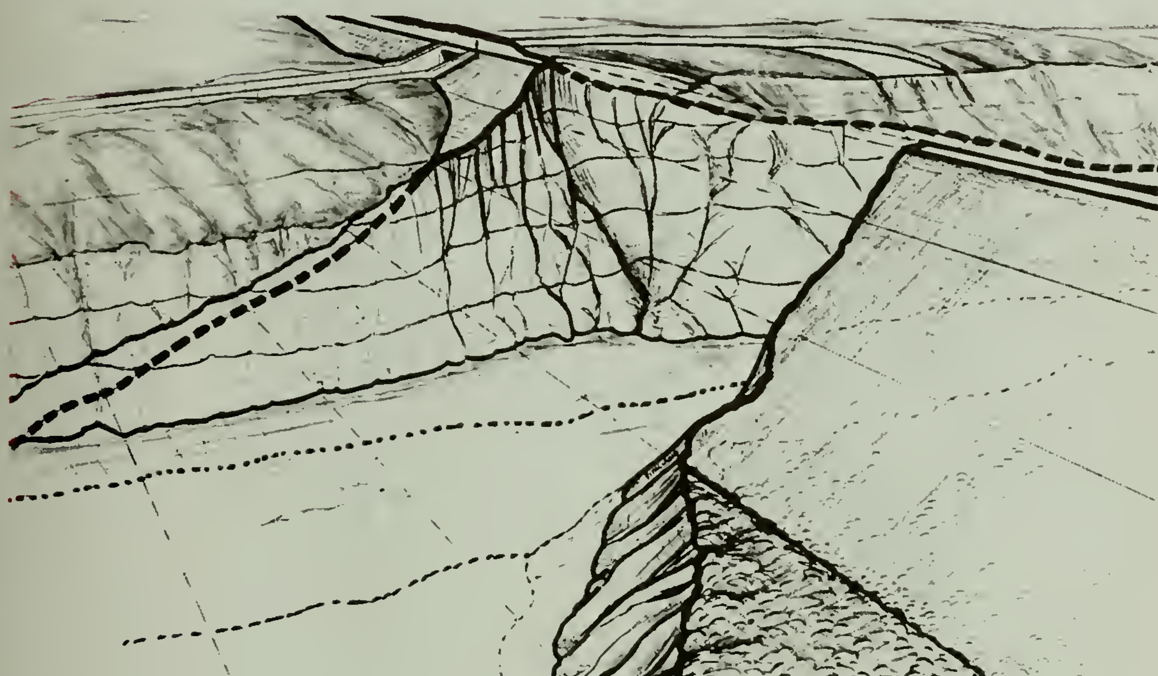
FAILURE OF TETON DAM

A REPORT OF FINDINGS



by
U.S. DEPARTMENT OF THE INTERIOR
TETON DAM FAILURE REVIEW GROUP
April 1977

Frontispiece.—Aerial view of Teton damsite after the failure of the dam.

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F. WILLIAM EIKENBERRY, CHAIRMAN
HAROLD G. ARTHUR
NEIL F. BOGNER
FLOYD P. LACY, JR.
ROBERT L. SCHUSTER
HOMER B. WILLIS

April 1977

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Transmittal Document

In compliance with the request of the Under Secretary of the Interior, dated June 8, 1976, the Department of the Interior Teton Dam Failure Review Group submits this report on the Teton Dam failure.

Respectfully submitted by:

F. William Eikenberry
F. William Eikenberry, Chairman,
Office of the Assistant Secretary–
Land and Water Resources, Department
of the Interior

H. G. Arthur
Harold G. Arthur, Director of
Design and Construction,
Bureau of Reclamation

Neil F. Bogner
Neil F. Bogner, Director,
Engineering Division, Soil
Conservation Service, Department
of Agriculture

Floyd P. Lacy, Jr.
Floyd P. Lacy, Jr., Assistant
Director of Division of Engineering
Design, Tennessee Valley Authority

Robert L. Schuster
Robert L. Schuster, Chief,
Engineering Geology Branch,
Geological Survey

Homer B. Willis
Homer B. Willis, Chief of
Engineering Division, Directorate
of Civil Works, Corps of Engineers

April 1977

Abstract

This report is the result of a study by the Department of the Interior Teton Dam Failure Review Group to examine the causes of failure of the Teton Dam and make recommendations to prevent the recurrence of such failure. The study included site inspections, interviews, and examination of design and construction data. Specific questions were directed to the Bureau of Reclamation on design processes and decisions. Postfailure field, laboratory, and office investigations were also conducted. Three task groups were formed to make indepth reviews of geology, foundation grouting, and embankment construction. A continuing portion of the study, not reported herein, is a review of the Bureau of Reclamation's technical decisionmaking process and further field investigations.

Teton Dam is located in a steep-walled canyon carved into a gently rolling silt covered volcanic upland. The rock that forms the canyon walls and which constitutes the major bedrock formation throughout the damsite and reservoir area is a rhyolite welded ash-flow tuff. The rock is characterized by the presence of prominent and abundant open joints and fissures.

Considering all site selection factors, the site selected for the dam was the best of the available alternate sites for the purposes of the project. The preconstruction geologic investigations were sufficient to identify the important geologic features that needed to be considered for design.

The dam was constructed in accordance with the intent of the designers and in agreement with the plans and specifications. The dam was a 305-foot-high zoned earthfill embankment with a thick central silt core. A gated spillway was located at the top of the right abutment; a powerhouse was located in the base of the left abutment. The main river outlet works was located in a tunnel in the left abutment; an auxiliary outlet works was located in a tunnel in the right abutment. A cutoff trench to rock was excavated in the valley section. Deep key trenches were provided in both abutments. A deep grout curtain was constructed under the entire length of the dam.

The dam failed as a result of inadequate protection of the impervious core material from internal erosion. The most probable physical mode of failure was cracking of the impervious core material either due to hydraulic fracturing or differential settlement within the embankment that allowed the initiation of erosion. Somewhat less probable is the concept that damaging seepage started at the contact of the zone 1 (impervious core) material and the rock surface. The open fractures in the abutment foundation rock allowed direct access by reservoir water to the impervious core on the upstream side of the key trench. Any water flowing through the impervious core could exit into open fractures on the downstream side of the key trench. The design failed to provide a defense against flow through embankment cracks or against erosion of the impervious core at rock surfaces. The rock surface was not adequately sealed under the impervious core upstream and downstream from the key trench. Defensive measures were within the state-of-the-art of dam design at the time Teton Dam was designed, and should have been used.

The major recommendations to minimize the possibility of recurrence of a failure such as that

at Teton Dam are; (1) an independent board of review should be convened for each major dam project to review both design and construction at frequent intervals, (2) design decisions should be formally documented, and (3) design personnel should remain involved with a project during construction, including frequent scheduled site visits.

Preface

This report presents the results of the Department of the Interior Teton Dam Failure Review Group's investigation. This failure was a major disaster in terms of damage caused and loss of life. The lessons learned from this and other investigations will be of immeasurable value if they prevent the occurrence of other such disasters.

The Interior Review Group (IRG) was assisted during the investigation by members of the participants' staffs. Alternate members of the IRG were Donald Giampaoli, Bureau of Reclamation; Ernest Dodson, Corps of Engineers; David Ralston, Soil Conservation Service; and J. T. McGill, Geological Survey. C. J. Monahan (Corps of Engineers, Retired), served as the IRG field representative.

Much of the investigation was performed by the Geology, Grouting, and Embankment Construction Task Groups. The members are:

Geology Task Group: Robert Schuster, Chairman, J. T. McGill and D. J. Varnes, Geological Survey; and Lloyd Underwood, Corps of Engineers. Assisting in the geologic investigations were E. G. Crosthwaite, M. S. Bedinger, S. S. Oriel, H. J. Prostka, D. M. Perkins, and D. A. Swanson, Geological Survey; Brent Carter, Dan Magleby, and Daniel Hubbs, Bureau of Reclamation; and Paul Huebschman, John Stanton, Millard Stone, and Walter Wickbolt, Corps of Engineers.

Grouting Task Group: Paul Fisher, Chairman, Corps of Engineers; James Coulson, Tennessee Valley Authority; and Ray Cope, Soil Conservation Service.

Embankment Construction Task Group: David Ralston, Chairman, Soil Conservation Service; Neil Parrett, Corps of Engineers; and Samuel Stone, Tennessee Valley Authority. Assisting in investigations of the design and construction of the embankment were Ralph Beene, Stanley Johnson, and John Palmerton, Corps of Engineers.

Bureau of Reclamation Personnel making major contributions: Robert Robison, Project Construction Engineer; Peter Aberle, Field Engineer; Ralph Mulliner, Materials Engineer; and the rest of the Teton Project staff were very helpful, as were Lloyd Gebhart, Construction Liaison Engineer; Robert Farina, Geologist; Richard Kramer and Luther Davidson, Soil Engineers, Engineering and Research Center (E&R Center) staff. Sam Guy, also of the E&R Center, served as investigation coordinator. His services are greatly appreciated.

Assisting in the completion of this report were Brenda Tremper and Irene Murphy, Office of the Assistant Secretary for Land and Water Resources, Department of the Interior. Paul Fisher, Corps of Engineers, and James Coulson, Tennessee Valley Authority, made a major contribution by their overall coordination and technical editing of this report.

The Interior Review Group wishes to express its appreciation to Dennis N. Sachs, former Deputy Assistant Secretary, Land and Water Resources who provided strong leadership during his service as Chairman of the IRG from its formation until February 25, 1977.

