO	MO	CT	CP	CV	SL	GROUT		
111	N56E	53SE	D	T	P	S					20				
112	N70E	18SE	D	T-0	C-W	S	10L	20							0 = 1/4" - 1/2"
113	N14W	10SW	D	T	P	S		20							Between joints 113 & 119, several joints
114	N52W	19SW	D	T	C-W	S		40	5						spaced 2"-4" apart, attitudes range
115	N40E	44NW	D	T	C	S			60						between that of 113 & 114, joints are
116	N10E	Vert.	D	T	T	R									smooth, curving and tight.
117	N20W	7SW	D	D-0	P	S						70			0 = 1/2"
118	N10E	11SW	D	T	P	S									
119	N20E	85SE	D	T	C	S									
120	N4W	81NE	D	T	I	R									
121	N84E	83SE	D	T-0	I	R									0 = 3/4"
122	N27E	78NW	D	T	C	S	15								
123	N57E	9SE	D	T	C	S-R									
124	N82W	16SW	D	T	C	S	10		10						
125	N60E	20NW	D	T	C	S			50						
126	N35E	18SE	D	T	C-W	S				35					
127	N32E	33NW	D	T-0	P	S			20						0 = 1/4" - 1/2"
128	N59E	7SE	C	T	C-W	S			25						
129	N30E	45NW	D	T	I	S-R									
130	N35E	14SE	D	T	C-I	S			10						
131	N67E	73SE	D	T	I	R				30					
132	N83W	10SW	D	T-0	C-W	S									0 = 1/4"
133	N47E	34NW	MC	T	P	S				20					
134	N15E	17NW	D	T	C	S									
135	N30E	6NW	D	T	C	S									
136	N25E	30NW	MC	T	P-W	S			10						
137	N41E	5SE	D	T	C	S		40							
138	N54E	66SE	D	T	C	S			60						
139	N48E	Vert.	D	T-0	P	S									0 = 1/4"

E-38



Fig. E-1 Right wall of Teton Canyon upstream of dam axis showing intensive jointing in uppermost rock zone. (post-failure)

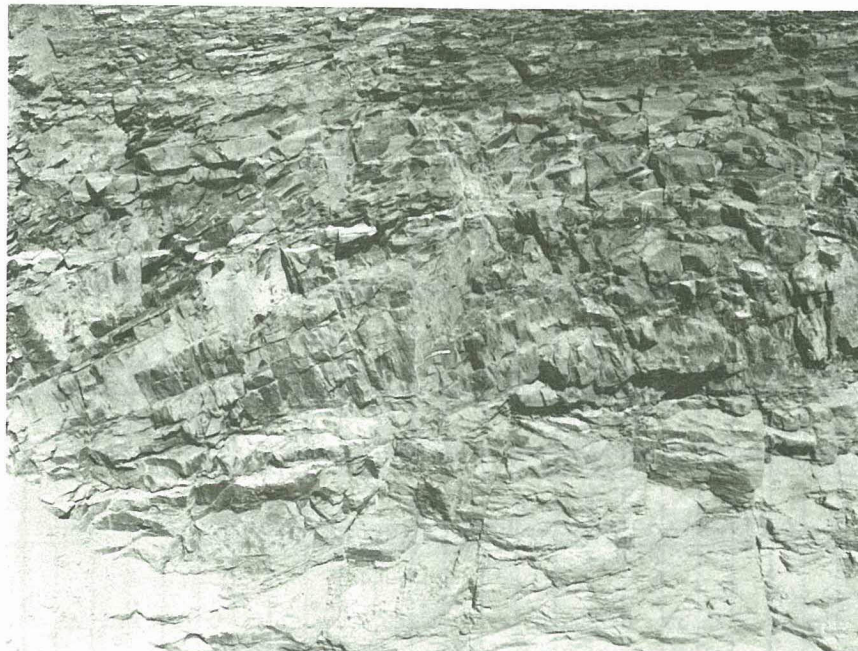


Fig. E-2 Ash-flow tuff in right wall of Teton Canyon upstream of key trench. Note near-horizontal planes of separation. (post-failure)

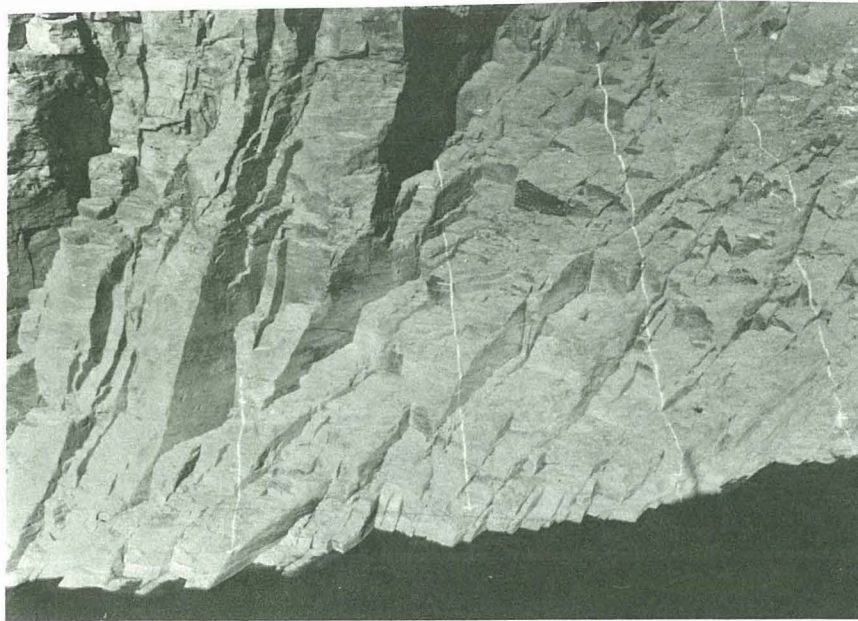


Fig. E-3 Right wall of Teton Canyon downstream from grout cap. (post-failure)

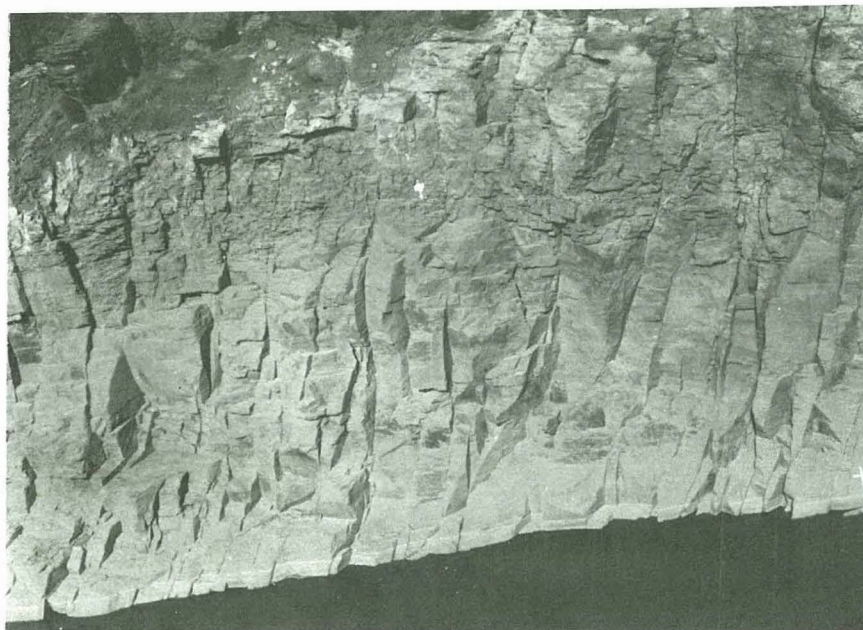


Fig. E-4 Right wall of canyon between dam and spillway stilling basin. (post-failure)

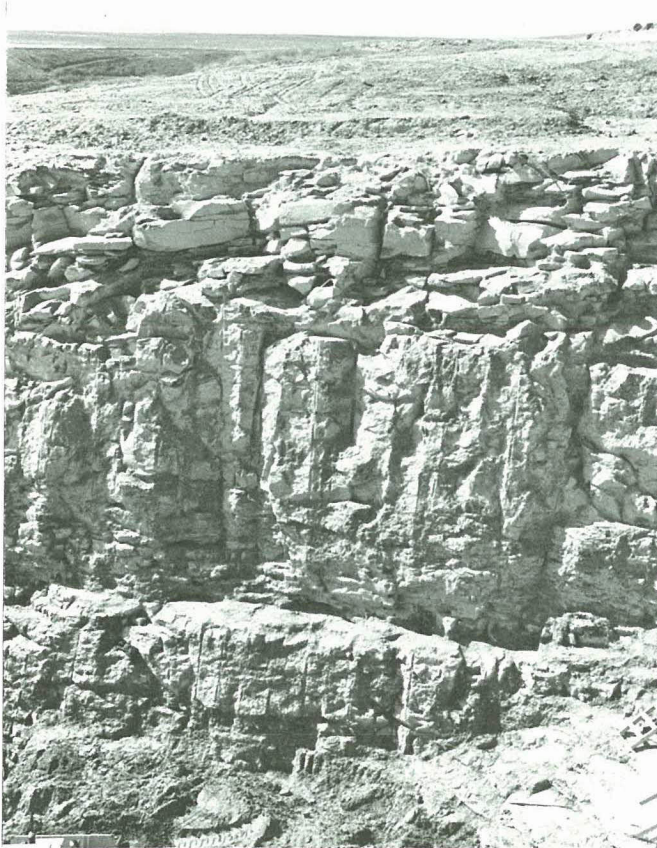


Fig. E-5 Downstream face of key trench in right abutment. (post-failure)

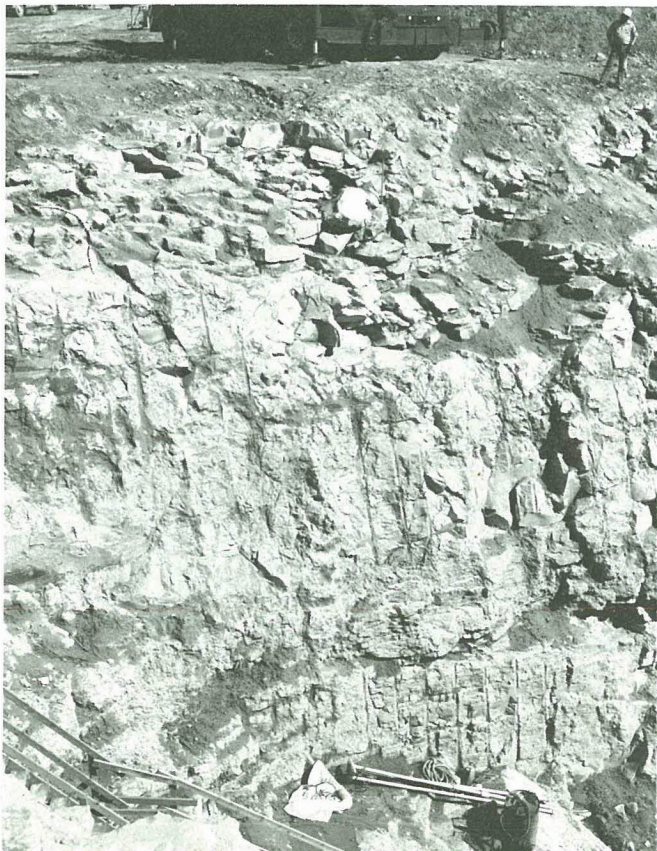


Fig. E-6 Upstream face of key trench in right abutment. (post-failure)



Fig. E-7 Downstream face of key trench in right abutment at El. 5290.
(post-failure)



Fig. E-8 Intensively jointed ash-flow tuff in downstream
face of right abutment key trench at El. 5290.
(post-failure)

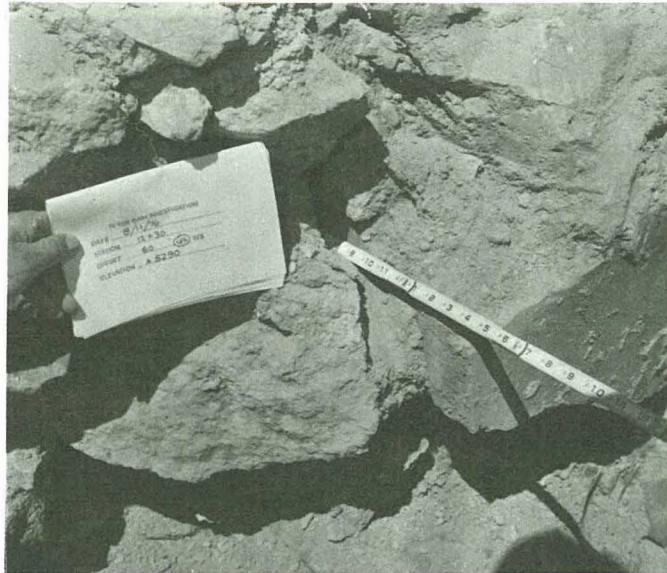


Fig. E-9 Open joint in upstream face of right abutment key trench at Sta. 12+40, El. 5290. (post-failure)

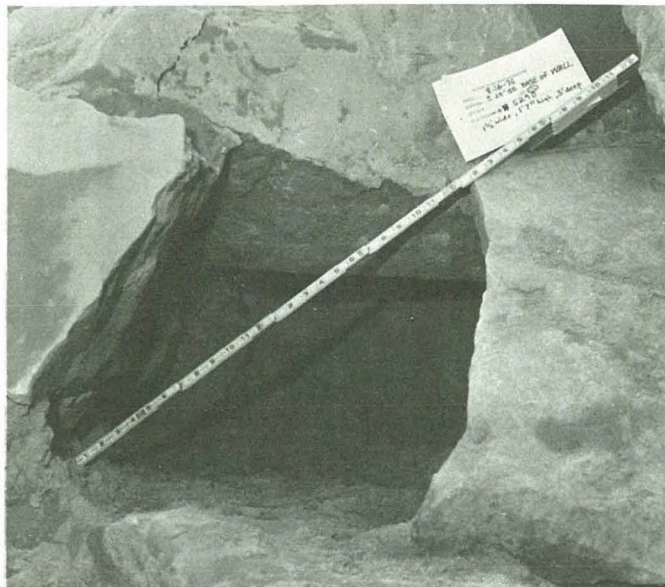


Fig. E-10 Open joint in downstream face of right abutment key trench at El. 5290. (post-failure)



Fig. E-11 Downstream of wall of right abutment key trench. Elevation in center of photo is about 5290. (post-failure)



Fig. E-12 View along grout cap; key trench, and spillway in background. (post-failure)



Fig. E-13 Rim of upstream wall of right abutment key trench. (post-failure)



Fig. E-14 Upstream face of right abutment key trench at Sta. 12+70. Fill at El. 5240. (post-failure)



Fig. E-15 Upstream wall right abutment key trench at Sta. 12+65. Elevation of fill is 5240. (post-failure)



Fig. E-16 Downstream wall of right abutment key trench. (post-failure)



Fig. E-17 Key trench excavation on right abutment. (post-failure)



Fig. E-18 Right abutment of dam immediately downstream from grout cap. (post-failure)

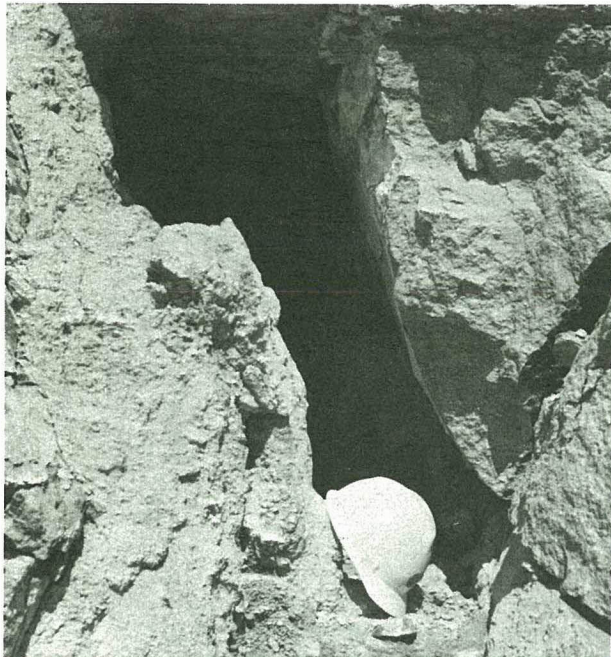


Fig. E-19 Fissure at rim of right wall of canyon 1/8 to 1/4 mile upstream from dam. (post-failure)

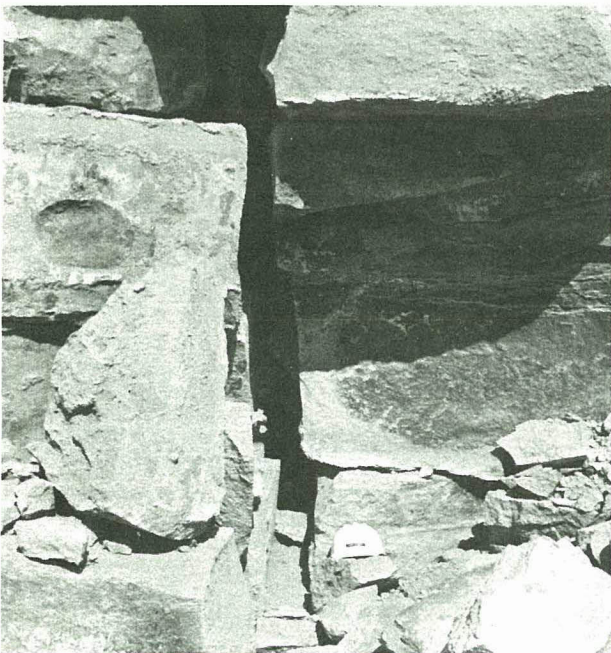


Fig. E-20 One of several large fissures near rim of canyon right wall 1/8 to 1/4 mile upstream from dam. (post-failure)



Fig. E-21 Prominent rock joints in vicinity of missing grout cap segment — bottom of photo. (post-failure)

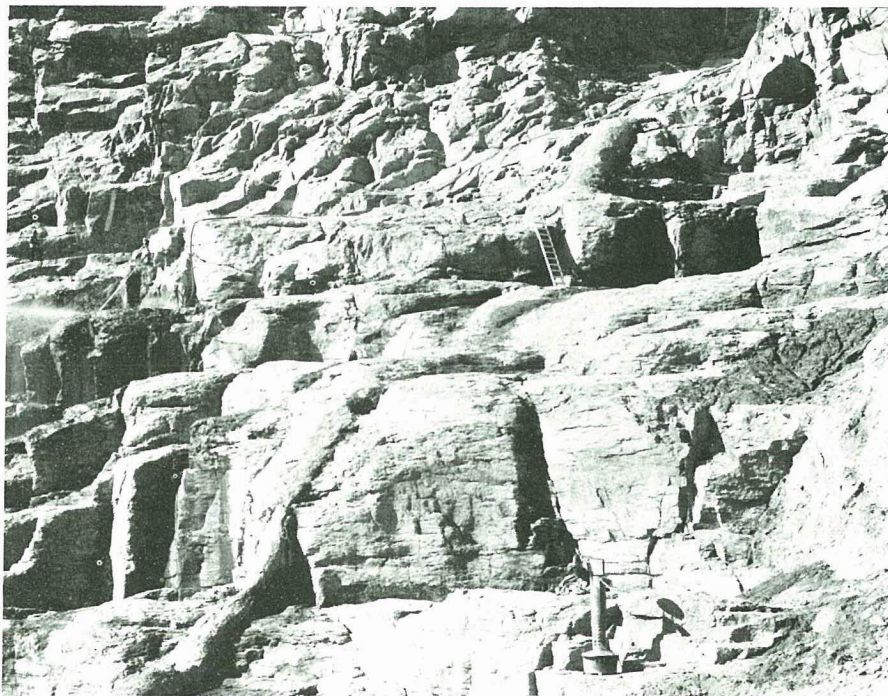


Fig. E-22 Photo overlaps Fig. E-21. (post-failure)



Fig. E-23 Joint system near missing segment of grout cap Sta. 14+00 behind ladder in lower center. (post-failure)



Fig. E-24 View across grout cap toward prominent joint near Sta. 13+30. (post-failure)

March 14, 1974

Memorandum

To: Director of Design and Construction, Denver, Colorado
Attn: 1300 and 220

From: Project Construction Engineer, Newdale, Idaho

Subject: Proposed Treatment of Fissures and Cavities in Right
Abutment Key Trench - Specifications No. DC-6910 -
Morrison-Knudsen-Kiewit, Teton Dam, Power and Pumping
Plant, Teton Project, Idaho

The geology of the fissures and cavities which have recently been exposed in the excavation for the right abutment key trench is described in the attached report. Preliminary drawings numbers 549-147-131 and 549-147-132, and photographs of the fissure zones and cavities are also included.

The following proposed treatment of the fissure zones and related cavities as discussed with members of your staff is summarized as follows:

1. Locate the cavities with pilot angle holes upstream and downstream from the foundation key trench using an air-trac drill set up on the original ground surface. The estimated pilot hole footage is about 500 lin. ft.
2. Drill 10-inch diameter holes (8" casing) to intersect cavities at locations determined by the pilot drilling and approximately as shown on Drawing No. 549-147-131. Ten-inch diameter holes as follows:
 - a. One 10-inch diameter hole to intersect cavity in fissure zone at Station 4+44 upstream. The estimated depth of this hole is 60 feet.
 - b. One 10-inch diameter hole to intersect cavity in continuation of above fissure zone at Station 4+21 downstream. The estimated depth of this hole is 70 feet.

- c. One 10-inch diameter hole to intersect cavity in fissure zone at Station 3+66 upstream. The estimated depth of this hole is 70 feet.
 - d. The need for a 10-inch diameter hole in the continuation of the above fissure zone downstream at Station 3+45 is questionable; however, the final determination of the need for a larger hole in this area should be based on the results of the pilot hole drilling.
3. Fill the cavities with high slump backfill concrete discharged into the 10-inch diameter holes (8" cased) described above. Discussions with the prime contractor indicate that backfill concrete using a 4-bag mix will be the most economical filler material for these cavities. It is anticipated that a local ready mix concrete supplier will furnish the concrete to the prime contractor at a substantially lower price than can be batched on the job with the contractor's batching facilities.
 4. Place nipples in the voids along fissure zones in the bottom of foundation key trench and embed in concrete during placement of grout cap. Trenches 3 to 5 feet deep and about 3 feet wide have been excavated along the strike of the two main fissure zones as shown on drawing No. 549-147-132. Nipples will be placed in open joints or holes in the floor of the key trench near centerline at Stations 5+03, 5+68, and 6+18; and about five feet left of centerline between Stations 6+03 and 6+08.
 5. Intersect fissure zones at various depths in the bottom of key trench with grout holes, then grout voids using grout mixes and procedures previously established on the project for grouting similar areas.

The estimated cost for accomplishing this proposed work is as follows:

<u>Work or Material</u>	<u>Quantity</u>	<u>Unit</u>	<u>Price</u>	<u>Amount</u>
1. Mobilization and demobilization of drill equipment	For the lump sum of			<u>1,000.00</u>
2. Air-trac pilot holes	300	lin.ft.,	5.00	<u>1,500.00</u>
3. Ten-inch diameter holes with 8-inch casing	200	lin.ft.,	35.00	<u>7,000.00</u>
4. Backfill concrete	350	cu.yds.,	30.00	<u>10,500.00</u>
5. Block cavern entrances	For the lump sum of			<u>1,000.00</u>
Total estimated cost				<u>\$ 21,000.00</u>

It is critical that this work begin as soon as possible to avoid delaying the contractor in his scheduled grouting program on the right abutment. Backfilling of the cavern areas with concrete should precede grouting to prevent leakage of more costly grout into the large voids. The contractor has indicated that pilot drilling could begin during the week of March 18 and begin filling the cavities in early April.

I suggest that representatives from your office visit the project during the week of March 18 for an examination of the fissure and cavity zones and discuss with our staff the proposed treatment of these areas. To expedite the early commencement of the treatment work, it is requested that authority be granted this office to proceed with price negotiations with the contractor.

Your early reply would be appreciated.

Enclosures

cc: Regional Director, Boise, Idaho
Attn: 200 w/Enc.

Note to Regional Engineer:

We would appreciate having a member of your staff present during the visit of the Denver Office representatives to the Teton Project.

R Robison:lc 3-14-74

bc: AO, PCE, OE, EE, Aberle

Attachment to letter dated March 14, 1974 addressed to Director of Design and Construction, Denver, Colorado, from Project Construction Engineer, Newdale, Idaho.

GEOLOGY

The excavation for the right abutment keyway trench has disclosed two unusually large fissures that cross the floor and extend into the walls of the keyway near the toe of the walls. On the floor of the keyway, the fissures are filled with rubble; but at both locations, the contractor has excavated a trench about three to four feet wide and about five feet deep. Both fissures apparently were developed along joints that strike about N80°W and are vertical to steeply inclined. The largest fissure crosses the keyway from station 4+44 of the upstream face to station 4+21 at the downstream face. The other crosses from station 3+66 on the upstream face to station 3+45 on the downstream face. A smaller fissure strikes about N75°W and crosses the keyway trench from station 5+33 of the upstream face to station 5+11 at the downstream face.

The largest and most extensive open zone extends into the upstream wall from the toe of the keyway wall near station 4+44. The opening at the toe is about five feet wide and three feet high. There is a rubble-filled floor about four feet below the lip of the opening. A few feet in from the wall the fissure is about seven feet wide, but a very large block of welded tuff detached from the roof and/or the north wall rests in the middle. Beyond the large block about 20 feet in from the opening the fissure narrows to about 2½ feet wide. The rubble floor slopes gently away from the opening and the vertical clearance is about ten feet. About 35 feet in, the rubble floor slopes rather steeply and the roof swings sharply upward. About 50 feet in from the opening, the vertical clearance is about 40 feet and the fissure curves out of view at the top. About 75 feet back the fissure curves slightly southward out of view. The smaller fissure is mostly rubble filled and is open only at the upstream face. The opening is about one foot square at the face and the fissure appears to be rubble filled about five feet back from the face.

The continuation of this fissure intersects the downstream wall of the keyway near station 4+21. The opening is about four feet wide and four feet high. A rubble-filled floor lies about four feet below the lip of the opening. The large opening only extends about five feet back from the face then a foot wide fissure at the north edge continues about ten feet back and about ten feet upward before going out of view.

The other large open zone extends into the upstream wall from the toe of the wall near station 3+66. The opening at the toe of the wall is about 1½ feet wide and 1½ feet high. From the opening, the fissure extends about 10 feet down to a rubble floor and about 15 feet back before going out of view. The continuation of this fissure intersects the downstream wall of the keyway at about station 3+45. There is no open fissure at the downstream wall but

there is a 3.5 feet wide zone of very broken rock with open spaces up to 0.8 foot wide. About 2.5 feet north, there is an open joint about 10 feet long and 0.2 foot wide that dips about 78 degrees south.

At both the upstream and downstream locations of the fissure zones, broken rock extends to about midway up the keyway walls. Above the broken zones there appears to be filled fissures about 0.5 foot wide that extend vertically to the top of the keyway cut.

Other open joints or holes were observed on the floor of the keyway near centerline at stations 5+03, 5+68, and 6+18 and about five feet left of centerline between stations 6+03 and 6+08. The holes were rubble filled at shallow depths and their lateral extent, if any, was covered by rubble. Heavy calcareous deposits were associated with all of the open zones except for a sharp, 0.2 foot wide open joint between stations 6+03 and 6+08.

The fissures have developed along planes of weakness, probably joints but possibly faults in the welded tuff. Ancient fumarole activity probably was common throughout the region considering the volume of trapped gases that would have been associated with the ash flow or flows that formed the welded tuff. Violent hot spring and geyser activity probably occurred in connection with the fumaroles. Such activity is believed to have occurred in the dam-site area in both the right and left abutments of the dam a few hundred feet back from the canyon. The ancient Teton River apparently had some controlling influence since there is no evidence of such activity in the welded tuff that forms the canyon walls. A high area of pre-welded tuff sediments upstream from the inlet portal of the river outlet works also may have been related to the hot spring and geyser activity. The thinner body of the welded tuff cooled more rapidly and the sediments formed a base for water moving through the welded tuff.

In the initial fumarole activity, hydrothermal fluids developed from the ash flow deposit are believed to have caused alteration and some opening along joints and zones of weakness in the welded ash flow tuff. Ground water moving through the welded tuff probably became superheated and built up a considerable pressure, very likely the release of pressure that occurred when the water reached joints or vents that were open to the surface, causing the water, at least in part, to flush to steam and to move with great force through the joints or vents and surface as violent hot springs or geysers. The violent activity is believed to have eroded the softer, hydrothermally-altered rock adjacent to existing open joints and vents. Continued activity further altered and eroded the rock, resulting in open fissures and vents that have been encountered in the abutments. Apparently the large fissures developed in zone of numerous smaller, interconnected fissures. Blocks of welded tuff isolated by the small fissures eventually collapsed into the void spaces created by the small fissures to develop the large fissures floored by welded tuff rubble.