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FIGURES

fault — A surface or zone of rock fracture along which there has been displacement, from a few inches to many miles in scale.

flume — An artificial channel, commonly an inclined chute or trough, for carrying water.

foundation grouting — The practice of injecting a water-cement grout mixture into the foundation of an engineered structure for the purpose of reducing water seepage and/or strengthening the foundation.

fumaroles — A vent, usually volcanic, from which gases and vapors are emitted; it is characteristic of a late stage of volcanic activity.

grout — A cementitious material of high water content, fluid enough to be poured or injected into spaces and thereby fill or seal them.

grout curtain — A curtain-shaped zone or rock or soil treated by grout injection holes in order to reduce the amount of water seepage through the rock or soil.

high-angle — Indicates that the inclination of the feature being discussed is greater than 45° from the horizontal.

hydraulic fracturing — The fracturing of a material, such as the material composing an embankment, by excess fluid pressure between the constituent particles of the material.

hydraulic gradients — The rate of change of hydrostatic pressure per unit of distance of flow at a given point and in a given direction.

hydraulic uplift — The upward force exerted on a material by fluid within the material.

in situ stress — The stress present within a soil or rock mass.

joint — A surface of actual or potential fracture or parting in a rock without displacement.

key trench — A deep, narrow trench cut into a dam foundation for the purpose of cutting off waterflow.

lacustrine — Pertaining to, produced by, or formed in a lake or lakes.

leveling — The operation of determining the comparative altitude of different points on the earth's surface.

low-angle	Indicates that the inclination of the feature being discussed is less than 45° from the horizontal.
Modified Mercalli Scale	One of the earthquake intensity scales, having twelve divisions ranging from I (not felt by people) to XII (damage nearly total).
monument	A natural or artificial (but permanent) physical structure that marks the location on the ground of a survey point.
ogee	An S-shaped curve, for example, the curved crest of a spillway structure.
perched ground water	Unconfined groundwater separated from an underlying main body of groundwater by an unsaturated zone.
percolation	Flow of water, usually downward, through small openings in a porous material.
permeability	The property of a porous rock or soil medium for transmitting a fluid without impairment of the structure of the medium.
piezometric surface	An imaginary surface representing the static head of groundwater and defined by the level to which water will rise in a well.
piping	Erosion by percolating water in a soil resulting in caving and the formation of narrow conduits, tunnels, or pipes through the soil.
Pliocene	An epoch of the Tertiary period, ranging in time from about 2 to 6 million years ago.
Proctor	A method developed by R. R. Proctor for measuring the degree of compaction of a soil.
pyroclastic	Pertaining to particulate rock material formed by volcanic explosion or aerial expulsion from a volcanic vent.
rhyolite	An extrusive igneous (volcanic) rock having essentially the same composition as granite.
Richter scale	The range of numerical values of earthquake magnitude devised in 1935 by seismologist C. F. Richter. The scale is logarithmic and is arranged so that very small earthquakes can have negative magnitude values. The strength of earth materials imposes an upper limit for possible magnitude of slightly less than 9.
seepage	The act or process involving the slow movement of water or other fluid through a porous material such as soil or rock.

seismicity	The phenomenon of earth movements (earthquakes or earth vibrations).
Shelby tube	A soil sampling device consisting of thin-wall tubing which is driven into a soil to obtain a sample.
siltstone	A rock composed largely of silt.
sinkhole	A closed depression formed by solution of caving of a rock or soil material.
slopewash	Soil and rock material that is or has been transported down a slope by gravity assisted by running water not confined to channels.
slurry	A very wet, highly mobile, semiviscous mixture or suspension of finely divided, insoluble material.
tectonics	A branch of geology dealing with the broad architecture of the upper part of the earth's crust; that is, the regional assembling of structural and deformational features.
triaxial strength text	A text of the strength of a soil or rock in which a cylindrical sample is subjected to an all-around confining pressure and then subjected to an increasing axial load until it breaks. The test may be drained (internal sample fluid allowed to drain away).
trilateration	A method of surveying in which the lengths of the three sides of a series of touching or overlapping triangles are measured (usually by electronic methods) and the angles are computed from the measured lengths.
turbid	Stirred up or disturbed, such as by sediment; not clear or translucent, being opaque with suspended matter.
upstream cofferdam	A temporary dam placed upstream from the location of the permanent dam for the purpose of allowing construction of the main dam in dry conditions.
void	An opening in a rock or soil not occupied by solid matter.
volcanism	The processes by which magma (mobile rock material) and its associated gases rise into the crust and are extruded onto the earth's surface and into the atmosphere.
welded tuff	A pyroclastic rock which has been made rocklike by the combined action of the heat retained by particles, the weight of the overlying materials, and hot gases.

Introduction

Formation and Charge of the Interior Review Group

On June 8, 1976, the then Under Secretary of the Interior, D. Kent Frizzell established the Department of the Interior Teton Dam Failure Review Group (IRG), composed of representatives of selected Federal agencies. The IRG was formed to examine the causes of the failure of Teton Dam and to make recommendations, as appropriate, to prevent the recurrence of such failures. The Under Secretary directed that the IRG review the following aspects of the failure: geologic, engineering design, construction, hydrologic factors, and all other pertinent background information and testimony. (See Appendix A.)

The IRG is composed of representatives from the Soil Conservation Service (Department of Agriculture), Geological Survey (Department of the Interior), Bureau of Reclamation (Department of the Interior), Corps of Engineers (Department of the Army), and Tennessee Valley Authority. Dennis N. Sachs, Deputy Assistant Secretary of the Interior for Land and Water Resources, served as Chairman from June 1976 until February 1977.

Three task groups were formed to make indepth studies: one for geology, the second for foundation grouting, and a third for embankment construction. These task groups consisted of members of the IRG and/or their staffs. Each of the task groups made one or more visits to the damsite to inspect site conditions, interview construction personnel, and examine construction records. Reports of these three groups are found in Appendixes B, C, and D.

Other Investigations of Teton Dam Failure

Another review group, composed of experts not associated with the Federal Government, was formed by the then Secretary of the Interior, Thomas S. Kleppe and the then Governor of Idaho, Cecil D. Andrus. The charge to this Independent Panel[1] was essentially the same as that directed to the IRG. The Independent Panel and the IRG operated simultaneously from June to December 1976. Field investigations directed by both the IRG and the Independent Panel were coordinated to avoid duplication of effort. The results of all investigations were shared by the two groups, but their analyses and conclusions were arrived at independently. The conclusions of the Independent Panel were confidential until the publication of its final report on January 6, 1977.

Other reviews of the failure of the dam have been undertaken by the General Accounting Office at the request of the Energy and Natural Resources Subcommittee of the House Committee on Government Operations, and by the Congressional Research Service, at the request of a subcommittee of the Senate Committee on Interior and Insular Affairs.

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[1] Independent Panel to Review Cause of Teton Dam Failure

The Department of the Interior has recognized the need for a review of Bureau of Reclamation procedures beyond the scope of activities of the Independent Panel or the IRG. A comprehensive review of these procedures is now underway: (1) a nontechnical review of the Bureau of Reclamation's administrative procedures and guidelines [completed December 1976]; (2) a technical review of the Bureau's decisionmaking procedures and criteria [under contract for 1 year beginning in April 1977]; and (3) an assessment of the safety of existing Bureau dams by the National Academy of Sciences' National Research Council.

Previous Reports

The IRG has published two interim reports covering the progress of investigations and the development of hypotheses concerning the modes of failure. The first report, completed on July 14, 1976, included general information about the dam and its failure as well as recommendations for the future activities of the IRG. This report listed seven aspects of design and construction of the dam that might have contributed to the development of internal erosion and the rapid failure.

On October 21, 1976, a second interim report was completed. It offered a more detailed description of six possible causes of failure, ranked by relative probability of occurrence. Each of these was discussed in some detail. Foremost on the list was seepage and resultant piping due to cracking or hydraulic fracturing of the impervious core of the embankment (zone 1) material.

Status of the IRG Review

A substantial portion of the investigative work required by the charge to the IRG is complete. In general, conclusions presented here are based on evidence acquired through detailed examination of all available data. Extensive studies were made of those portions of the right abutment, key trench, and embankment that remained after failure of the dam. The destruction of the right side of the embankment and a portion of its foundation removed much of the direct evidence of the cause of failure. It is conceivable that conditions in the left embankment remnant closely resemble prefailure conditions in the right embankment remnant. Specifically, evidence may exist of cracking and piping of the zone 1 material. Mapping of the postfailure surface cracks in the crest of the left embankment remnant has been completed. Following the 1976-77 winter shutdown, a detailed investigation will be made of the left embankment remnant. In addition, some additional rock core borings will be drilled along the grout curtain in the right abutment. These two tasks are the only major field work items remaining and they may provide further confirmation of the conclusions presented herein.

In addition to these field investigations, the IRG will review the technical procedures and decisionmaking process used during design and construction of Teton Dam. Completion of these investigations and review will facilitate the formulation of recommendations to reduce the risk of future failures.

General Project Description

Description of Facilities

The Teton Dam and Reservoir are the principal features of the Lower Teton Division, Teton Basin Project, Idaho, a multipurpose project designed to serve the objectives of irrigation, power production, flood control, and recreation.

The project was designed to provide a supplemental water supply for 111,250 acres of irrigated lands and for flood control. Supplemental water was also to be made available for additional lands. A 16,000-kW generator was provided for power production. The location of the project is shown on Figure 1.