APPENDIX F

POST-FAILURE EXPLORATION

DRILL LOGS

<u>Hole Designation</u>	<u>Appendix No.</u>
DH601	F-1
DH602	F-2
DH603	F-3
DH604	F-4
DH605	F-5
DH606	F-6
DH606A	F-6A
DH607	F-7
DH607A	F-7A
DH608	F-8
DH609	F-9
DH610	F-10
DH611	F-11
DH612	F-12
DH613	F-13
DH614	F-14
DH615	F-15
DH616	F-16
DH617	F-17
DH618	F-18
DH619	F-19
DH620	F-20
DH621	F-21
DH622	F-22
DH623	F-23
DH624	F-24
DH625	F-25
DH626	F-26
DH627	F-27
DH628	F-28
DH629	F-29
DH630	F-30
DH631	F-31
DH632	F-32
DH650	F-33
DH650 Hole Survey	F-33A
DH651	F-34
DH651B	F-34A
DH652	F-35
DH652 Hole Survey	F-35A

WATER PRESSURE TEST RESULTS

DH-601 thru 609	F-36
DH-610 thru 632	F-37
DH650	F-38
DH651	F-39
DH652	F-40

APPENDIX F-1

HOLE DESIGNATION: DH-601
 BEARING: N74°W
 ANGLE: 30° to right from vertical
 DEPTH: 109.2 ft.

LOCATION: Right bay spillway
 at STA 10+60.4 2 ft.
 downstream of up-
 stream grout curtain
 ELEVATION: 5299
 Started: 9/8/76
 Finished: 9/11/76

LOG:

0-5.4 ft. Concrete spillway apron (good bond with rock)
 5.4-bottom WELDED ASH-FLOW TUFF
 5.4-20.0 light gray, slightly porphyritic with phenocrysts up to 1/16" in diameter, low density rock
 20.0-30.0 prominent "banding", red-purple tint to a medium gray, "bands" are flattened pumice fragments and aligned vesicles with probable vapor phase minerals
 30.0-101.0 light to medium gray, slightly porphyritic
 101.0-109.2 medium to dark gray, markedly vesicular, vesicles flattened 60° to core axis

Core breaks:

5.4'-20.0' typical core fragment is 0.8' long, range 0.1'-1.0', most breaks are planar, oriented 60° to the core axis, and have a dark surface stain
 20.0'-30.0' typical core fragment is 0.4' long, range from less than 0.1' to 1.0', most breaks are smooth-surfaced, planar or arcuate joints
 30.0'-109.2' typical core fragment is 1.5' long, range 0.4'-4.0'; most breaks are planar, oriented at an angle of 60° to the core axis

Prominent Features

9.2' chalky grout to 1/8" filling a smooth planar joint oriented 60° to the core axis
 40.1'-40.7' grout to 1/8" filling a rough-surfaced, irregular fracture oriented roughly 10° to the core axis
 52.0'-54.7' grout to 1/4" filling a rough-surfaced, irregular fracture which is roughly parallel to the core axis
 62.3'-63.5' grout to 1/8" filling a smooth-surfaced, planar to arcuate joint roughly parallel to the core axis
 68.3'-69.8' calcite (0.5' thick) over silt (0.1' thick), over sand filling a rough fracture roughly parallel to the core, sand terminates on a smooth, planar joint surface oriented 35° to the core axis; sand grain-size increases downward
 77.2'-78.0' grout to 1" filling a rough-surfaced fracture oriented about 30° to the core axis, bottom portion of fracture filled with layered medium sand
 79.6'-80.2' intersection of a smooth planar joint and a rough-surfaced planar fracture, both calcite coated and oriented 30° to the core axis

APPENDIX F-2

HOLE DESIGNATION: DH-602
BEARING: ---
ANGLE: Vertical
DEPTH: 94.7 ft.

LOCATION: Right spillway bay
at STA 10+63.4 2 ft.
downstream of up-
stream grout curtain
ELEVATION: 5299
Started: 9/4/76
Finished: 9/8/76

LOG:

0-4.7 ft. Concrete spillway apron (good bond with rock)
4.7-bottom WELDED ASH-FLOW TUFF
4.7-17.2 light gray, slightly porphyritic with phenocrysts to 1/16" in diameter; light weight rock with scattered pumice fragments
17.2-30.0 pinkish-gray, with prominent light-colored bands oriented roughly perpendicular to the core axis
30.0-94.7 medium to dark gray, slightly porphyritic

Core breaks:

4.7'-17.2' typical core fragment is about 0.8' long, range 0.3'-1.3', most breaks are planar and are roughly perpendicular to the core axis
17.2'-30.0' typical core fragment is about 0.3' long, range from less than 0.1' to 0.8', most breaks are smooth-surfaced, planar, and roughly perpendicular to the core axis
30.0'-94.7' typical core fragment is about 1.8' long, range from less than 0.1' to more than 5.0'

Prominent Features

4.7'-7.8' grout to 1/8" filling a calcite-lined arcuate joint parallel to the core axis
7.8'-8.1' pumice fragment
11.1'-11.6' pumice fragment
13.0'-14.2' minor grout in a rough-surfaced arcuate joint roughly parallel to the core axis
43.0'-43.5' vesicular zone, vesicles flattened approximately 80° to the core axis
49.1'-49.5' vesicular zone, vesicles flattened approximately 80° to the core axis
70.6'-73.9' rough-surfaced, irregular fracture with vapor phase coatings; core broken from 71.0'-72.3'
74.3'-76.1' rough-surfaced, irregular fracture with vapor phase coatings; core broken from 74.4'-76.8'
77.4'-78.1' grout to 1/8" thick partially filling a rough-surfaced irregular fracture oriented 10° to core axis
88.3'-89.7' rough-surfaced, irregular fracture approximately parallel to the core axis, core broken 89.5'-89.7'

APPENDIX F-3

HOLE DESIGNATION: DH-603
 BEARING: S74E
 ANGLE: 30° to left from vertical
 DEPTH: 108.7 ft.

LOCATION: Right bay of spillway,
 STA 10+66.4 2 ft.
 downstream of up-
 stream grout curtain
 ELEVATION: 5299
 Started: 9/11/76
 Finished: 9/15/76

LOG:

0-5.1 ft. Concrete spillway apron (good bond with rock)
 5.1-bottom WELDED ASH-FLOW TUFF
 5.1-22.2 light gray, slightly porphyritic with phenocrysts up to 1/16" in diameter; scattered pumice fragments up to 3" in diameter; low density rock
 22.2-38.0 prominent "bands" of collapsed pumice fragments and aligned vesicles oriented 60° to the core axis; red-gray color
 38.0-108.7 medium gray, porphyritic

Core breaks:

5.1'-22.2' typical core fragment about 1.0' long, range 0.7'-2.0', most joints are smooth, planar and oriented 60° to the core axis
 22.2'-38.0' typical core fragment is 0.5' long, range 0.2'-1.0' most joints are rough-surfaced and parallel to the "bands" which are angled 60° to the core axis
 38.0'-108.7' typical core fragment is 1.0' long, range 0.4' to more than 3.0', breaks are smooth, planar joints roughly 60° to core axis

Prominent Features

10.7' pumice fragment approximately 3" in diameter
 11.2'-11.3' grout to 1/4" filling a smooth planar joint oriented at 60° to the core axis
 15.0'-15.8' rough-surfaced arcuate core break roughly 10° to the core axis
 40.3'-40.5' pumice fragment with vapor phase minerals
 53.1'-53.3' vesicular zone with voids up to 3/4" in diameter
 55.5'-56.6' large void approximately 1/2" in diameter, lined with quartz crystals
 57.0' grout to 1/4" filling smooth joint, calcite lined, 40° to core
 63.8'-64.7' grout to 1/16" present in layers angled 10° to the core axis
 65.8'-69.3' grout to 3/4" filling planar joint oriented roughly parallel to the core axis
 84.9'-85.4' vesicular zone, voids to 3/4"
 88.4'-88.8' grout to 3/4" partially filling voids
 91.0'-91.4' grout and silt at intersection of 2 joints
 97.7'-98.5' layers of aligned vesicles oriented from 10° to roughly parallel to the core axis

APPENDIX F-4

HOLE DESIGNATION: DH-604
 BEARING: N74°W
 ANGLE: 30° to right from vertical
 DEPTH: 109.7 ft.

LOCATION: Center bay of spillway at STA 10+86 2 ft. downstream of upstream grout curtain
 ELEVATION: 5299
 Started 9/18/76
 Finished 9/21/76

LOG:

0-6 ft. Concrete spillway apron (smooth contact with rock) without apparent adhesion, some suggestion of chalky cement
 6-bottom WELDED ASH-FLOW TUFF
 5.9-21.9 light gray, slightly porphyritic with phenocrysts up to 1/16" in diameter, 2 large pumice fragments, light weight
 21.9-29.0 prominent "bands" of collapsed pumice fragments oriented 60° to the core axis, core breaks frequently parallel to "bands", red to purple gray color
 29.0-48.0 few light-colored "banded" zones, breaks not necessarily through "bands", less intense reddish color
 48.0-109.7 light to medium gray, porphyritic, prominently vesicular below 104.5'

Core breaks:

5.9'-21.9' typical core fragment is about 0.8' long, range 0.2'-1.8', most core breaks are smooth, planar joints oriented 60° to the core axis
 21.9'-29.0' typical core fragment is 0.4' long, range 0.1'-0.7', most breaks are through, or parallel to light-colored "bands" at 60° to core axis
 29.0'-48.0' typical core fragment is 0.4' long, range 0.1'-1.8', most breaks are smooth-surfaced planar joints oriented 60° to the core axis
 48.0'-109.7' typical core fragment is approximately 2.0' long, range 0.4'-4.0', most breaks are smooth-surfaced planar joints oriented about 60° to the core axis

Prominent Features

14.2' pumice fragment to 1-1/4"
 15.4'-16.4' calcite coated planar joint at 10° to core axis
 16.7'-17.1' pumice zone
 42.5'-44.0' rough-surfaced, arcuate fracture with silt coatings, oriented approximately 10° to the core axis
 61.7'-62.2' calcite filled (to 1/16" thick) planar joint at 20° to the core axis
 62.3'-62.5' vesicular zone
 81.5'-83.0' grout to 1" filling a rough-surfaced fracture or void
 95.5' grout to 1/4" filling a planar joint or fracture perpendicular to the core axis
 95.8'-96.4' grout to 1/8" filling a smooth planar joint oriented at 10° to the core axis
 108.1'-109.0' grout to 3/8" filling a planar joint oriented at 15° to the core axis, the joint surface has irregularities or offsets parallel to flattened vesicle layers which are roughly perpendicular to the core axis

APPENDIX F-5

HOLE DESIGNATION: DH-605
 BEARING: ---
 ANGLE: Vertical
 DEPTH: 96 ft.

LOCATION: Center bay of spillway, STA 10+89 2 ft. downstream of upstream grout curtain
 ELEVATION: 5229
 Started: 9/16/76
 Finished: 9/18/76

LOG:

0-5.8 ft. Concrete spillway apron (core break at contact, but bond appears to have been satisfactory)
 5.8-bottom WELDED ASH-FLOW TUFF
 5.8-21.5 light gray, slightly porphyritic (phenocrysts to 1/16" in diameter), rare pumice fragments mostly less than 3/4" in diameter; very light weight rock
 21.5-25.3 light reddish gray, bands of aligned vesicles and collapsed pumice fragments, many core breaks through "bands", breaks "bands" oriented roughly 80° to core axis
 25.3-40.5 medium gray with few "bands" of collapsed pumice oriented roughly 80° to core axis, some light-colored "bands" of probable vapor phase minerals are aligned roughly parallel to the core axis
 40.5-96.0 light gray, slightly porphyritic, amygdules and phenocrysts to 1/8" in diameter

Core breaks:

5.8'-21.5' typical core fragment is about 1.0' long, range 0.4'-2.2', most breaks are wavy or planar smooth-surfaced joints
 21.5'-25.3' typical core fragment is 0.4' long, range 0.1'-0.5'; most breaks are rough-surfaced and parallel the "banding" at 80° to the core axis
 25.3'-40.5' typical core fragment is 0.6' long, range 0.1'-2.0'; most breaks are smooth-surfaced
 40.5'-96.0' typical core fragment is about 1.7' long, range 0.4'-more than 3.0'; most breaks are smooth-surfaced planar joints oriented 70°-80° to the core axis

Prominent Features

7.8'-8.4' arcuate joint oriented approximately 10° to the core axis
 14.6'-14.9' grout to 1/4" thick filling a planar, silt lined joint oriented 15° to the core axis
 15.2'-15.5' grout and chalky grout to 1/8" filling an arcuate joint roughly parallel to the core axis
 16.4'-16.5' flattened pumice fragment, an apparent orientation of 60° to core axis
 17.2' chalky grout to 1/4" in smooth-surfaced joint, angled 40° to the core axis
 26.3'-27.1' wavy, arcuate joint roughly parallel to the core axis
 29.7'-30.1' grout and calcite in rough-surfaced, irregular fracture angled about 20° to core axis
 41.1'-41.5' aligned vesicle layers oriented about 70°-80° to core axis
 42.4'-44.0' arcuate joint oriented approximately 10° to core axis, partially filled with calcite to 1/16"
 92.1'-92.4' grout to 1/8" thick in smooth planar joint at 25° to core axis

APPENDIX F-6

HOLE DESIGNATION: DH-606
 BEARING: S74°E
 ANGLE: 30° to left from vertical
 DEPTH: 40.6 ft.

LOCATION: Center bay of spillway STA 10+92 2 ft. downstream of upstream grout curtain
 ELEVATION: 5299
 Started: 9/21/76
 Finished: 9/22/76

LOG:

0-7.1 ft. Concrete spillway apron (poor bond with rock)
 7.1-bottom WELDED ASH-FLOW TUFF
 7.1-24.0 light gray, slightly porphyritic with phenocrysts to 1/16", scattered pumice fragments to 1-1/2" in diameter
 24.0-40.6 prominent "banding" caused by collapsed pumice fragments and layers of aligned vesicles, few bands below 37.7'

Core breaks:

7.1'-24.0' typical core fragment is 1.2' long, range 0.2'-4.0', most breaks are rough-surfaced planar joints which cross the core at an approximate angle of 70°
 24.0'-40.6' typical core fragment is 0.3' long, range 0.1'-0.8', most core breaks are smooth-surfaced planar joints oriented 60° to the core axis

Prominent Features

11.1'-11.9' smooth-surfaced, partially calcite filled (honeycomb structure) arcuate joint oriented roughly 10° to core axis, chalky grout coating on some calcite
 18.1' grout to 1" filling a planar joint oriented 45° to the core axis
 18.3'-18.8' silt coating on a rough-surfaced joint oriented 10° to the core axis
 31.1'-31.4' 4 layers of silt and calcite oriented approximately 55° to the core axis
 38.9'-39.0' scattered grout filling voids in a vesicular zone aligned roughly 80° to the core axis

APPENDIX F-6A

HOLE DESIGNATION: DH-606A
BEARING: S74E
ANGLE: 30° to left from vertical
DEPTH: 111 ft.