The reservoir was 17 miles long, with 200,000 acre-feet of active capacity, 87,780 acre-feet of inactive capacity, and 470 acre-feet of dead storage, for a total capacity of 288,250 acre-feet. The design of the dam and appurtenant features is described in this chapter. Recreation facilities were to be located on the south shore of the reservoir immediately upstream of the dam. The layout of the dam, reservoir, and appurtenant structures is shown on Figure 2.

Twenty-seven water replacement wells were to be drilled in the Snake River Plain Aquifer downstream from the dam. In dry years, these wells were to be used to replace water required by holders of senior river water rights.

Project Siting

Siting of Teton Dam was a major factor in determining the scope of the project. While there have been reconnaissances of power and storage sites on the Teton River since 1904, the first actual investigation was conducted in 1932 at a site 15 miles upstream and due east of the site where the dam was eventually constructed. The 1932 site was discussed by the U.S. Geological Survey in "Water-Supply Paper 657," dated 1935. The scope of the project evolved over the years. Initially, a run-of-the-river powersite with limited storage capacity was considered; however, ultimately it was decided to control all river drainage so that maximum benefits for flood control and irrigation would result. In determining the economics of the proposals studied, the yield of the lands to be irrigated, the cost of distributing water to them, the flood control benefits, and the total project costs were considered.

In 1946, the Bureau of Reclamation investigated two sites on Canyon Creek, a tributary to the Lower Teton River. A March 1947 report showed that these sites were not economical, and that seepage losses from the reservoirs could be expected. These sites were 9 miles southeast of Teton Dam and, because of their locations on a tributary, could not have provided control for all the Teton River flow.

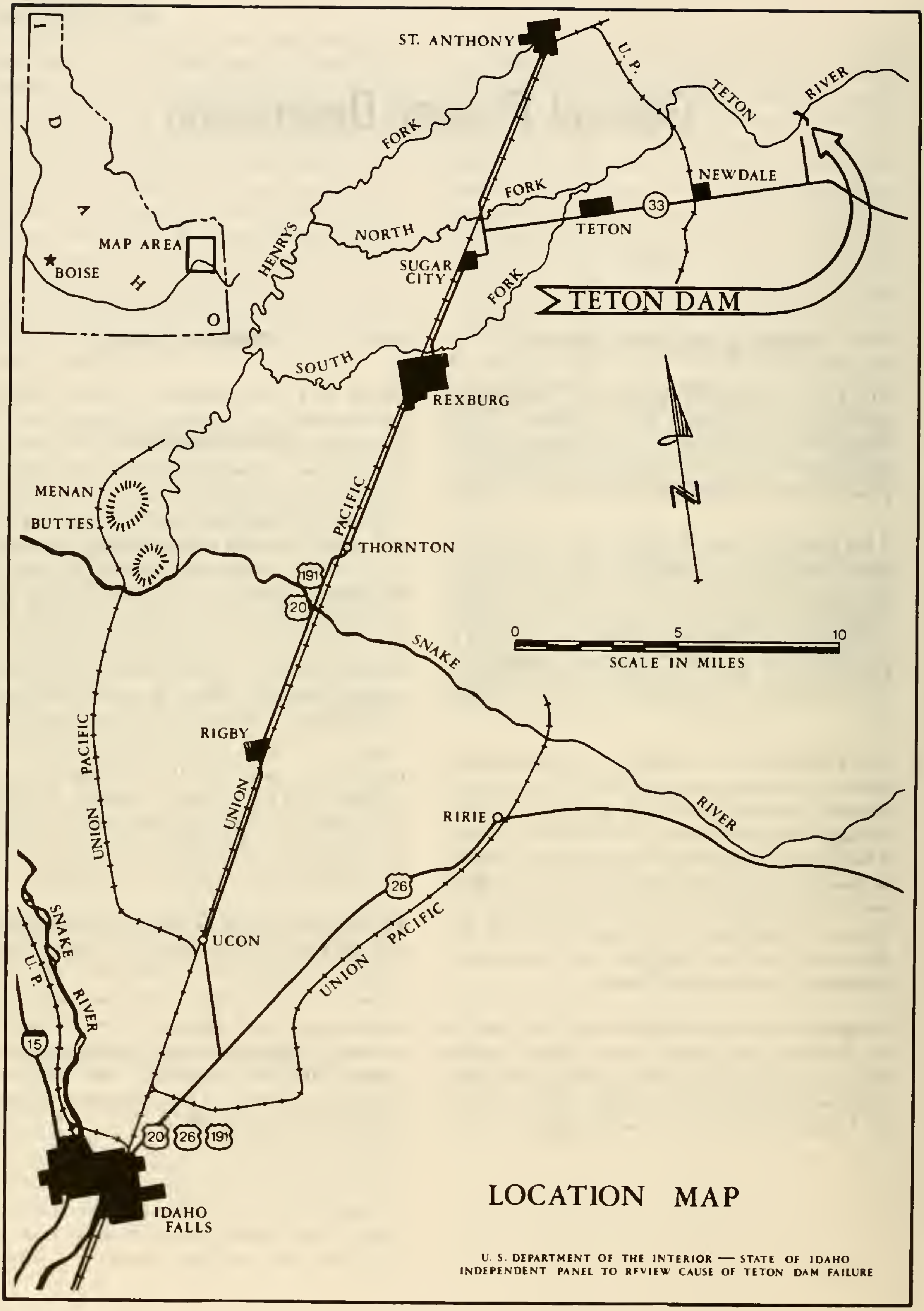


Figure 1.—Teton Dam location map.

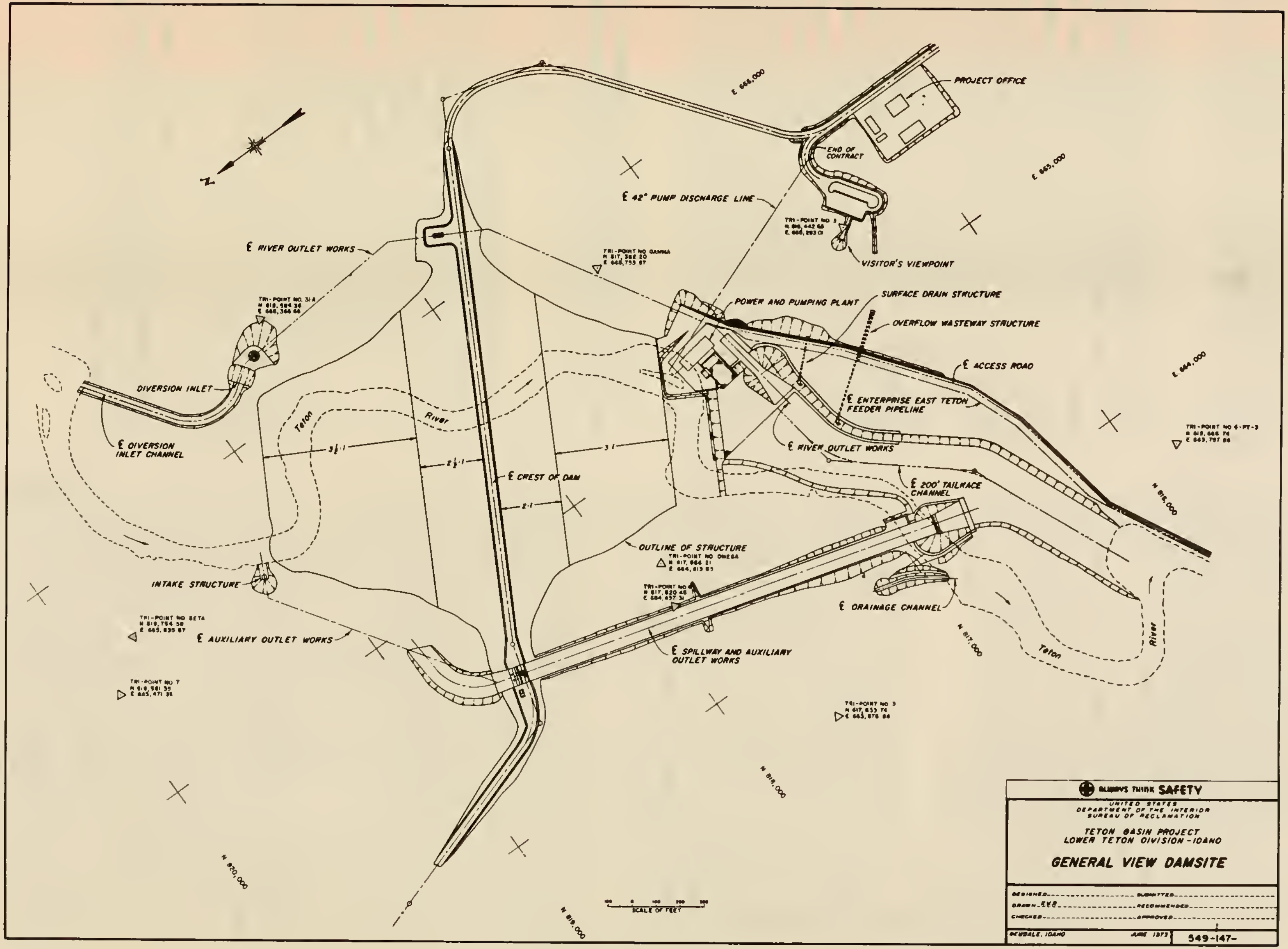


Figure 2.—General project map.

In 1956, the Bureau investigated a scheme for diverting water from the Teton River to provide flood control and other benefits. The scheme included a 46-foot-high dam at the mouth of the Teton River Canyon and a canal trending northwest to the Snake River Plain. The scheme did not provide irrigation storage or water for irrigation of additional lands. Flood control for lands along the lower reach of the Teton River was the primary benefit. Secondary benefits included recharge of groundwater aquifers under the Snake River Plain. In March 1957, the Bureau reported that the scheme was not geologically feasible.

The Corps of Engineers selected a site for investigation at the location of Teton Dam and drilled two holes in July 1957. One of these holes was drilled through the alluvium underlying the valley floor; the other hole was drilled in the left abutment. The quality of rock encountered by this drilling was considered to be structurally adequate for a dam. It was recognized that seepage from the reservoir would occur. Further studies to determine the impact of seepage on the economics of the site were considered necessary.

In the fall of 1956, a joint Bureau of Reclamation and Corps of Engineers committee was formed to assign responsibilities for investigations in the Upper Snake River Basin to one of these agencies. This committee agreed to assign investigations of the Teton Project to the Bureau of Reclamation. In 1961, the Corp of Engineers and the Bureau of Reclamation issued a joint report on the Upper Snake River Basin. The Teton Project was included in this report.

The Bureau of Reclamation prepared a reconnaissance-type geologic report for Teton Dam (Fremont Site) in January 1961 and started core drilling at the site in July 1961. A second report describing a dam and ancillary works, with a layout similar to that which was eventually built, was issued in March 1962. By this time, the Bureau had completed five holes for a total of seven core holes at the site. In March 1962, the Corps issued Interim Report R, "Review Report on Columbia River and Tributaries," covering the Lower Teton Project, and the Bureau issued a report entitled "Teton Basin Project, Lower Teton Division." The Bureau report recommended construction of the Teton Project, including Teton Dam (Fremont Site).

Additional siting studies were conducted by the Bureau in late 1961 and early 1962, and geologic studies were made at five sites along the river between the mouth of Linderman Draw and the mouth of the Little North Fork. The apparent advantage of a site in this reach of the river was that a greater amount of land could be irrigated. The disadvantages were found to be some loss of floodflow regulation and economic considerations, such as increased cost of distribution and reduced climatic suitability of land to be irrigated. The sites were found to be as geologically feasible as the Teton (Fremont) site, without benefits to be gained with respect to the quality of foundation rock or seepage losses from the reservoir. A report describing site conditions and their influence on the selection of a site was issued in March 1962. The locations of the various alternative damsites are shown on Figure 3.

Subsequent to this report, all investigations were directed toward developing data and information for the Teton Dam and Reservoir at the present location.

In reviewing the siting progress for Teton Dam, it is apparent that acquisition of geologic knowledge of the damsite and reservoir was given high priority. A number of the reports were prepared by geologists. It was recognized early that the reservoir rim could transmit water in large quantities and that seepage from the reservoir could occur. Geologic conditions at all the sites studied were found to be comparable, as were reservoir seepage conditions. The sites differed primarily in their effects on the features to be included in the project.

Investigations of the site by means of a number of geotechnical methods continued through the design and construction periods. These are discussed elsewhere in this report.

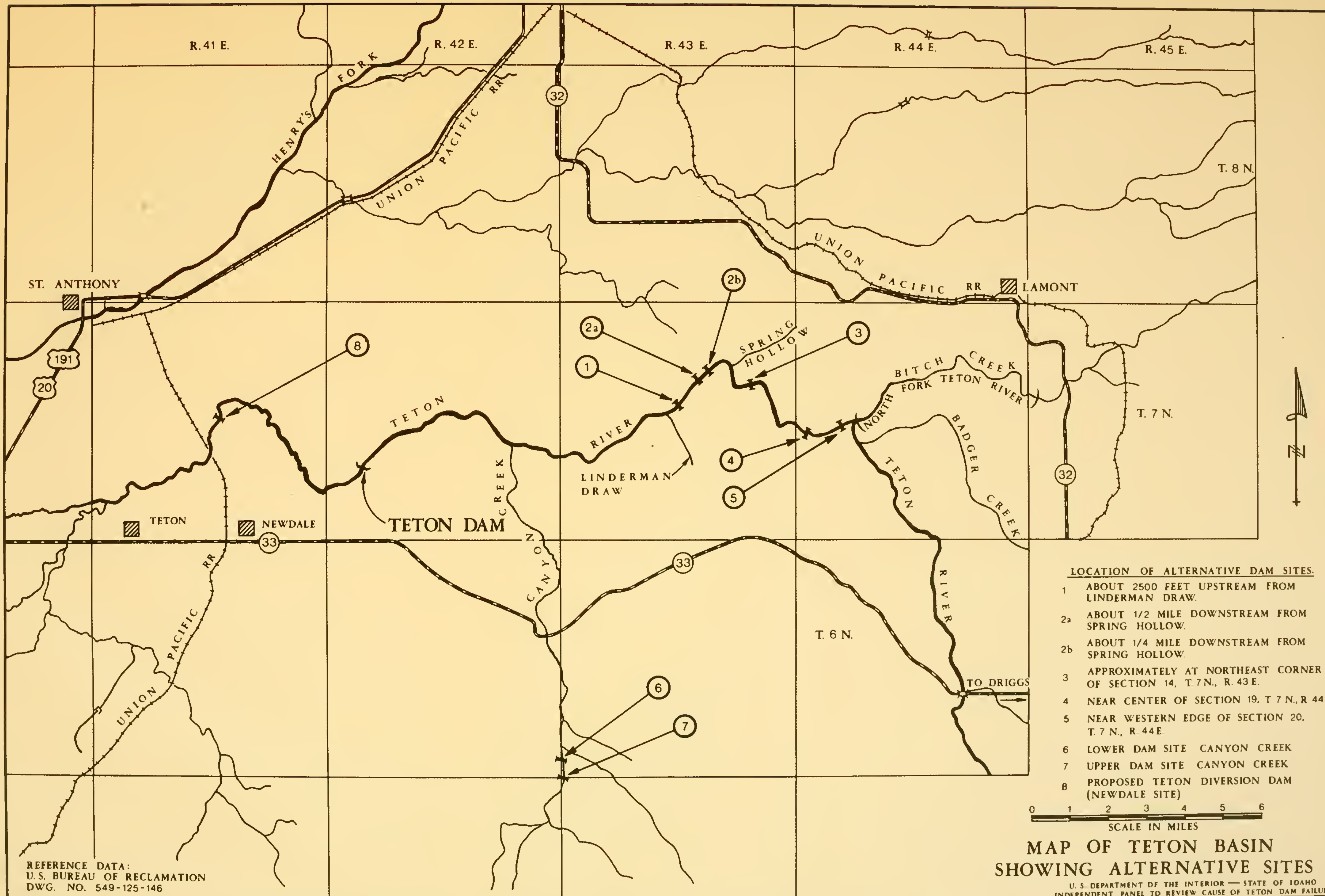


Figure 3.—Location of alternate dam sites

Geology

This section summarizes the geology of Teton damsite. Detailed geologic information is presented in Appendix B.

Regional Geologic Setting.–The Lower Teton Division of the Teton Basin Project is located in and adjacent to the eastern Snake River Plain, a 50-mile-wide, volcanic-filled depression that was formed by downwarping and downfaulting in late Cenozoic time. Volcanism has been concurrent with the tectonic subsidence, so that the older volcanic rocks are now exposed primarily along the margins of the plain and dip gently beneath younger basalt lava flows that form the plain itself. The eastern Snake River Plain is bounded on the northwest and southeast by mountainous terrain of typical basin-range structure that formed concurrently with the plain. Teton Dam and Reservoir are located in the steep-walled canyon that the Teton River has incised into a volcanic upland near the eastern margin of the plain.

A set of well-defined, northwest-trending faults has been mapped in and near the upper canyon of the Teton River to within about 8 miles east of Teton Dam. Other well-defined, northwest-trending faults have been mapped to within about 11 miles southwest of the dam. Some inferred faults with northwest trend are to be located 3 to 4 miles northeast of the dam.

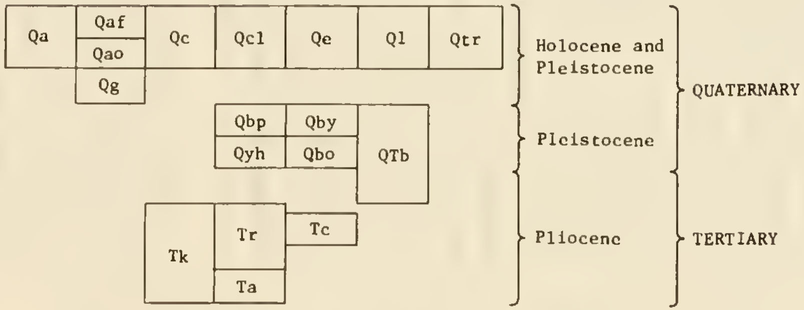
Northeast-trending faults in the Snake River Plain and its margins probably developed during subsidence and crustal extension of the plain. Well-defined faults with this trend have been mapped no closer to the dam than about 7 miles east-northeast and about 10 miles south. Northeast-trending lineaments located closer to the dam are conspicuous on aerial photographs and large-scale topographic maps of the area, but none has been confirmed as being of fault origin. No known active faults occur at or near the damsite. The geology of the region that includes the damsite is shown on Figures 4 and 5.