LOCATION: Center bay of spillway STA 10+92 2.7 ft. downstream of upstream grout curtain
ELEVATION: 5299
Started: 9/29/76
Finished: 9/29/76

LOG:

0-7.1 ft. Concrete spillway apron (poor bond with rock silt coatings on rock surface)

7.1-bottom WELDED ASH-FLOW TUFF

7.1-24.1 light gray, slightly porphyritic with phenocrysts to 1/16", scattered pumice fragments to 3", low density rock

24.1-43.0 prominent "banding" caused by flattened vesicles and pumice fragments oriented 60° to the core axis, medium gray with a pink or red tone

43.0-111.0 dark gray, slightly porphyritic, several vesicular zones

Core breaks:

7.1'-24.1' typical core fragment is approximately 1.5' long, range 0.3' to about 3.0', most joints are smooth-surfaced, planar, darkstained, and oriented 60° to the core axis

24.1'-43.0' typical core fragment is approximately 0.4' long, range 0.1'-1.0', most breaks parallel the "banding" at 60° to the core axis

43.0'-111.0' typical core fragment is 0.9' long, range 0.1'-3.0', most breaks are smooth planar joints oriented 50°-60° to the core axis

Prominent Features

17.1'-17.4' pumice fragment

17.4'-17.7' grout to 3/4" thick in an irregular joint angled 35° to the core axis

19.8'-20.5' rough-surfaced, irregular fracture oriented roughly parallel to the core axis

30.1'-30.4' vesicular zone, layers of aligned vesicles 60° to the core axis

50.1'-50.5' grout to 3/8" in a smooth, planar joint oriented roughly 30° to the core axis, joint surfaces are calcite lined

55.8'-58.0' grout partially filling 1/4" wide joint with calcite lining roughly parallel to the core axis

62.2'-62.7' vesicular zone

APPENDIX F-7

HOLE DESIGNATION: DH-607
BEARING: N74W
ANGLE: 30° to right from vertical
DEPTH: 8.4 ft.

LOCATION: Left bay of spillway
at STA. 11+11 2 ft.
downstream of up-
stream grout curtain
ELEVATION: 5299

LOG:

0-5.7 ft. Concrete spillway apron (good bond with rock)
5.7-bottom WELDED ASH-FLOW TUFF
5.7-8.4 light gray, slightly porphyritic with fine-grained matrix

Core breaks: typical core fragment is 0.2' long, from badly broken zones
to fragments 0.5' long

Prominent Features

- 4.2' layer of aligned vesicles oriented roughly 20° to the core axis
- 7.8' intersection of two rough-surfaced planar joints angled 20° and 60° to the core axis, both with silt coatings

APPENDIX F-7A

HOLE DESIGNATION: DH-607A
BEARING: N74W
ANGLE: 30° to right from vertical
DEPTH: 109.5 ft.

LOCATION: Left bay of spillway,
STA 11+11.6 2.7 ft.
downstream of up-
stream grout curtain
ELEVATION: 5299
Started: 9/29/76
Finished: 10/2/76

LOG:

0-5.7 ft. Concrete spillway apron (good contact with rock)
5.7-bottom WELDED ASH-FLOW TUFF
5.7-24.0 light gray, slightly porphyritic with fine-grained matrix,
phenocrysts to 1/16" in diameter; scattered pumice fragments to 1"
diameter, some fragments slightly flattened perpendicular to the
core axis
24.0-28.8 closely spaced light-colored bands angled 80° to the core axis;
medium to dark gray with purple-pink tones
28.8-109.5 light to medium gray, porphyritic, amydules to 1/8" in diameter

Core breaks:

5.7'-24.0' typical core fragment 0.9' long, range 0.2'-1.8', most breaks
are smooth, planar, and angled 60° to the core axis
24.0'-28.8' typical core fragment 0.3' long, range from less than 0.1'-1.2',
most breaks are parallel to the "banding"
28.8'-109.5' typical core fragment is 1.5', range 0.2'-5.7', most breaks
are smooth, planar joints

Prominent Features

5.9'-6.7' rough-surfaced, irregular fracture roughly 10° to
core axis
21.9'-22.2' rough, irregular fracture oriented 30° to core axis,
silt coatings
24.5' grout and calcite filling a planar joint oriented
approximately 30° to the core axis
46.8'-47.3' layers of aligned vesicles with some chalky grout
fillings
50.5' grout and chalky grout to 3/8" thick filling a smooth
planar joint oriented roughly 35° to the core axis
60.0'-60.6' grout to 3/8" filling a smooth-surfaced arcuate
joint angled roughly 10° to the core axis
70.0'-70.2' scattered grout in vesicle layers oriented approximately
70° to the core axis

APPENDIX F-8

HOLE DESIGNATION: DH-608
BEARING: ---
ANGLE: Vertical
DEPTH: 125.8 ft.

LOCATION: Left bay of spillway
at STA 11+14.6 2 ft.
downstream of upstream
grout curtain
ELEVATION: 5299
Started: 9/25/76
Finished: 9/28/76

LOG:

0-6.1 ft. Concrete spillway apron (poor bond with rock)
6.1-bottom WELDED ASH-FLOW TUFF
5.8-21.2 light gray, slightly porphyritic with phenocrysts to 1/16" in diameter, rare pumice fragments to 3" in diameter
21.2-33.0 prominent light colored "banding" oriented roughly perpendicular to the core axis, pinkish-gray hue
33.0-104.0 medium gray, slightly porphyritic, amygdules of quartz and feldspar
104.0-125.8 fairly prominent "banding" caused by layers of aligned vesicles and collapsed pumice oriented roughly 80° to the core axis

Core breaks:

5.8'-21.2' typical core fragment is 0.8' long, range 0.3'-1.6', most breaks are along planar joints having smooth to slightly rough surfaces
21.2'-33.0' typical core fragment is 0.3' long, range 0.1'-0.5', most breaks are along smooth, planar joints oriented roughly perpendicular to the core axis
33.0'-125.8' typical core fragment is about 1.0' long, range 0.1'-2.5', most breaks are along rough-surfaced planar joints

Prominent Features

12.1'-14.0' rough-surfaced, irregular fracture roughly parallel to the core axis, pumice fragments 1"-3" in diameter at 12.6', 12.9', and 13.6', fracture surface has silt coating
63.0'-64.1' grout coated, rough-surfaced, arcuate joint with iron staining oriented roughly 10° to the core axis
76.3'-78.0' grout to 1/2" thick filling a zone of intersecting joints
78.6'-78.7' grout 1" thick filling a calcite lined joint oriented perpendicular to the core axis
83.8'-85.5' rough-surfaced, irregular fracture roughly parallel to the core axis, grout filled to 3/8"
105.3'-105.8' grout to 1-1/4" filling a planar joint oriented at 10° to the core axis
111.3'-113.1' rough-surfaced, irregular fracture oriented roughly parallel to the core axis, grout filled below 112.6'
123.4'-123.8' planar joint oriented 30° to the core axis, grout and calcite to 1" thick

APPENDIX F-9

HOLE DESIGNATION: DH-609
 BEARING: S74°E
 ANGLE: 30° to left from vertical
 DEPTH: 145 ft.

LOCATION: Left bay of spillway
 STA 11+17.6 2 ft.
 downstream of upstream
 grout curtain
 ELEVATION: 5299
 Started: 10/2/76
 Finished: 10/6/76

LOG:

0-6.2 ft. Concrete spillway apron (good bond with rock)
 6.2-bottom WELDED ASH-FLOW TUFF
 6.2-25.7 light gray, slightly porphyritic, fine-grained matrix with phenocrysts up to 1/16" in diameter
 25.7-40.0 dark gray color with reddish/purple hue with prominent light-colored "bands" angled 40° to the core axis
 40.0-145.0 medium gray, slightly porphyritic, occasional banding and layers of aligned vesicles angled approximately 70° to core axis

Core breaks:

6.2'-25.7' typical core fragment is 1.7' long, range 0.3'-3.0', most breaks are smooth planar joints angled 30° to the core axis
 25.7'-40.0' typical core fragment 0.3' long, range 0.1'-0.8', breaks are rough-surfaced and angle 40°-50° to the core axis
 40.0'-145.0' typical core fragment is 0.8' long, range 0.1'-2.5', 2 sets of joints, one angled 30° to core axis, the other roughly 70° to the core

Prominent Features

14.2'-14.4' rough-surfaced planar joint filled with silt up to 1/8" thick, oriented 35° to the core axis
 16.0'-16.2' grout to 1/16" partially filling a silt lined smooth joint angled 45° to the core axis
 41.7'-42.2' layers of aligned vesicles with flattened voids up to 1/2"
 64.7'-65.3' arcuate joint with silt coating oriented roughly 10° to core axis
 65.3' joint open to 3/8" partially filled with calcite, angled 40° to core axis
 65.5'-66.4' broken core from vesicular zone, limonite stain on vapor phase minerals, grout filling voids between 66.2' and 66.4' angled 40° to the core axis
 87.3'-87.5' aligned layers of vesicles angled 50° to the core axis
 102.3'-104.3' rough-surfaced irregular fracture roughly parallel to the core axis, silt filled

APPENDIX F-10

HOLE DESIGNATION: DH-610
BEARING: Parallels grout cap
ANGLE: 42° to right from vertical
DEPTH: 24.8 ft.

LOCATION: ON GROUT CAP AT
STA 12+73
ELEVATION: 5222.1

LOG:

0-4.8 ft. Concrete grout cap (good bond with rock)

4.8-24.8 WELDED ASH-FLOW TUFF

uniform medium gray with porphyritic texture, fine-grained matrix with phenocrysts to 1mm in diameter

typical joint spacing 0.6', ranging from 0.1' to 1.3'; most are smooth-surfaced with a dark stain and are oriented approximately 40° to core axis; a few joints are oriented at 20° to the core axis

Prominent Features

- 5.6'-6.5' smooth arcuate joint, approximately 10° to core axis
- 13.5'-14.8' smooth arcuate joint roughly parallel with core axis

APPENDIX F-11

HOLE DESIGNATION: DH-611
BEARING: Parallels grout cap
ANGLE: $20\frac{1}{2}^{\circ}$ to right from vertical
DEPTH: 23.9 ft.

LOCATION: ON GROUT CAP AT
STA 12+74
ELEVATION: 5222

LOG:

0-3.9 ft. Concrete grout cap (good bond with rock)

3.9-23.9 WELDED ASH-FLOW TUFF

a uniform medium gray with porphyritic texture, fine-grained matrix; lines of vesicles at 30° to core axis are common below 22.0'

typical joint spacing 0.5', ranging from 0.3' to 1.0'; most are smooth-surfaced with a dark stain and make an angle of approximately 50° with the core axis, some at 20° to the core axis

Prominent Features

- 6.3' smooth planar joint with calcite coating, angled 40° to core axis
- 6.8' smooth planar joint with calcite filling (to 1/8"), angled 40° to core axis
- 8.7'-9.3' rough-surfaced core break at approximately 10° to the core axis, silt coatings on the fracture surface

APPENDIX F-12

HOLE DESIGNATION: DH-612
BEARING: Parallels grout cap
ANGLE: Vertical
DEPTH: 23.5 ft.

LOCATION: ON GROUT CAP AT
STA 12+75
ELEVATION: 5222

LOG:

0-3.5 ft. Concrete grout cap (good bond with rock)

3.5-23.5 WELDED ASH-FLOW TUFF

a uniform medium gray with porphyritic texture, fine-grained matrix; lines of vesicles at 10° to core axis are common between 15.0'-17.3'

typical joint spacing 0.4', ranging from 0.05'-1.1'; most are smooth-surfaced with a dark stain and make an angle of approximately 70° with the core axis, some at 40° to core axis

Prominent Features

22.9' minor grout coatings on smooth planar joint angled at 45° to core axis

APPENDIX F-13

HOLE DESIGNATION: DH-613
BEARING: Parallels grout cap
ANGLE: 45° to right from vertical
DEPTH: 24.8 ft.

LOCATION: ON GROUT CAP AT
STA 13+15
ELEVATION: 5206

LOG:

0-4.8 ft. Concrete grout cap (good bond but rock is badly broken 0.15 ft. below contact)

4.8-24.8 WELDED ASH-FLOW TUFF

medium gray, porphyritic texture, coarse-grained; amygdales common (to $3/8''$)

core is marked by smooth-surfaced arcuate joints nearly parallel to the core axis in the upper part, making fracture density meaningless; there are no natural breaks below 19.5'

Prominent Features

6.3'-7.9' aligned vesicles with vapor phase minerals

13.5'-14.0' smooth arcuate joint partially filled with calcite, joint originally open to $1/4''$; angled 10° to core axis; truncated by joint at 14.0'

14.0' smooth planar joint at 50° to core axis, calcite coated

18.1'-20.0' aligned vesicle layer and arcuate joint roughly parallel to core axis, partial calcite filling of vesicle layer

APPENDIX F-14

HOLE DESIGNATION: DH-614
BEARING: Parallels grout cap
ANGLE: $22\frac{1}{2}^{\circ}$ to right from vertical
DEPTH: 24.4 ft.

LOCATION: ON GROUT CAP AT
STA 13+15
ELEVATION: 5206

LOG:

0-4.4 ft. Concrete grout cap (good bond with rock)

4.4-24.4 WELDED ASH-FLOW TUFF

light to medium gray, porphyritic; amygdales filled with vapor phase minerals (to $\frac{3}{8}$ ") up to 10% of rock

typical joint spacing greater than 1.0', ranging from 0.4'-2.0' except for intensely jointed zones 5.6'-6.3' and 11.8'-13.0'

Prominent Features

- 4.6'-5.6' smooth planar joint and aligned vesicle layer, both roughly parallel to core axis
- 12.0'-12.4' chalky grout in planar joint at 25° to core axis
- 18.5' void space in aligned vesicle layer
- 21.8' core break through limonite stained "band" at 25° to core axis
- 23.6'-24.4' aligned vesicles with vapor phase minerals at 10° to core axis

APPENDIX F-15

HOLE DESIGNATION: DH-615
BEARING: Parallels grout cap
ANGLE: Vertical
DEPTH: 24.9 ft.

LOCATION: ON GROUT CAP AT
STA 13+15
ELEVATION: 5206

LOG:

0-4.9 ft. Concrete grout cap (good bond with rock)

4.9-24.9 WELDED ASH-FLOW TUFF

medium gray, slightly porphyritic, coarse-grained; vesicles and aligned vesicles are common

typical joint spacing above 13.0' is 0.6', ranging from 0.1'-2.4', below 13.0', typical spacing is greater than 2.0', ranging from 1.5'-more than 4.0'

Prominent Features

- 5.0'-6.0' rough fracture at approximately 10° to core axis
- 9.2'-11.0' aligned vesicles and bands of light colored minerals at 10° to core axis
- 11.1' grout coating on smooth planar joint roughly 30° to core axis

APPENDIX E-15 (Con't.)

Prominent Features (Con't.)

- 12.2'-12.6' rough fracture with probable vapor phase minerals roughly 15° to core axis
- 15.2'-15.7' rough fracture with dark staining oriented 15° - 20° to core axis
- 22.2'-23.5' rough fracture with dark stain, 30° to core axis

APPENDIX E-16

HOLE DESIGNATION: DH-616
BEARING: Parallels grout cap
ANGLE: $47\frac{1}{2}^{\circ}$ to right from vertical
DEPTH: 24.3 ft.

LOCATION: ON GROUT CAP AT
STA 13+29
ELEVATION: 5197

LOG:

0-4.3 ft. Concrete grout cap (good bond, but rock badly broken)

4.3-24.3 WELDED ASH-FLOW TUFF

medium gray with porphyritic texture, some "banding" in lower portion

very few joints, a typical spacing would be greater than 2.0', some unbroken core longer than 5.0'

Prominent Features

- 4.7'-6.1' smooth arcuate joint with calcite coating nearly parallel to core axis, paralleled by rough fracture 4.7'-5.5'
- 5.9' calcite coating on smooth planar joint, 30° to core axis
- 6.3' calcite coating on smooth planar joint, 30° to core axis
- 13.4'-13.8' aligned vesicles with significant void space, dark stain
- 15.1' core break nearly perpendicular to core axis through layer of limonite stained probable vapor phase minerals
- 17.5' grout filled, smooth, planar joint roughly 20° to core axis
- 20.2' grout filled, smooth, planar joint roughly 20° to core axis

APPENDIX F-17

HOLE DESIGNATION: DH-617
BEARING: Parallels grout cap
ANGLE: 21° to right from vertical
DEPTH: 23.9 ft.

LOCATION: 0.9 ft. upstream
grout cap center-
line at STA 13+30
ELEVATION: 5197

LOG:

0-3.9 ft. Concrete grout cap (good bond with rock)
3.9-23.9 WELDED ASH-FLOW TUFF

medium gray with porphyritic texture; some vesicle alignment
below 11.5' at an angle of 60° to core axis

typical joint spacing 1.5', ranging from 0.2'-more than 6.0';
most are smooth-surfaced, dark stained, and make an angle of
approximately 40° with the core axis

Prominent Features

4.0'-4.7' smooth arcuate joint and rough fracture, both
nearly parallel to core axis
4.7'-6.7' smooth arcuate joint roughly parallel to core axis
8.0'-10.0' smooth arcuate joint roughly parallel to core axis,
partially filled with grout

APPENDIX F-18

HOLE DESIGNATION: DH-618
BEARING: Parallels grout cap
ANGLE: Vertical
DEPTH: 24.2 ft.

LOCATION: ON GROUT CAP AT
STA 13+30
ELEVATION: 5197

LOG:

0-4.2 ft. Concrete grout cap (good bond with rock)
4.2-24.2 WELDED ASH-FLOW TUFF

medium to dark gray, porphyritic; aligned vesicles below 14.0',
light colored "bands" of collapsed pumice and vapor phase
minerals present, but not obvious (oriented roughly perpendicular
to core axis)

typical core fragment is 1.3' long, range 0.1'-1.9', most breaks
are through "bands", smooth-surfaced joints are not common

Prominent Features

4.9'-5.9' smooth, dark stained joint at 15° to core axis
15.8'-16.8' rough-surfaced fracture with dark surface stain at
10° to core axis

APPENDIX F-18 (Con't.)

Prominent Features (Con't.)

16.9'-17.4' rough-surfaced fracture with dark surface stain
at 10°-20° to core axis
21.6'-22.3' smooth, planar joint at 10° to core axis
22.8' smooth planar joint roughly perpendicular to core
axis, partially filled with calcite

APPENDIX F-19

HOLE DESIGNATION: DH-619
BEARING: Parallels grout cap
ANGLE: 31° to left from vertical
DEPTH: 26.7 ft.

LOCATION: ON GROUT CAP AT
STA 13+31
ELEVATION: 5197

LOG:

0-6.7 ft. Concrete grout cap (concrete appears to lie on a smooth joint
surface oriented 60° to the core axis, no apparent adhesion)

6.7-26.7 WELDED ASH-FLOW TUFF
medium to dark gray porphyritic rock with crystalline matrix;
aligned vesicles below 15.0', average spacing of 0.5'

typical joint spacing 1.2', ranging from 0.3'-2.6'; most are
smooth-surfaced, planar with a dark stain

Prominent Features

10.1'-10.6' smooth arcuate joint, roughly 25° with core axis
11.6'-12.2' smooth arcuate joint with chalky grout coating,
roughly 25° to core axis
18.9'-19.3' smooth planar joint making 40° angle with core
axis is calcite lined and filled with 3/4" of
calcareous silt
21.9' rough-surfaced fracture with calcite lining filled
with calcareous silt (to 3/8")
25.1' joint up to 1/2" wide filled with grout and silt

APPENDIX F-20

HOLE DESIGNATION: DH-620
BEARING: Parallels grout cap
ANGLE: 44° to right from vertical
DEPTH: 100 ft.

LOCATION: 1.4 ft. downstream
of grout cap center-
line at STA 13+46
ELEVATION: 5189

LOG:

0-4.4 ft. Concrete grout cap (good bond, but rock is badly broken along rough, dark-stained fractures

4.4 to 100 WELDED ASH-FLOW TUFF

dark gray, porphyritic, general increase in the number of "bands" of collapsed pumice and vapor phase minerals with depth, "bands" are prominent below 50.0'

typical core fragment is 0.9' long, breaks occur primarily at dark stained, planar joints above 45.0', along or through "bands" below 45.0'

Prominent Features

- 7.2' rough-surfaced fracture at 30° to core axis, silt filled to $3/8$ "
- 7.7' smooth, planar joint at 30° to core axis, silt filled to $3/8$ "
- 7.6'-8.0' smooth planar joint at 30° to core axis
- 20.7' rough-surfaced fracture at 50° to core axis, calcite filled (to $3/4$ "
- 24.8' smooth planar joint at 45° to core axis
- 25.0'-25.3' planar joint at 30° to core axis, partially filled with grout
- 25.5'-26.1' smooth arcuate joint with dark surface stain oriented 15° to core axis
- 26.1' smooth planar joint at 45° to core axis, partially filled with calcite
- 41.1'-41.6' smooth planar joint at 30° to core axis, grout lined and filled with silt (to $3/8$ "
- 41.8'-42.0' planar joint at 30° to core axis, grout lined and filled with silt (to $3/8$ "
- 49.0'-50.0' joint oriented at 10° to core axis has a calcite lining and is filled with grout (to $1/2$ " and silt that appears to have been washed in with the grout
- 78.9' grout and silt in an area of broken core, appears to be a joint oriented 15° - 20° to the core axis with a grout lining and silt filling
- 91.1'-91.4' calcite lined planar joint with silt filling (to $1/4$ "

APPENDIX F-21 .

HOLE DESIGNATION: DH-621
BEARING: Parallels grout cap
ANGLE: $22\frac{1}{2}^{\circ}$ to right from vertical
DEPTH: 23.5 ft.

LOCATION: 1.4 ft. downstream
of grout cap center-
line at STA 13+47
ELEVATION: 5189

LOG:

0-3.5 ft. Concrete grout cap
3.5-23.5 WELDED ASH-FLOW TUFF

dark gray with reddish iron staining, porphyritic rock with crystalline matrix; rare vesicles with vapor phase minerals up to 1/2" in diameter, some "banding"

typical joint spacing greater than 2.0', typical core break along bands of collapsed pumice and vapor phase minerals 0.7', ranging from 0.1'-1.3'

Prominent Features

- 4.0'-4.8' badly broken core; appears to be a smooth arcuate joint at 15° to core axis and a rough-surfaced fracture nearly parallel to the core axis (trace of fracture extends to 5.4'), fracture and joint surfaces are coated with chalky grout
- 6.3'-6.5' smooth planar joint at 40° to core axis has grout coating on calcite lining
- 6.7'-8.6' curving rough-surfaced fracture with grout filling (up to 1/4") over calcite coating, oriented roughly 10° to core axis
- 11.5'-12.6' smooth-surfaced planar joint with silt filling over calcite coating
- 12.6' smooth planar joint at 60° to core axis with chalky grout
- 14.6' grout filled (to 1/2") planar joint at 80° to core axis
- 14.7' grout filled (to 1/2") planar joint at 80° to core axis
- 19.2'-19.3' chalky grout (?) coating on calcite lining on planar joint to 60° to core axis
- 20.2'-20.9' broken core; intersection of joints at 20° and 40° to core axis with rough fracture nearly parallel to core axis, silt coatings on joint and fracture surfaces

APPENDIX F-22

HOLE DESIGNATION: DH-622
 BEARING: ---
 ANGLE: Vertical
 DEPTH: 23.5 ft.

LOCATION: 1.4 ft. downstream
 of grout cap center-
 line at STA 13+48
 ELEVATION: 5189

LOG:

0-3.5 ft. Concrete grout cap (good bond with rock)

3.5-23.5 WELDED ASH-FLOW TUFF

medium to dark gray, slightly porphyritic, slightly vesicular rhyolite; aligned vesicle layers and "bands" of collapsed pumice and vapor phase minerals are common, especially below 14.0'

typical core break 0.4', ranging 0.1'-1.3'; breaks primarily through "bands" in lower half of hole, along smooth, planar joints in upper half; joints oriented at 50°-70° with core axis

Prominent Features

3.5'-7.5' badly broken core; appears to have intersected previous drill hole at 4.5' and 7.3'(?); vesicle layers aligned parallel to core axis
 13.4' smooth planar joint roughly perpendicular to core axis, chalky grout coating

APPENDIX F-23

HOLE DESIGNATION: DH-623
 BEARING: Parallels grout cap
 ANGLE: 22½° to left from vertical
 DEPTH: 21.4 ft.

LOCATION: 1.4 ft. downstream
 from grout cap
 centerline at STA
 13+49
 ELEVATION: 5189

LOG:

0-1.4 ft. Concrete grout cap (good bond, but rock breaks along a smooth, planar joint at 20° to core axis)

1.4-21.4 WELDED ASH-FLOW TUFF

dark gray, slightly porphyritic; vesicular with aligned vesicle layers

core fragments typically 0.6' long, ranging 0.1'-1.6'; breaks equally divided between smooth-surfaced joints and breaks through "bands" of collapsed pumice and vapor phase minerals

APPENDIX F-23 (Con't.)

Prominent Features

- 1.2'-1.6' smooth, planar joint at 20° angle to core axis, weakens effectiveness of concrete/rock bond
- 3.2'-4.4' broken core, rough-surfaced fractures with limonite staining coupled with layers of vesicles aligned roughly parallel to the core axis and at 60° to the core axis
- 13.0'-13.4' grout coating on smooth, arcuate joint oriented roughly 20° to the core axis
- 20.5'-21.1' dark stained rough-surfaced fracture and aligned vesicle layers oriented approximately 10° to core axis

APPENDIX F-24

HOLE DESIGNATION: DH-624
BEARING: Parallels grout cap
ANGLE: 45° to right from vertical
DEPTH: 24.3 ft.

LOCATION: ON GROUT CAP AT
STA 13+76
ELEVATION: 5170

LOG:

0-4.3 ft. Concrete grout cap (core break at contact, suggestion of chalky cement)

4.3-24.3 WELDED ASH-FLOW TUFF

dark gray, porphyritic rhyolite with frequent (roughly every 0.2') "bands" of collapsed pumice and vapor phase minerals, bands are oriented approximately 40° - 50° to core axis

typical core break spacing along "bands" is 0.8', ranging from 0.2'-2.1', rare smooth surfaced joints oriented 40° - 50° to core axis; most smooth-surfaced joints are silt coated, breaks along "bands" do not have silt

Prominent Features

- 22.8'-23.5' rough-surfaced fracture along probable vesicle layer at 20° to core axis has silt filling over chalky grout coating on calcite lining

APPENDIX F-25.

HOLE DESIGNATION: DH-625
BEARING: ---
ANGLE: Vertical
DEPTH: 26.3 ft.

LOCATION: ON GROUT CAP AT
STA 13+76
ELEVATION: 5170

LOG:

0-5.3 ft. Concrete grout cap (poor bond with rock, chalky cement at contact)
5.3-26.3 WELDDED ASH-FLOW TUFF

medium to dark gray, coarse-grained porphyritic rhyolite, yellow stained "bands" of collapsed pumice and vapor phase minerals oriented roughly perpendicular to core axis

typical core break spacing is 0.5', ranging from 0.2'-1.2', along "bands"; only 5 smooth-surfaced core breaks in core

Prominent Features

- 8.7' smooth, planar joint at right angle to core axis, coating on joint surfaces suggest vapor phase minerals
- 12.7' smooth, planar joint at 80° to core axis
- 17.0' smooth, planar joint at right angle to core axis, coating on joint surfaces suggest vapor phase minerals
- 19.0' smooth, planar joint at right angle to core axis, coating on joint surfaces suggest vapor phase minerals
- 23.3'-23.5' relatively smooth, planar joint with dark stain oriented roughly 30° to core axis, calcite filling to 1/4"

APPENDIX F -26

HOLE DESIGNATION: DH-626
BEARING: Parallels grout cap
ANGLE: 22½° to left from vertical
DEPTH: 26.7 ft.

LOCATION: ON GROUT CAP AT
STA 13+77
ELEVATION: 5170

LOG:

0-6.7 ft. Concrete grout cap (concrete appears to lie on a smooth planar joint surface at 70° to the core axis, no apparent adhesion)

6.7-26.7 WELDDED ASH-FLOW TUFF

medium to dark gray, coarse-grained slightly porphyritic rhyolite; has a marked "banded" appearance due to aligned vesicles and yellowish "bands" of collapsed pumice and vapor phase minerals oriented approximately 65° to core axis

typical core fragment about 0.4' long, ranges from 0.2'-1.4', breaks occur most frequently through "bands", but smooth joint surfaces are also common

APPENDIX F-26 (Con't.)

Prominent Features

- 9.1'-9.6' smooth planar joint forms 20° degree angle with core axis, filled with chalky grout to 1/4" and silt coatings
- 20.0'-20.8' badly broken core, rough-surfaced fracture nearly parallel to core axis, dark stain

APPENDIX F-27

HOLE DESIGNATION: DH-627

LOCATION: On grout cap

BEARING: Parallels grout cap alignment

alignment at

ANGLE: 46° to right from vertical

STA 14+08

DEPTH: 21.0 ft.