Seismicity.–Seismicity believed to be associated with movement along prominent faults is characteristic of the mountain ranges bordering the northern, eastern, and southern sides of the eastern Snake River Plain. Several earthquakes with maximum Modified Mercalli intensities of VIII have been experienced in the region. For this reason, southeastern Idaho has been included in U.S. Seismic Risk Zone 3, as shown on Figure 6. However, in the area of the eastern Snake River Plain, the level of locally generated historical seismicity is low. This is determined from the historical record (see Fig. 7) and, since June 1974, from a cooperative Bureau of Reclamation-Geological Survey program. This program was to study the seismicity of the Teton Dam and Reservoir area, particularly to investigate possible seismic effects of reservoir filling and evidence of fault activity within about a 25-mile radius of the damsite.

The cooperative program involved the installation of a monitoring network of three seismic stations located about 18 miles north, east, and southeast of Teton Dam. The seismic station locations are shown on Figure 8. In the 2 years of operation of the network up to the time of failure of the dam, no seismic activity of Richter magnitude 2.2 M_L or greater was observed within 18 miles of the dam, and all events within 12 miles of the dam were caused by blasting. In addition, during the 2 months prior to failure of the dam, no seismic activity was observed within an 18-mile radius of the dam, with the exception of identified blasts. No increase in seismic activity near the dam was recorded while the reservoir was being filled. For at least 4-1/2 hours, beginning within 1 minute of 11:47 a.m., m.d.t., June 5, 1976, the seismic monitoring network recorded ground motion generated by the breakup of the dam and release of the reservoir water. These observations demonstrate that the failure of Teton Dam was not the result of seismic activity.

General Geology of Damsite.–The rock that forms the canyon walls throughout the damsite is a rhyolite welded ash-flow tuff, the middle member of the Huckleberry Ridge Tuff. This welded tuff is extensively exposed in the canyon walls, partly as rock ledges, but over large areas of these slopes it is obscured by a cover of slopewash. Near the axis of the dam, the

CORRELATION OF MAP UNITS



CONTACT--Dashed where approximately located; dotted where it separates areas of detailed mapping from reconnaissance mapping of Paleozoic rocks

NORMAL FAULT--Dashed where approximately located; dotted where concealed; queried where questionable. Bar and ball on downthrown side

STRIKE AND DIP OF BEDS

BASALTIC ERUPTIVE VENT

CLOSED DEPRESSION

DESCRIPTION OF MAP UNITS

SURFICIAL DEPOSITS (HOLOCENE AND PLEISTOCENE)--Age ranges of individual units overlap

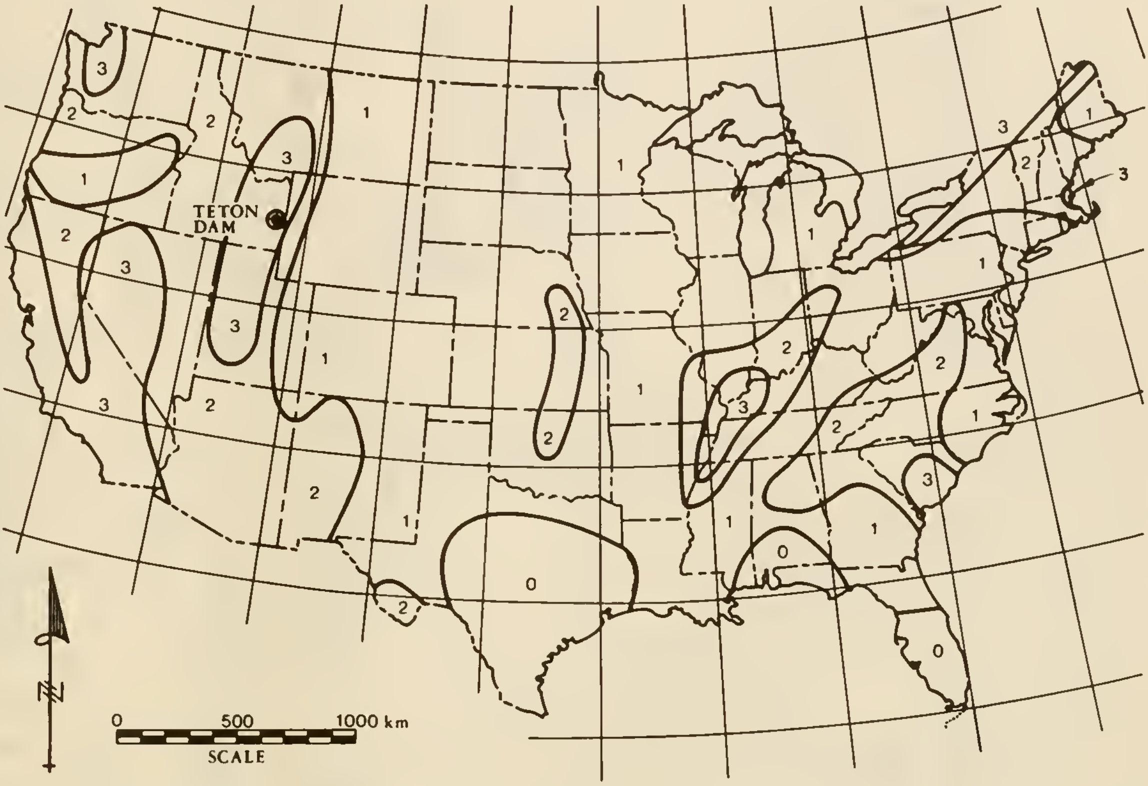
- Qa Alluvial deposits undivided--Unconsolidated gravel, sand, and silt of flood plains, stream terraces, and alluvial fans
- Qaf Alluvial fan deposits--Unconsolidated gravel, sand, and silt
- Qao Older alluvial deposits--Unconsolidated gravel, sand, and silt of older terraces and fans
- Qc Colluvium--Unconsolidated poorly sorted debris deposited on slopes by mass movements and rockfalls
- Qcl Landslide deposits--Unconsolidated poorly sorted slide deposits generally with hummocky bouldery surfaces
- Qe Dune-sand deposits--Unconsolidated well-sorted sand
- Ql Loess--Unconsolidated well-sorted windblown silt
- Qtr Travertine--Light-gray calcareous hot-spring deposits
- Qg Glacial till--Unconsolidated poorly sorted glacial till generally lacking distinct morainal form; mostly covered by loess

VOLCANIC ROCKS AND ASSOCIATED SEDIMENTS

- Qbp Basaltic pyroclastic deposits (Pleistocene)--Unconsolidated to poorly consolidated deposits of basaltic cinders, scoria, ash, and bombs, in part palagonitic, near basalt vents. Includes some welded spatter
- Qby Snake River Group (Pleistocene)--Lava flows of basalt, mostly pahoehoe
- Qyh Huckleberry Ridge Tuff (Pleistocene)--Rhyolite welded ash-flow tuff. Extensively covered by loess in Rexburg Bench area
- Qbo Older basalt (Pleistocene)--Lava flows of basalt; minor amounts of interlayered coarse- to fine-grained sediments and basaltic pyroclastic deposits. Extensively covered by loess in Rexburg Bench area
- QTb Basalt undivided (Pleistocene and Pliocene)--Lava flows of Snake River Group, older basalt, and basaltic pyroclastic deposits in southeast quarter of map area
- Tk Kirkham Hollow Volcanics (Pliocene)--Mainly rhyolitic welded tuffs, lava flows, and nonwelded tuffs of several ages in southeast quarter of map area
- Tr Rhyolite (Pliocene)--Mainly rhyolitic welded tuffs, lava flows, and nonwelded tuffs of several ages
- Tc Conglomerate (Pliocene)--Moderately well consolidated conglomerate
- Ta Andesite (Pliocene)--Lava flows of platy-jointed andesite

Figure 4.—Description and correlation of map units and symbols for Figure 5.

Figure 5.—Geologic map of the region of Rexburg Bench and the canyon of Teton River. (From Prostka, H. J., and Hackman, R. J., 1974, Preliminary geologic map of the NW 1/4 Driggs 1° by 2° quadrangle, southeastern Idaho: U.S. Geol. Survey Open-File Report 74-105.)



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.

*Modified Mercalli Intensity Scale of 1931.

SEISMIC RISK MAP
OF THE UNITED STATES

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

Figure 6.—Seismic risk map for the United States.

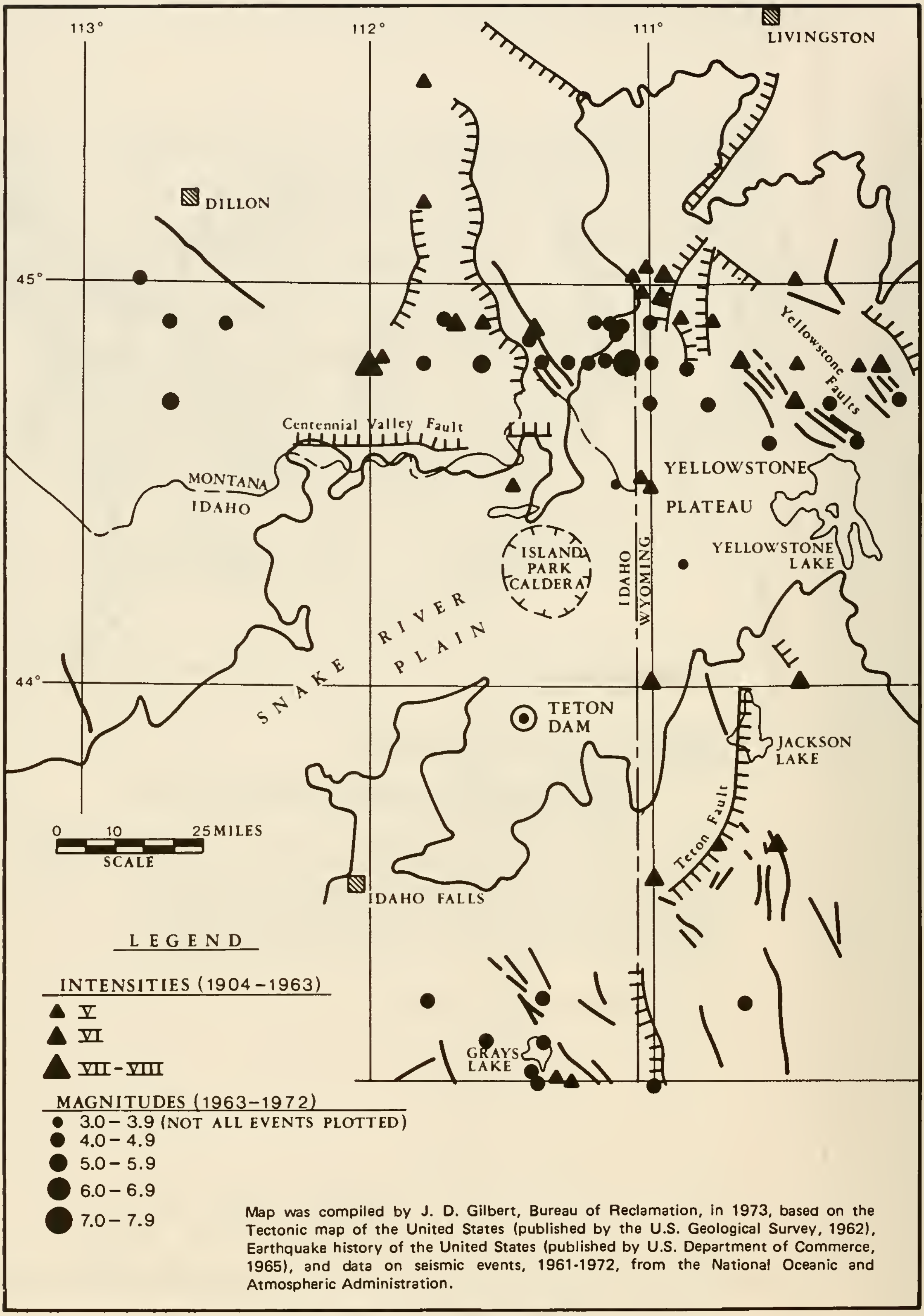


Figure 7.—Earthquake occurrence near the eastern Snake River Plain.

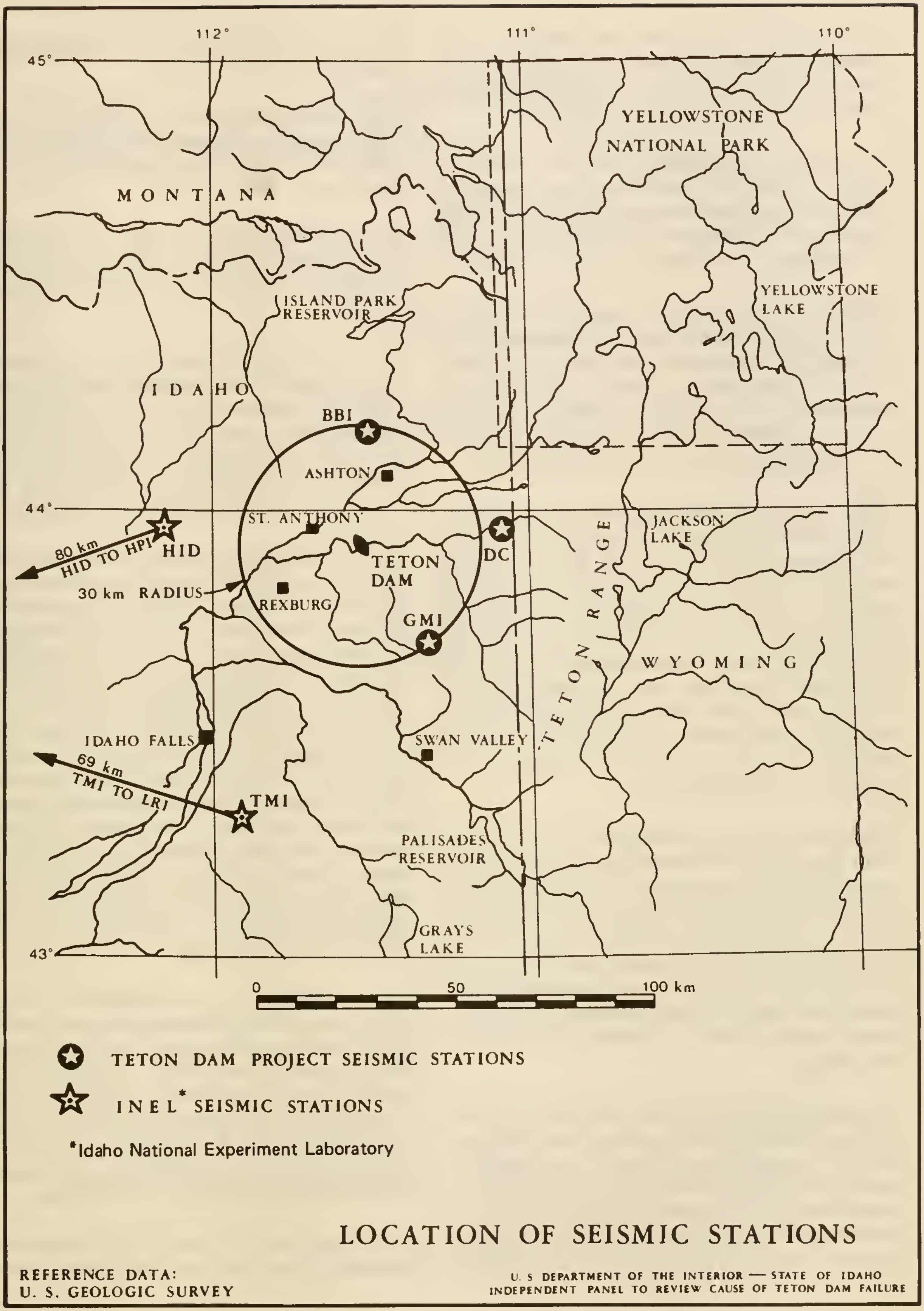


Figure 8.—Location of seismic stations.

thickness of the welded tuff ranges from a minimum of about 50 feet to a maximum of at least 575 feet. The relationship of welded tuff to other formations is shown on Figure 9.

The welded tuff generally exhibits foliation, which is a result of processes of accumulation, compaction, and welding of the ash-flow deposits. The foliation is generally flat-lying or gently dipping, but locally it is steeply inclined.

The welded tuff at the damsite and along the canyon of the Teton River is characterized by the presence of prominent and abundant joints. Most of the joints probably resulted from tensional stresses caused by cooling of the rock after it solidified. The joint system consists of intersecting high-angle and low-angle joints.

Most of the high-angle joints are nearly vertical and strike northwest. The major set, which strikes about N. 25° W. to N. 30° W., is well developed on both abutments and in the rock intersected by both outlet tunnels. A second set strikes about N. 60° W. to N. 70° W.; it is well developed in the lower upstream part of the right abutment, throughout the river outlet works tunnel, and in the downstream part of the auxiliary outlet works tunnel. A minor set of northeast-trending, high-angle joints is also present in the welded tuff.

High-angle joints in the right abutment have been traced for continuous lengths of as much as 200 feet, but most are between 20 and 100 feet in length. Spacing between high-angle joints generally ranges from a few feet to about 10 feet, but locally is from less than 1 foot to as much as 60 feet. The width of most high-angle joints is less than one-half inch, but many joints are as much as several inches wide and some are several feet wide.

Low-angle joints parallel the flat-lying or gently dipping foliation. The lengths of the low-angle joints are generally less than 50 feet. However, several low-angle joints in the upper part of the middle unit of the welded tuff have been traced for about 200 feet, and a jointlike discontinuity between the middle and lower units has been traced for at least 400 feet upstream from the dam centerline.

The spacing between low-angle joints is generally wide except in the upper 70 feet of the welded tuff, where closer spacing results in a platy structure.

Many joints are open; others are partially or wholly filled with clay, silt, silty ash, soil, or rubble, especially near the natural ground surface.

The permeability of the welded tuff, which has been by far the most important geologic problem in the development of the damsite, is due almost entirely to the presence of open joints. The joints are more abundant and open, and rock-mass permeability accordingly is much higher above about El. 5100 than below that elevation.

Underlying the welded tuff are materials of lacustrine, alluvial, and pyroclastic origin, probably of Pliocene Age. These materials generally have been referred to as "sediments" or "lakebed sediments." Information about these materials has come mainly from drill holes, commonly with poor or no core recovery, and to some extent from deep grout holes, and thus is rather fragmentary. Little is known about the distribution and interrelations of the various units, but sand and gravel and variably cemented sandstone and conglomerate are commonly present, and thick claystone and siltstone are present under at least part of the left abutment and channel section. Thin ash-fall tuff and other pyroclastic materials were found below the welded tuff in some core holes.

The contact between the sedimentary materials and the welded tuff is an irregular erosion surface with a local relief of at least 440 feet and some slopes steeper than 30°. The sedimentary materials are at least 390 feet thick, as explored by postfailure drillhole 651-B in the right abutment which bottomed in gravel. The characteristics of, and depth to, the materials underlying these sediments are unknown.