ELEVATION: 5149

LOG:

0-21.0 ft. WELED ASH-FLOW TUFF

dark gray, slightly porphyritic rhyolite; prominent "banding" with dark red "earthy" stain and coating on aligned vesicle surfaces and layers of collapsed pumice and vapor phase minerals, most bands roughly 50° to core axis

most core breaks occur through "bands", typically 0.8' apart, ranging from 0.2'-1.6'

Prominent Features

- 8.8' planar joint at 30° to core axis has chalky surfaces that look more like hydrothermal alteration than vapor phase mineralization
- 11.7' rough-surfaced fracture with vapor phase minerals oriented 50° to core axis, silt coated surfaces
- 15.0' chalky grout coating on rough-surfaced core break at 50° to core axis
- 16.6' silt coating on rough-surfaced core break 50° to core axis
- 19.1'-19.8' smooth, planar joint at 15° to core axis is silt filled to 1/2", joint surfaces calcite lined

Note: Grout cap missing

APPENDIX F-28

HOLE DESIGNATION: DH-628
BEARING: ---
ANGLE: Vertical
DEPTH: 21.0 ft.

LOCATION: On grout cap
alignment at
STA 14+10
ELEVATION: 5149

LOG:

0-21.0 ft. WELDED ASH-FLOW TUFF

dark gray, slightly porphyritic rhyolite; prominent "banding" caused by aligned vesicles and layers of vapor phase minerals, banding is roughly perpendicular to the core axis

most core breaks occur through "bands", typically 0.4' apart, ranging from 0.1'-0.9'

Prominent Features

- 0.9' silt in core break through vapor phase "band", probably washed in during clean-up
- 1.1' silt in core break through vapor phase "band", probably washed in during clean-up
- 7.3' smooth, planar joint through vapor phase "band", chalky grout to 1/8"
- 15.2'-16.2' rough-surfaced fracture nearly parallel with core axis
- 20.5' rough-surfaced planar joint at 50° to core axis

Note: Grout cap is missing

APPENDIX F -29

HOLE DESIGNATION: DH-629
BEARING: Parallels grout cap alignment
ANGLE: 34° to left from vertical
DEPTH: 21.0 ft.

LOCATION: 1.1 ft. downstream
of grout cap align-
ment at STA 14+08
ELEVATION: 5149

LOG:

0-21.0 ft. WELDED ASH-FLOW TUFF

dark gray, porphyritic rhyolite; prominent "banding" due to aligned vesicles and layers of vapor phase minerals, "banding" roughly 60° to core axis

core breaks primarily through "bands", typical spacing 0.4', ranging from 0.1'-1.1'

APPENDIX F-29 (Con't.)

Prominent Features:

- 2.2'-2.5' planar joint with grout filling to 1/16", angled 30° to core axis
- 9.2' smooth planar joint with grout coating, angled 70° to core axis
- 9.3' smooth planar joint, angled 70° to core axis
- 15.7'-16.0' smooth planar joint with dark stain and silt coating, angled 22° to core axis
- 17.5'-17.7' rough-surfaced, planar joint with dark stain and silt filling to 3/16", angled 35° to core axis

Note: Grout cap is missing

APPENDIX F-30

HOLE DESIGNATION: DH-630

LOCATION: ON GROUT CAP AT

BEARING: Parallels grout cap

STA 14+27

ANGLE: 47° to right from vertical

ELEVATION: 5141

DEPTH: 21.7 ft.

LOG:

0-1.7 ft. Concrete grout cap (good bond with rock)

1.7-21.7 WELDED ASH-FLOW TUFF

dark gray, coarse-grained, slightly porphyritic rhyolite; vesicular (to 8% of rock mass), vesicles to 1/4" in diameter; "banding" due to aligned vesicles and layers of vapor phase minerals, "banding" oriented at 50° to core axis

core fragments typically 1.0' long, range from 0.1'-2.0', most breaks occur through "bands", but several occur at smooth-surfaced planar joints

Prominent Features

- 9.7'-10.1' smooth planar joint at 30° to core axis
- 10.5'-10.9' smooth planar joint at 30° to core axis
- 21.0'-21.2' smooth planar joint at 35° to core axis
- 21.3'-21.5' smooth planar joint at 25° to core axis intersects a smooth planar joint, 21.5'-21.6', which is oriented 60° to the core axis

APPENDIX F-31

HOLE DESIGNATION: DH-631
BEARING: ---
ANGLE: Vertical
DEPTH: 21.6 ft.

LOCATION: ON GROUT CAP AT
STA 14+26
ELEVATION: 5142

LOG:

0-1.6 ft. Concrete grout cap (good bond with rock)

1.6-21.6 WELDED ASH-FLOW TUFF

dark gray, porphyritic, vesicular, "banding" due to closely spaced layers of aligned vesicles and vapor phase minerals

typical core fragment 0.4' long, range 0.1'-1.2', breaks are almost exclusively through "bands" and roughly perpendicular to core axis

Prominent Features

- 1.6'-2.4' layers of aligned vesicles roughly parallel to core axis, partially filled with chalky grout
- 13.0'-13.3' smooth, planar joint at 20° to core axis, off-set along "band" perpendicular to core axis

APPENDIX F-32

HOLE DESIGNATION: DH-632
BEARING: Parallels grout cap
ANGLE: 34° to left from vertical
DEPTH: 23.1 ft.

LOCATION: ON GROUT CAP AT
STA 14+28
ELEVATION: 5141

LOG:

0-3.1 ft. Concrete grout cap (good bond to rock)

3.1-23.1 WELDED ASH-FLOW TUFF

dark gray, porphyritic, vesicular, "banding" due to closely spaced layers of aligned vesicles and vapor phase minerals

typical core fragment 1.0' long, range 0.15'-1.4' breaks through "bands" which are oriented approximately 40° to the core axis

Prominent Features

- 3.1'-3.6' badly broken core, closely spaced layers of vesicles aligned roughly parallel to the core axis
- 13.6'-13.8' core break along rough, dark stained surface angled approximately 40° to core axis
- 14.3'-14.7' core break along rough, dark stained surface angled 10° to core axis

APPENDIX F-33

HOLE DESIGNATION: DH-650
 BEARING: S21° E
 ANGLE: 31° to left from vertical
 DEPTH: 351.5 ft.

LOCATION: Dam crest, 4.7 ft.
 upstream of center-
 line at STA 3+00
 ELEVATION: 5332

LOG:

0-90 ft. Dam embankment

WELDED ASH-FLOW TUFF from 90 ft. to hole bottom

90.0'-150.0' light gray, porphyritic with fine-grained matrix, scattered pumice-like fragments to 3/4" in diameter, vesicles and amydules common
 150.0 -172.0 light to medium gray, prominent "banding" due to layers of aligned vesicles and collapsed pumice fragments with probable vapor phase mineralization, "bands" 45°-55° to core axis
 172.0-183.0 light gray, prophyritic
 183.0-308.0 light to medium gray, prominent "banding" due to layers of aligned vesicles and collapsed pumice fragments with probable vapor phase mineralization, "bands" roughly 50° to core axis
 308.0-351.5 pronounced thin, wavy "banding" (eutaxitic texture), staining of light colored minerals below 325.0

Core breaks: almost all breaks are rough-surfaced and through or parallel to "bands"

90.0'-150.0' typical fragment about 2.0' long, range 0.1'-6.0'
 150.0'-172.0' typical fragment about 0.5' long, range 0.1'-1.2'
 172.0'-183.0' typical fragment about 0.8' long, range 0.6'-2.3'
 183.0'-308.0' typical fragment about 0.5' long, range 0.1'-2.1'

Prominent Features

107.6'-110.6' layers of aligned vesicles roughly parallel to core axis
 118.5' rough-surfaced, planar fracture filled with grout to 1-1/2", oriented 40° to core axis
 128.7'-132.0' layers of aligned vesicles with significant void space angled 20° to core axis
 132.6' grout to 1/2" filling vesicle layer at 30° to core axis
 150.0' grout to 1/2" filling rough-surfaced fracture at 50° to core axis
 150.6' grout to 1/4" filling rough-surfaced fracture at 50° to core axis
 173.7' smooth, planar joint at 30° to core axis, calcite lined, grout filled to 1/4"
 195.3' grout and calcite in rough fracture at 45° to core axis, grout to 1/16"
 206.3'-206.6' sand grout to 3/4" thick filling smooth planar joint roughly 30° to core axis

APPENDIX F-33A
HOLE SURVEY

HOLE DESIGNATION: DH-650
BEARING: S21°E (Surface measurement)
ANGLE: 31° to left from vertical
(Surface measurement)

LOCATION: Dam crest, 4.7 ft.
upstream of centerline
at STA 3+00
ELEVATION: 5332

<u>Depth</u> <u>ft.</u>	<u>SPERRY SUN SURVEY:</u> <u>Angle to left</u> <u>from vertical</u>	<u>Bearing</u>
0	33	
10	33	
20	33.5	
30	34	
40	35.5	
50	36	
60	35.5	
70	35.5	
80	35	
90	36	
110	36	
120	35	
130	35	S27°E
140	35	S26°E
150	35	S26°E
160	35	S26°E
170	35	S26°E
180	35	S26°E
190	35	
200	34.5	
210	34.5	
220	34	
230	34	
240	34.5	
250	34	
260	34.5	
270	34	
280	34	

APPENDIX F-34

HOLE DESIGNATION: DH-651
 BEARING: ---
 ANGLE: Vertical
 DEPTH: 622.4 ft.

LOCATION: At STA 4+34 on
 centerline of dam
 ELEVATION: 5332

LOG:

0-78.9 ft. Dam embankment
 78.9-80.4 Concrete (good bond with rock)
 80.4-502.2 WELDED ASH-FLOW TUFF
 80.4-121.0 light gray, porphyritic with fine-grained matrix; small amygdules and phenocrysts (to 1/4" diameter) present as less than 10% of rock mass
 121.0-140.0 light to medium gray, porphyritic, vesicular with layers of aligned vesicles and collapsed pumice fragments oriented roughly perpendicular to the core axis, vesicles to 1/2"
 140.0-160.0 light to medium gray, porphyritic, layers of aligned vesicles at approximately 60° to core axis, phenocrysts and amygdules to 1/4"
 160.0-270.0 basic color is medium gray, although overall rock has a light gray appearance because amygdules and layers of collapsed pumice fragments with probable vapor phase minerals, phenocrysts to 3/8", amygdules to 1", flattened amygdules and "pumice" layers oriented 60° to core axis
 270.0-345.0 light "bands" of collapsed pumice and vapor phase minerals have an irregular orientation resulting in a mottled core appearance, bands 60° to core axis, scattered pumice fragments to 2" in diameter
 345.0-444.0 closely spaced, wavy, light and dark colored bands up to 1/2" thick, average 1/4", individual shards and crystals appear pressed together, eutaxitic structure, "bands" are oriented roughly 60° to core axis
 444.0-480.0 chocolate brown color with light colored bands (wider spacing than above), general decrease in grain size
 480.0-502.2 porphyritic with nearly glassy, dark brown matrix, phenocrysts to 1/4" making up to 15% of rock mass
 502.2-bottom of hole SEDIMENTARY SEQUENCE
 502.2-506.5 ash-fall tuff
 506.5-517.7 sand: sedimentary layering at 45° to core axis, grain-size segregation of tuffaceous material, purple to reddish brown color
 517.7-535.9 conglomeratic sandstone: slightly indurated, coarse, poorly sorted sand conglomerate, gravel to 3/4" in diameter
 535.9-547.1 sand: poorly sorted sands with little or no induration, tan to buff colored
 547.1-555.0 sand and gravel: gravel to 2" in diameter, poorly sorted sand matrix
 555.0-561.5 sandstone: tuffaceous, slight induration, air dries hard, gray color

APPENDIX F-34 (Con't.)

- 561.5-582.9 sand and gravel: gravels to 3", some clumps of fine sand, recovery is almost totally "washed" gravels
582.9-596.4 clay: clay with fine sand, light brown color
596.4-622.4 sand and gravel: clayey, fine sand conglomerate with gravels to 1" in diameter, slightly indurated; recovery below 600.0' consists of "washed" gravels

Core breaks:

- 80.4'-121.0' typical core fragment is 2.0' long, range 0.3' to more than 7.0', most breaks are rough-surfaced and approximately perpendicular to the core axis
121.0'-140.0' typical core fragment about 0.5' long, range 0.1'-1.3', most breaks are rough-surfaced and through or parallel to "bands" roughly perpendicular to the core axis
140.0'-160.0' typical core fragment 1.0' long, range 0.1'-2.0', rough breaks perpendicular to the core axis
160.0'-270.0' typical core fragment is 0.8' long, range 0.2'-1.7', breaks are rough-surfaced and through or parallel to the "banding", makes an angle of 60° with core axis
270.0'-300.0' typical core fragment is roughly 1.5' long, range 0.2'-1.7', rough breaks parallel to "banding", 60° to core axis
300.0'-345.0' typical core fragment is 3.0' long, range 0.5'-4.6', rough breaks parallel to "banding"
345.0'-444.0' typical core fragment is 0.8' long, range 0.2'-2.0', rough-surfaced breaks through or parallel to "bands", 60°-70° to core axis
444.0'-480.0' typical core fragment is 0.8' long, range 0.2'-1.8', breaks are oriented at 30° and 60° to core axis; breaks at 30° orientation are planar, rough to smooth
480.0'-502.2' typical core fragment is more than 1.0' long, some fragments are longer than 5.0'; breaks are along rough-surfaced fractures roughly parallel to the core axis or angled 10°-20° to the axis

Prominent Features

- 82.7'-84.2' grout to 1/2" wide filling and coating a rough-surfaced, irregular fracture, probable vesicle layer
88.6'-93.1' good bond between sand grout and calcite lined rough-surfaced fracture, 92.0'-92.5' core consists entirely of grout
96.4'-98.1' grout, chalky grout, and sand grout in good bond to rough, irregular fracture
114.3'-116.0' aligned vesicles roughly parallel to core axis
154.1'-154.7' grouted vesicle layer oriented 10° to core axis, grout to 1/8" thick
161.1' grout to 1/4" filling rough-surfaced fracture that is roughly perpendicular to the core axis

APPENDIX F-34 (Con't.)

Prominent Features (Con't.)

200.0'-200.6'	grout to 1/16" partially filling rough fracture at 10° to core axis
336.0'	chalky grout to 1/16" thick, silt, and fine sand in a rough-surfaced planar joint oriented 45° to the core axis
353.7'-353.8'	a light-colored "band" of collapsed pumice containing a slightly flattened vesicle with a maximum dimension of 1-1/4"
439.0'-439.9'	smooth-surfaced planar joint roughly parallel to core axis, yellow staining and silt coating
486.3'-491.4'	rough-surfaced, irregular fracture roughly parallel to the core axis, yellow stained silt coatings
517.7'	the contact between the sand and the underlying sand and gravel is oriented 30° to the core axis
561.5'	the contact between the sand and the underlying sand and gravel is oriented 50° to the core axis

APPENDIX F-34A.

HOLE DESIGNATION: DH-651B
 BEARING: ---
 ANGLE: Vertical
 DEPTH: 885.3 ft.