The permeability of most of the sedimentary materials is less than that of the highly fractured

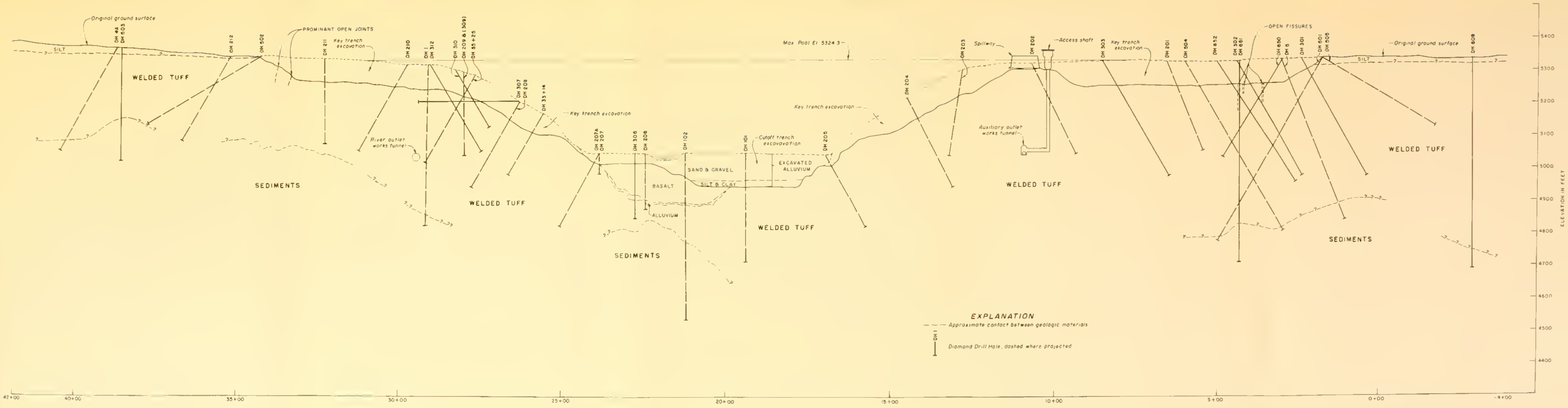


Figure 9—Teton Dam—geologic profile on dam axis

welded tuff. However, the permeability varies considerably with the texture, apparently being very high in some of the gravels and very low in the thick claystone bodies.

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

On the uplands bordering the canyon the welded tuff is overlain by windblown silt, or loess, which ranges in thickness from less than 1 foot near the canyon edge to more than 50 feet. These deposits served as the source of the zone 1 material of the embankment.

The flood plain of the Teton River is underlain by alluvial deposits having a maximum thickness of about 100 feet at the damsite. This alluvium consists of an upper unit about 80 feet thick composed of sand and gravel with some cobbles and boulders, and a lower unit about 20 feet thick composed of silt and clay.

Large areas of the canyon walls are obscured by a blanket of slopewash generally less than 10 feet thick. The slopewash consists of a mixture of silty soil and fragments of welded tuff. A small area of landslide debris occupies a topographic recess just upstream from the river outlet works intake tower.

Groundwater.–In general, the regional groundwater table in the vicinity of Teton Dam and Reservoir lies from 200 to 500 feet below the ground surface. It shows an annual low about May-June and an annual high in September-November. The regional slope of the groundwater gradient is westward, with many irregularities.

In the immediate vicinity of Teton Dam and Reservoir, there is also a perched water table which, prior to reservoir filling, was 100 feet or more above the regional water table. This perched water table was somewhat above river level in the area immediately southeast of the river and as much as 50 feet below river level northwest of the river.

Design

General.–Teton Dam was designed by the Division of Design, Bureau of Reclamation E&R Center, Denver. Figures 10 and 11 show the general plan and sections of the dam. The dam was designed as a zoned earthfill embankment with crest El. 5332 and with a maximum height of 305 feet above the valley floor and 405 feet above the lowest point excavated in the foundation. The crest length was about 3,100 feet, including the spillway. Appurtenant features are: (1) a three-gated, chute-type spillway on the right abutment, Figure 12, (2) an auxiliary outlet works and access shaft in the right abutment, Figure 13, and (3) a 16,000-kW power generating and pumping plant at the base of the left abutment, (4) a river outlet works with gate chamber and access shaft, Figure 14, and (5) the Enterprise-East Teton Feeder Pipeline and Canal Outlet Works Control Structure. Because the IRG has determined that none of the appurtenant features were involved in the failure, they will not be discussed further in this report.

Embankment.–The embankment is a zoned earthfill with a thick central core. Figure 10 presents cross sections that show the dimensions of the embankment and the relative positions of zones 1 through 5. The location of each zone is outlined briefly below:

Zone 1–Central core

Zone 2–Upstream and downstream material adjacent to zone 1. Blanket under zone 3 in river valley and on abutments

Zone 3–Random fill downstream of zone 2

Zone 4–Upstream cofferdam, later incorporated into upstream toe of dam

Zone 5–Protective exterior upstream and downstream rockfill

Zone 1 material was obtained from the deposits of windblown silts covering most of the immediate area outside of the Teton River Canyon. These silts of low plasticity were compacted to at least an average density of 98 percent of Bureau of Reclamation Standard Proctor Density, at an average moisture content between 0.5 and 1.5 percent dry of optimum moisture content. The permeability of zone 1 core material averaged 0.5 feet per year.

Zone 2 was composed of selected sands and gravels from the Teton River flood plain. Zone 2 was to be compacted to a relative density of at least 70 percent.

Further details of the materials found in the embankment are presented in Appendix D.

Foundation Treatment.–Foundation treatment consisted of four major components:

- a cutoff trench through the flood plain alluvium
- a key trench excavated into the rock abutments above El. 5100 except under the spillway
- a grout curtain that extended the full length of the dam
- rock surface treatment on the abutments under zone 1 material

The four components of foundation treatment are shown on Figures 10 and 11.

The cutoff trench was to be excavated through alluvial material to a maximum depth of 100 feet so that zone 1 material could be placed upon a rock foundation. The design width of the cutoff trench in this area was 30 feet at El. 4920.

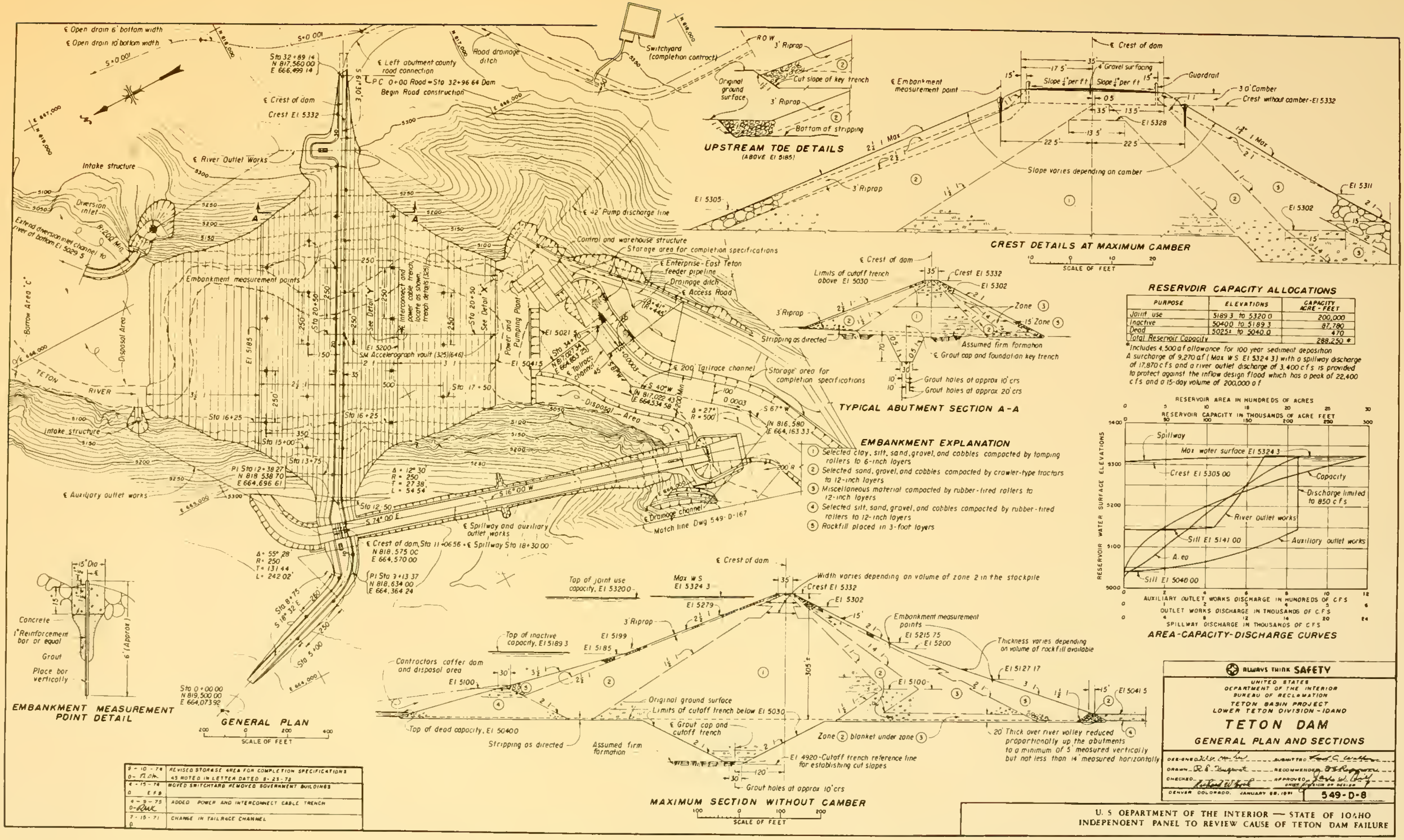
The use of the key trench was a direct result of a pilot grouting program which showed that above El. 5100, the upper 70 feet of rock was so permeable that grouting was not practical. The key trenches extended from El. 5100 on each abutment to the outer extremities of the embankment, except under the spillway. They were excavated to a depth of 70 feet with bottom widths of 30 feet.