LOCATION: At STA 4+19 on
 centerline of dam
 ELEVATION: 5332

0-498.4 no core, used rock bit
 498.4'-513.8' conglomeratic sand: poorly sorted angular to sub-angular sands and rounded gravels; pebbles up to 2" in diameter; principally coarse to very coarse sand above 505.0, below this the matrix is principally medium sand; gravels decrease in size and number with depth; some partially indurated zones
 513.8'-522.1' coarse gravel: rounded gravel and cobbles to 3-3/4"; unusual black colored matrix of sand and silt-sized material, appears to be a black volcanic sand, some of the larger particles appear glassy; partially indurated
 522.1'-527.0' no core: used rock bit
 527.0'-531.4' sandstone: weakly indurated coarse sand with grain-size range from silt to granules
 531.4'-546.2' gravel: gravel up to 2-1/2" in diameter; brown fine sand and silt matrix to 536.2', no matrix recovered below this except for some clayey material lodged in the end of the core barrel
 546.2'-558.0' no core: used rock bit
 558.0'-562.0' fine sand: brown silty, clayey, fine sand; soft to medium consistency
 562.0'-567.4' clayey silt: brown to tan, soft to medium
 567.4'-596.7' no core: used rock bit, driller reports:
 567.4'-572.0' "silty clay"
 572.0'-575.0' "sand and gravel"
 575.0'-595.0' "large gravel and cobbles"
 595.0'-596.7' "clay"
 596.7'-621.9' clay: tan, very stiff clay; slickensided joints oriented 30°, 45°, and 60° to the core axis, some joint surfaces have a partial dark stain, joint surfaces are oriented at various angles around the core axis, even joints with the same dip are not necessarily parallel; color changes rather abruptly from tan to grayish, off-white at 615.0'; average joint spacing roughly 0.8'; 602.0'-603.3' sandy silt layer; clay appears to have expanded as the core diameter is smaller in the silt layer
 621.9'-622.7' gravel: rounded gravels in an off-white stiff clay matrix; matrix is same as above material
 622.7'-625.6' sandy clay: color change from off-white to brown at 624.4'
 625.5'-626.8' silt: brown, clayey silt with increasing amounts of sand and gravel with depth
 626.8'-631.3' gravel: rounded gravel to 2" in diameter; silty clay matrix
 631.3'-632.6' sand: silty fine to medium sand
 632.6'-634.3' gravel: slightly indurated rounded gravels to 2" in diameter with poorly sorted clay, silt, and sand matrix

APPENDIX F-34A (Con't.)

634.3'-638.8' clay: stiff, silty clay; low plasticity; light tan to off-white in color

638.8'-650.0' no core: used rock bit, driller reports "clay"

650.0'-665.0' clay: brown to off-white clay; soft to medium consistency; low to moderate plasticity

665.0'-670.0' no core: used rock bit, driller reports "clay"

670.0'-675.0' sand: recovered 1.5' of fine to medium, brown-colored sand; two gravel layers up to 0.1' thick, mottled and stained

675.0'-700.0' no core: either used rock bit or had no recovery; driller reports:
 675.0'-699.0' "sand and gravel"
 699.0'-700.0' "sand and clay"

700.0'-702.0' clayey gravel: gravels to 3/4" in diameter in a tan, silty clay matrix

702.0'-710.0' no core: used rock bit

710.0'-710.5' sand: fine, almost white sand with some brown silty clay fragments; poor core recovery

710.5'-885.3' no core: either used rock bit or had no core recovery; driller reports:
 710.5'-712.8' "hard silty sand"
 712.8'-719.2' "gravel"
 719.2'-724.1' "hard silty sand"
 724.1'-729.0' "sand and gravel"
 720.0'-737.0' smooth drilling, "sand or clay"
 737.0'-740.6' hard, smooth drilling, "compact sand"
 740.6'-741.7' "gravels"
 741.7'-778.1' "firm material"
 778.1'-780.0' "gravel"
 780.0'-794.0' drilled smooth and slow, "compacted silty sand"
 794.0'-825.6' drilled firm and slow, with few gravel layers
 825.6'-826.0' "gravels", lost drilling water return
 826.0'-833.8' firm material with gravels throughout
 833.8'-834.9' "gravels and cobbles"
 834.9'-871.1' firm material with occasional gravel
 871.1'-882.2' firm material, no gravel
 882.2'-885.3' "gravel with large cobbles", lost drilling mud return at 883.8'

APPENDIX F-35

HOLE DESIGNATION: DH-652
 BEARING: N19W
 ANGLE: 34° to right from vertical
 DEPTH: 450 ft.

LOCATION: At STA 5+11.2
 5.5 ft. upstream
 on centerline of dam

ELEVATION: 5333

LOG:

0-93.5 ft. Dam embankment
WELDED ASH-FLOW TUFF from 93.5 to bottom of hole
 93.5-151.0 light gray, porphyritic with fine-grained matrix, scattered pumice fragments, layers of aligned vesicles below 129.0'
 151.0-301.0 medium gray, porphyritic with very fine-grained to glassy matrix, vesicular, layers of aligned vesicles and collapsed pumice fragments oriented at 40°-50° to the core axis
 301.0-331.0 closely spaced, wavy light and dark colored bands oriented 50°-60° to core axis, roughly 50% each of light and dark, eutaxitic structure, vesicular, amygdules with vapor phase minerals
 331.0-367.6 "banded" appearance (but not as pronounced as above), light "bands" make up 25% or less of the rock mass, "bands" are oriented 50°-60° to core axis, porphyritic with very fine-grained matrix
 367.6-450.0 "banded" appearance as above, but light "bands" are stained red-yellow, "bands" angled 50°-60° to core axis

Core breaks:

93.5'-151.0' typical core fragment more than 2.0' long, range 0.4'-5.5', breaks are rough-surfaced and oriented roughly 45° to core axis
 151.0'-301.0' typical core fragment 0.5' long, range 0.1'-2.5', most breaks are rough-surfaced and pass through or are parallel to "bands", most breaks at 50°-60° to the core axis
 301.0'-331.0' typical core fragments are about 1.0' long, range 0.1'-3.7', breaks are through light-colored "bands" at 50°-60° to the core axis

Prominent Features

101.0'-101.3' smooth, planar joint at 20° to core axis, grout filled to 1/2"
 112.1'-112.4' grout to 3" filling rough-surfaced fracture at 45° to core axis, possible rock fragments in the grout
 118.0' sand grout to 1/2" filling rough-surfaced opening in a vesicular zone, opening is at 50° to core axis
 129.5' intersection with open drill hole (Ax size) angled 30° to the core axis
 132.2'-135.5' concrete with aggregate to 1", good bond with rock, rock surfaces are rough, calcite lined, and angle 10°-20° to the core axis

APPENDIX F-35 (Con't.)

Prominent Features (Con't.)

- 138.4' open vesicle layer angled 30° to core axis
- 140.1' grout to 1/2" filling an irregular, rough-surfaced opening in a vesicular zone
- 140.1'-142.2' vesicular zone, aligned vesicle layers, voids to 1/2"
- 162.0' smooth planar joint at 50° to core axis, grout filled to 1/16"
- 165.9'-166.1' sand grout to 3/8" filling a smooth, planar joint oriented 45° to core axis
- 169.3' sand grout to 1/2" filling a rough-surfaced fracture oriented 40° to core axis
- 194.3' grout coating in a rough fracture oriented 50° to the core axis
- 260.2' grout to 1/16" filling a smooth, planar joint oriented 50° to the core axis
- 260.3' grout to 1/16" filling a smooth, planar joint oriented 50° to the core axis
- 270.1'-270.6' smooth, planar joint at 18° to core axis
- 272.4'-272.9' grout to 1" in a rough fracture, intersected by smooth, planar, calcite-lined joints oriented at 35° to the core axis
- 332.0'-333.5' smooth, planar joint with silt and calcite coatings, oriented 10° to core axis
- 389.7'-389.9' grout to 1/4" filling a smooth, planar joint oriented 35° to core axis
- 408.6' smooth, planar joint at 40° to core axis, calcite coating

APPENDIX F-35A
HOLE SURVEY

HOLE DESIGNATION: DH-652
 BEARING: N19W (Surface measurement)
 ANGLE: 34° to right from vertical
 (Surface measurement)

LOCATION: At STA 5+11.2 5.5 ft.
 upstream of centerline
 of dam
 ELEVATION: 5333

SPERRY SUN SURVEY:		
<u>Depth</u> <u>ft.</u>	<u>Angle to right</u> <u>from vertical</u>	<u>Bearing</u>
100	36	N17°W
110	35.9	N17°W
120	35.8	N17°W
130	35.5	N17°W
140	35.5	N16°W
150	35.5	N16°W
160	35	N17°W
170	35	N17°W
180	35	N17°W
190	35	N17°W
200	34.5	N17°W
210	34.5	N16°W
220	34	N16°W
230	34	N17°W
240	34	N17°W
250	33.5	N17°W
260	33.5	N17°W
270	33.5	N17°W
280	33	N17°W
290	33	N17°W

A bore-hole camera survey was conducted in this hole by the U.S. Army Corps of Engineers, November 4-6, 1976. Significant findings are noted in document date December 11, 1976, Appendix B.

APPENDIX F-36
GROUT CURTAIN WATER LOSS TESTING
AT SPILLWAY WEIR

Location			Pressure Tests						Gravity Tests
Sta	Hole DH-	Angle	Interval ft	Water Loss gpm	Interval ft	Water Loss gpm	Interval ft	Water Loss gpm	Water Loss gpm
10+82	601	30°R	4.0-40.0	6.9	39.6-74.6	0.2	74.2-109.2	0.3	2.6
	602	0°	4.7-34.7	2.0	34.7-64.7	2.1	64.7- 94.7	0.4	4.5
	603	30°L	4.5-39.5	1.0	40.1-75.1	0.1	73.7-108.7	0.4	2.3
11+06	604	30°R	5.5-40.5	0.8	40.1-75.1	0.5	74.7-109.7	0.2	0.5
	605	0°	6.0-36.0	0.2	36.0-66.0	0.2	66.0- 96.0	0.03	0
	606	30°L	5.6-40.6	1.1	-	-	-	-	-
	606A	30°L	6.8-41.8	0.4	41.4-76.4	0.6	76.0-111.0	0.2	0.4
11+30	607A	30°R	5.3-40.3	1.3	39.9-74.9	0.2	74.5-109.5	0.04	-
	608	0°	5.8-35.8	1.3	35.8-65.8	0.3	65.8- 95.8	0.07	0.6
			95.8-125.8	0.3					
	609	30°L	5.7-40.7	0.8	40.3-75.3	1.9	74.9-109.0	2.3	3.1
			110.0-145.0	0.3					

Interval is measured along axis of hole from the hole collar.
All pressure tests conducted at 10 psi at collar elevation.
All gravity tests conducted full length of hole.

APPENDIX F-37
GROUT CURTAIN WATER LOSS TESTING
IN RIGHT ABUTMENT KEY TRENCH

Location			Pressure Tests				Gravity Tests	
Sta	Hole DH-	Angle	Interval ft	Water Loss gpm	Interval ft	Water Loss gpm	Interval ft	Water Loss gpm
12+75	610	42°R	3.8-14.8	0.38	14.8-24.8	0.05		
	611	20°R	2.5-13.9	0.48	13.9-23.9	0.1		
	612	0°	2.1-13.5	0.5	13.5-23.5	0		
13+15	613	45°R	3.4-14.8	1.1	14.8-24.8	0.2		
	614	22°R	4.4-14.4	1.0	13.4-24.4	0.9	0-24.4	0.5
	615	0°	3.5-14.9	0	14.9-24.9	0		
13+30	616*	47°R	2.9-14.3	10.	7.9-14.3	0.5	0-14.3	3.5
			14.3-24.3	5.2	18.9-24.3	0	0-24.3	4.5
	617	21°R	2.5-13.9	0	13.9-23.9	0	0-23.9	0.4
	618	0°	2.8-14.2	0	14.2-24.2	0.8		
	619*	31°L	5.3-16.7	1.6	16.7-26.7	4.8	0-26.7	4.8
13+50	620	44°R	4.4-14.4	0.1	14.4-24.4	0.1		
			24.4-34.2	0.1				
	621	22°R	3.5-13.5	5.1	13.5-23.5	0.1		
			3.5-23.5	7.9	3.5-23.5	13.0	(20 psi)	
	622	0°	2.1-13.5	1.9	13.5-23.5	0		
	623*	22°L	3.4-13.4	12.7**			0-13.4	14.1**
11.4-21.4			1.2	6.4-21.4	1.6			
13+77	624*	45°R	2.9-14.3	3.5	14.3-24.3	0	0-24.3	1.8
	625	0°	3.9-15.3	0.4	15.3-26.3	0	0-26.3	0.2
	626*	22°L	5.3-16.7	12.5	16.7-26.7	7.5	(2 psi) 0-26.7	4.5

APPENDIX F-37 (Cont'd)
 GROUT CURTAIN WATER LOSS TESTING
 IN RIGHT ABUTMENT KEY TRENCH

Location			Pressure Tests				Gravity Tests	
Sta	Hole DH-	Angle	Interval ft	Water Loss gpm	Interval ft	Water Loss gpm	Interval ft	Water Loss gpm
14+10	627	46°R	1.6-11.0	0.2	11.0-21.0	0.1	0-21.7	0.03
	628	0°	4.6-11.0	0	11.0-21.0	0.14	0-21	0.1
	629*	34°L	1.0-11.0	7.1	2.6-11.0	0.7	0-21	0.3
			11.0-21.0	0.7				
14+26	630*	47°R	1.3-11.7	4.3	11.7-21.7	0.4	0-21.7	0.9
	631	0°	1.2-11.6	0.3	11.6-21.6	0.2	0-21.6	0.03
	632	34°L	1.7-13.1	2.6	13.1-23.1	1.6	0-23.1	0.7

Interval is measured along axis of hole from the collar.
 All pressure tests conducted at 10 psi at collar elevation except where noted.

*Return flows observed from joints and fractures downstream of grout cap.

** During drilling, a 50 percent water loss occurred at a depth of 2.4 ft or 1 ft below the base of the grout cap.

APPENDIX F-38

DRILL HOLE WATER PRESSURE TEST

HOLE DESIGNATION: DH-650
 BEARING: S19°E
 ANGLE: 30° to left from vertical
 DEPTH: 351.5 ft.

LOCATION: At STA 3+00 on dam
 crest 4.7 ft. up-
 stream of centerline
 Date drilled: 10/5 to 10/15/76
 ELEVATION: 5229

<u>DEPTH INTERVAL TESTED ft.*</u>	<u>WATER LOSS gpm</u>	<u>COMMENTS</u>
90.0-97.7	32.1	
99.8-104.8	32.9	
103.1-127.4	28.3	
127.6-162.6	13.4	
160.6-197.2	8.4	
197.6-232.6	0.0	
232.6-267.5	20.3	
267.5-302.5	6.1	
301.7-331.7	2.1	
331.5-351.5	0.0	

cased to 90.0; lost drilling water return at 91.6, never recovered

* footage measured along axis of hole from collar which is 0.4 feet above land surface

All test conducted at 10 psi measured at hole collar

APPENDIX F-39

DRILL HOLE WATER PRESSURE TEST

HOLE DESIGNATION: DH-651
 BEARING: ---
 ANGLE: Vertical
 DEPTH:

LOCATION: At STA 4+34 on
 centerline of dam
 Date drilled: 9/29 to
 ELEVATION: 5232

<u>DEPTH INTERVAL TESTED</u>	<u>WATER LOSS gpm</u>	<u>COMMENTS</u>
80.0-100.0	0.2	
100.0-120.0	0.2	
120.0-140.0	1.0	
140.0-160.0	0.4	
160.0-180.0	0.0	
180.0-200.0	0.0	
200.0-220.0	0.5	
219.9-239.9	0.4	
239.9-259.9	0.5	
259.9-279.9	0.4	
279.9-299.9	0.6	
299.9-319.9	1.3	
319.9-359.9	0.6	
359.9-399.9	1.3	
399.9-439.9	0.2	
432.4-472.4	0.0	
479.3-519.3	0.4	packer in grouted zone
517.7-527.2	2.2	
518.2-535.9	1.2	
530.8-543.7	6.2	
546.1-552.3	9.3	packer in casing at 499.7
546.1-566.1	2.1	packer in casing at 539.4
559.3-581.2	2.7	
560.0-600.0	35.0	hole drilled to 600.0, but bottom "washed in" to 580. could not get 10 psi at maximum pump capacity of 35 gpm

lost 75-80% of drilling water in Zone 1 material at 47.2 ft.; concrete/Zone 1 contact at 78.8 ft.

All tests run at 10 psi measured at hole collar

APPENDIX F-40

DRILL HOLE WATER PRESSURE TEST

HOLE DESIGNATION: DH-652
 BEARING: N18°W
 ANGLE: 33° to right from vertical
 DEPTH: 450 ft.

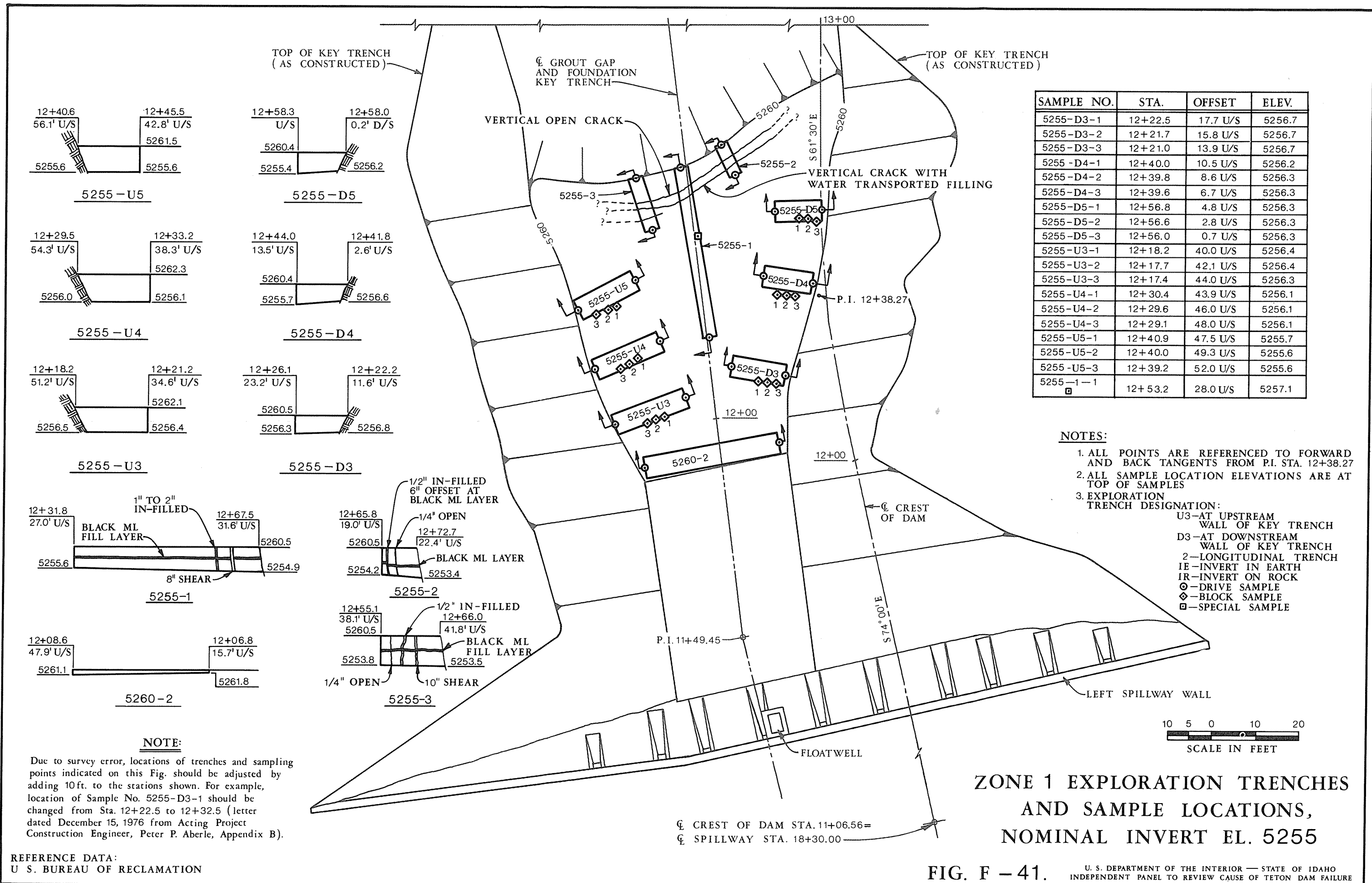
LOCATION: At STA 5+11.2 on dam
 5.5 ft. upstream of
 centerline
 Date drilled: 10/2 to
 ELEVATION: 5332

<u>DEPTH INTERVAL TESTED ft.*</u>	<u>WATER LOSS gpm</u>	<u>COMMENTS</u>
95.0-130.0	0.4	
130.0-165.0	0.4	
165.0-200.0	1.4	
200.0-235.0	0.1	
235.0-270.0	0.8	
267.9-302.9	1.2	
301.3-307.7	12.8	lost drilling water at 303
307.7-347.7	0.8	
347.7-387.7	0.3	
387.4-427.4	0.1	
425.0-450.0	0.5	

* footage measured along axis of hole from collar which is 0.4 feet above land surface

cased to 90.0 originally, then driven to 94.0

All tests run at 10 psi measured at hole collar



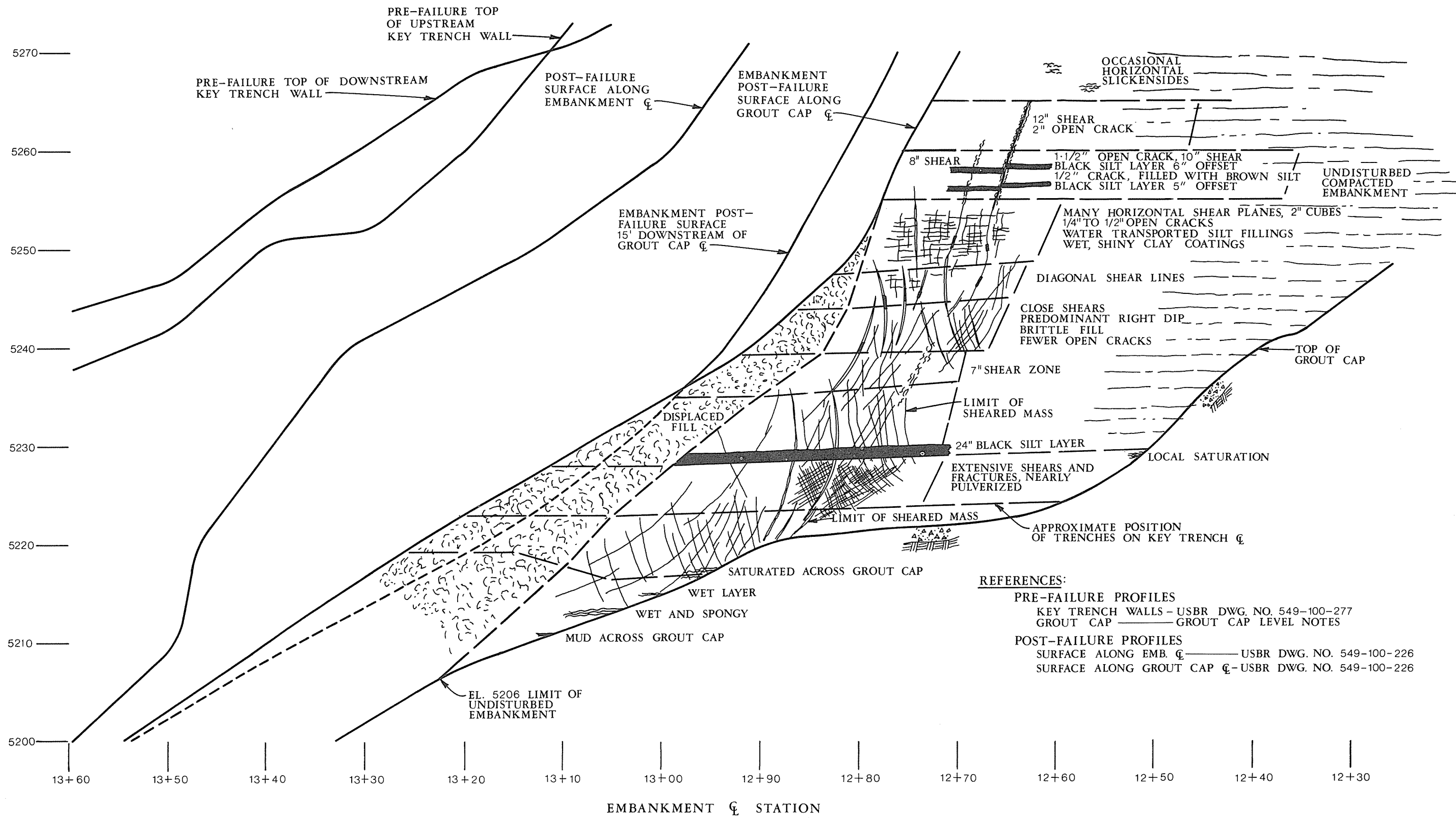
SAMPLE NO.	STA.	OFFSET	ELEV.
5255-D3-1	12+22.5	17.7 U/S	5256.7
5255-D3-2	12+21.7	15.8 U/S	5256.7
5255-D3-3	12+21.0	13.9 U/S	5256.7
5255-D4-1	12+40.0	10.5 U/S	5256.2
5255-D4-2	12+39.8	8.6 U/S	5256.3
5255-D4-3	12+39.6	6.7 U/S	5256.3
5255-D5-1	12+56.8	4.8 U/S	5256.3
5255-D5-2	12+56.6	2.8 U/S	5256.3
5255-D5-3	12+56.0	0.7 U/S	5256.3
5255-U3-1	12+18.2	40.0 U/S	5256.4
5255-U3-2	12+17.7	42.1 U/S	5256.4
5255-U3-3	12+17.4	44.0 U/S	5256.3
5255-U4-1	12+30.4	43.9 U/S	5256.1
5255-U4-2	12+29.6	46.0 U/S	5256.1
5255-U4-3	12+29.1	48.0 U/S	5256.1
5255-U5-1	12+40.9	47.5 U/S	5255.7
5255-U5-2	12+40.0	49.3 U/S	5255.6
5255-U5-3	12+39.2	52.0 U/S	5255.6
5255-1-1	12+53.2	28.0 U/S	5257.1

- NOTES:**
- ALL POINTS ARE REFERENCED TO FORWARD AND BACK TANGENTS FROM P.I. STA. 12+38.27
 - ALL SAMPLE LOCATION ELEVATIONS ARE AT TOP OF SAMPLES
 - EXPLORATION TRENCH DESIGNATION:
 U3-AT UPSTREAM WALL OF KEY TRENCH
 D3-AT DOWNSTREAM WALL OF KEY TRENCH
 2-LONGITUDINAL TRENCH
 IE-INVERT IN EARTH
 IR-INVERT ON ROCK
 ⊙-DRIVE SAMPLE
 ⊠-BLOCK SAMPLE
 ⊡-SPECIAL SAMPLE

NOTE:
 Due to survey error, locations of trenches and sampling points indicated on this Fig. should be adjusted by adding 10ft. to the stations shown. For example, location of Sample No. 5255-D3-1 should be changed from Sta. 12+22.5 to 12+32.5 (letter dated December 15, 1976 from Acting Project Construction Engineer, Peter P. Aberle, Appendix B).

REFERENCE DATA:
 U S. BUREAU OF RECLAMATION

ZONE 1 EXPLORATION TRENCHES AND SAMPLE LOCATIONS, NOMINAL INVERT EL. 5255



- REFERENCES:**
- PRE-FAILURE PROFILES
 - KEY TRENCH WALLS - USBR DWG. NO. 549-100-277
 - GROUT CAP - GROUT CAP LEVEL NOTES
 - POST-FAILURE PROFILES
 - SURFACE ALONG EMB. ζ - USBR DWG. NO. 549-100-226
 - SURFACE ALONG GROUT CAP ζ - USBR DWG. NO. 549-100-226

**LONGITUDINAL EXPLORATION TRENCHES
GENERAL OBSERVATIONS**

FIG. F-42. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

APPENDIX G

LIST OF ADDITIONAL REFERENCES ON FILE

APPENDIX G
LIST OF ADDITIONAL REFERENCES
ON FILE

1. "Summary — Soils Classification Tests," Teton Project Laboratory, September-October 1976.
2. Questions Asked, and Answers, of USBR by U.S. Department of Interior Teton Dam Failure Review Group Relative to Design, Beginning September 1, 1976.
3. "Triaxial Compression Tests, Unconfined Compression Tests, Teton Dam," Northern Testing Laboratories, Inc., Billings, Montana, October 1976.
4. "Evaluation of Erodibility Characteristics of Teton Dam Soils," K. Arulanandan, October 1976.
5. "Stress-Strain Characteristics of Zone 1 Material, Teton Dam," Earth Sciences Branch, USBR, December 6, 1976.
6. "Dispersion Characteristics of Soil from Teton Dam, Idaho," Edward B. Perry, Waterways Experiment Station, December 1976.
7. "Report of Soil Tests, Nx Core Sample, Teton Dam," Geo-Testing, Inc., December 1976.
8. "Summary Report of Laboratory Tests of Zone 1 Embankment Material, Teton Dam," Office of Design and Construction, USBR.
9. L-10 Progress Report, "Pilot Grouting, Lower Teton Division, Teton Basin Project," Idaho, USBR, undated.