As shown on Figure 11, a grout curtain was to be constructed under the entire length of the dam. The curtain was to be constructed with one to three rows of grout holes generally extending as deep as 260 feet below the bottom of the cutoff and key trenches.

Above El. 5030, excavation was to be extended the full width of zone 1, to cut through thin slopewash and rubble to the firm in-place foundation rock. Within this area, provision was made for special treatment of large surface joints. Further design details for foundation treatment are presented in Appendixes C and D.

Construction

Construction Sequence.–Construction of Teton Dam began in February 1972 with clearing and construction of access roads. The river outlet tunnel was started in March 1972. During tunnel construction, the river was diverted into a channel constructed along the right abutment. During the 1972 construction season, the original river channel adjacent to the left abutment was cleared and materials from required excavations were placed in the cofferdam and at selected locations outside the cutoff trench. Also during 1972, excavation was in progress in the key trenches and in the spillway.



PURPOSE	ELEVATIONS	CAPACITY ACRE-FEET
Joint use	5189 3 to 5320 0	200,000
Inactive	5040 0 to 5189 3	87,780
Dead	5025± to 5040.0	470
Total Reservoir Capacity		288,250 *

Figure 10.—Teton Dam—general plan and sections.

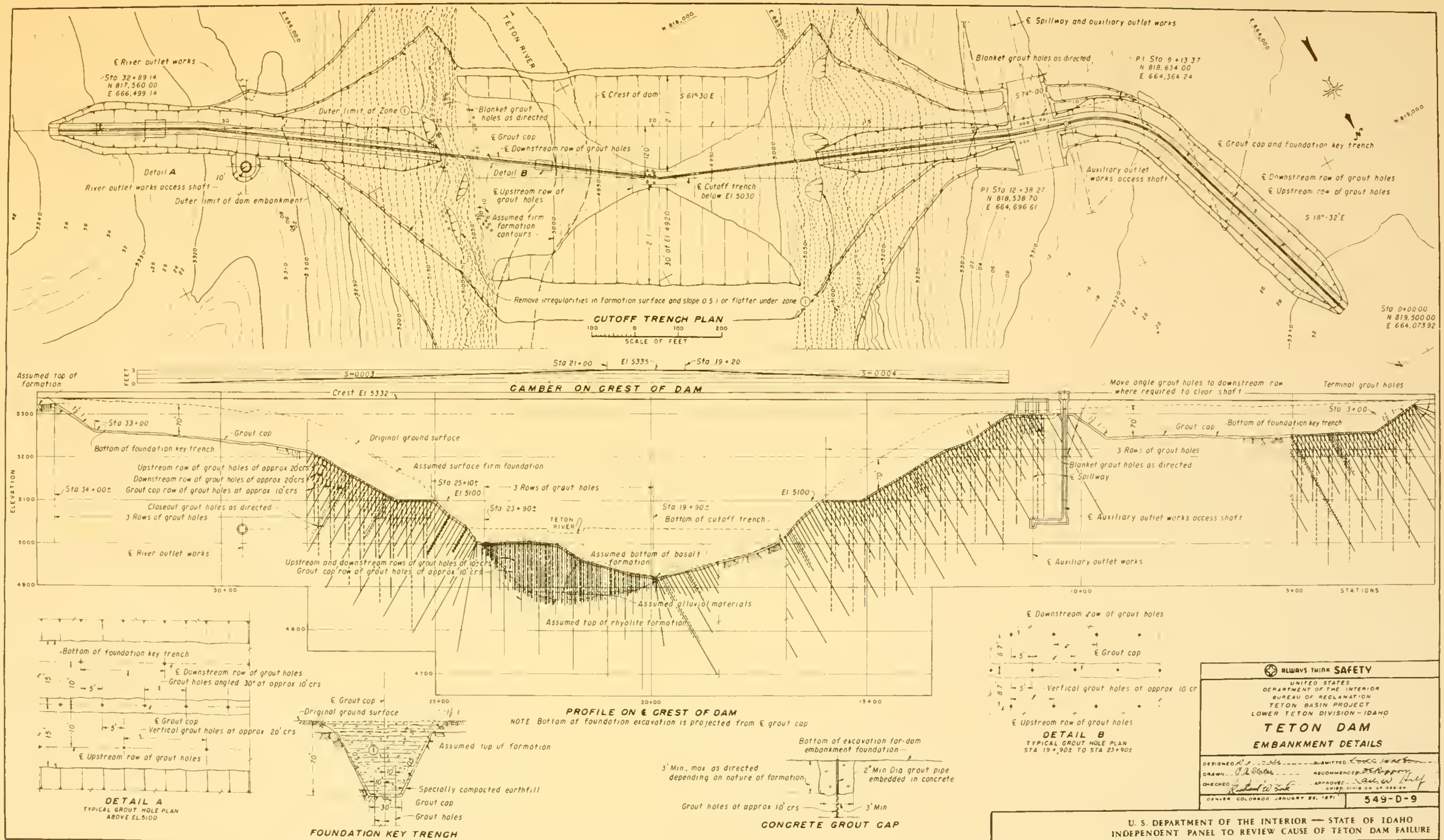


Figure 11.—Teton Dam—embankment details.

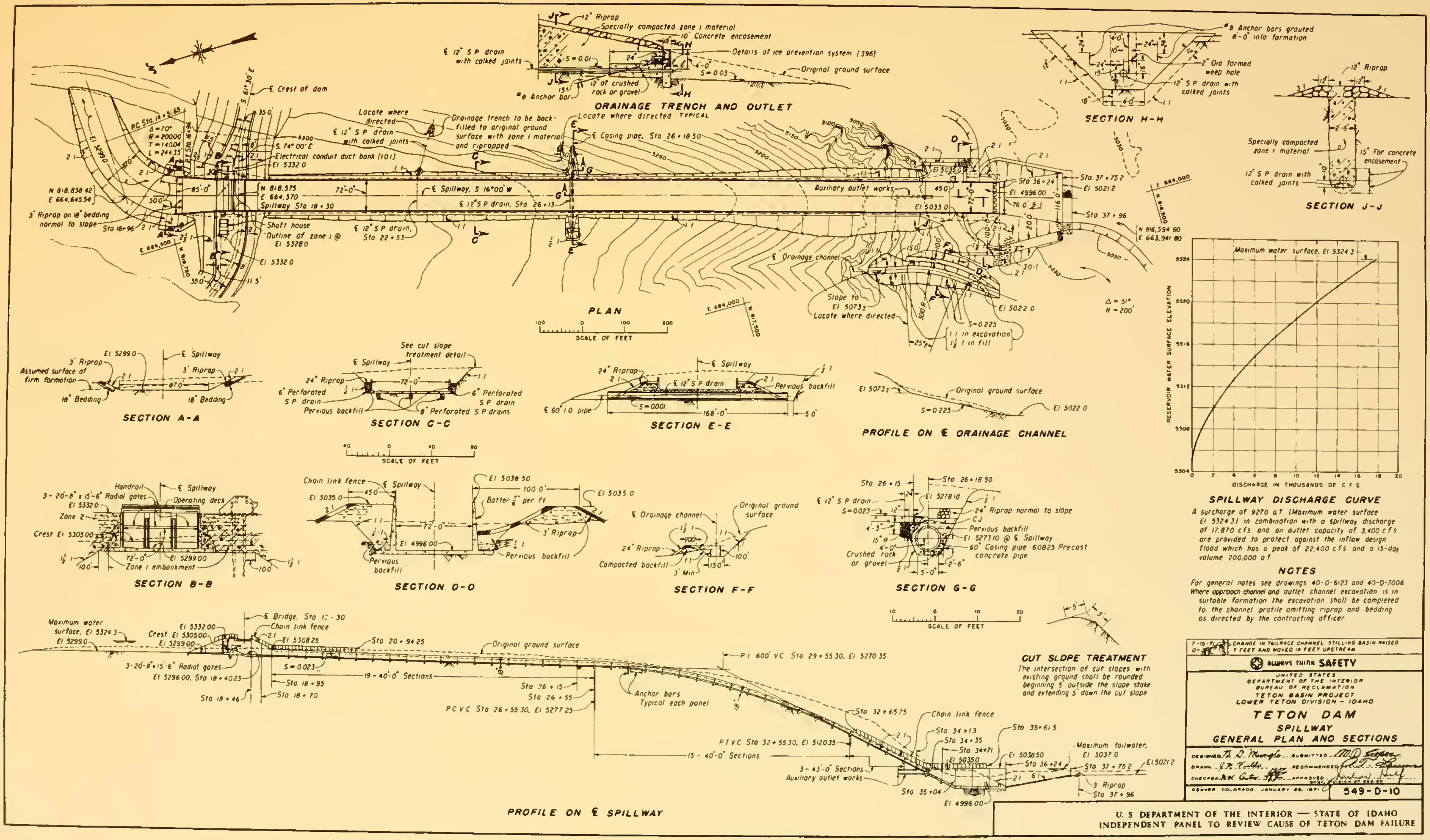


Figure 12.—Teton Dam—spillway.

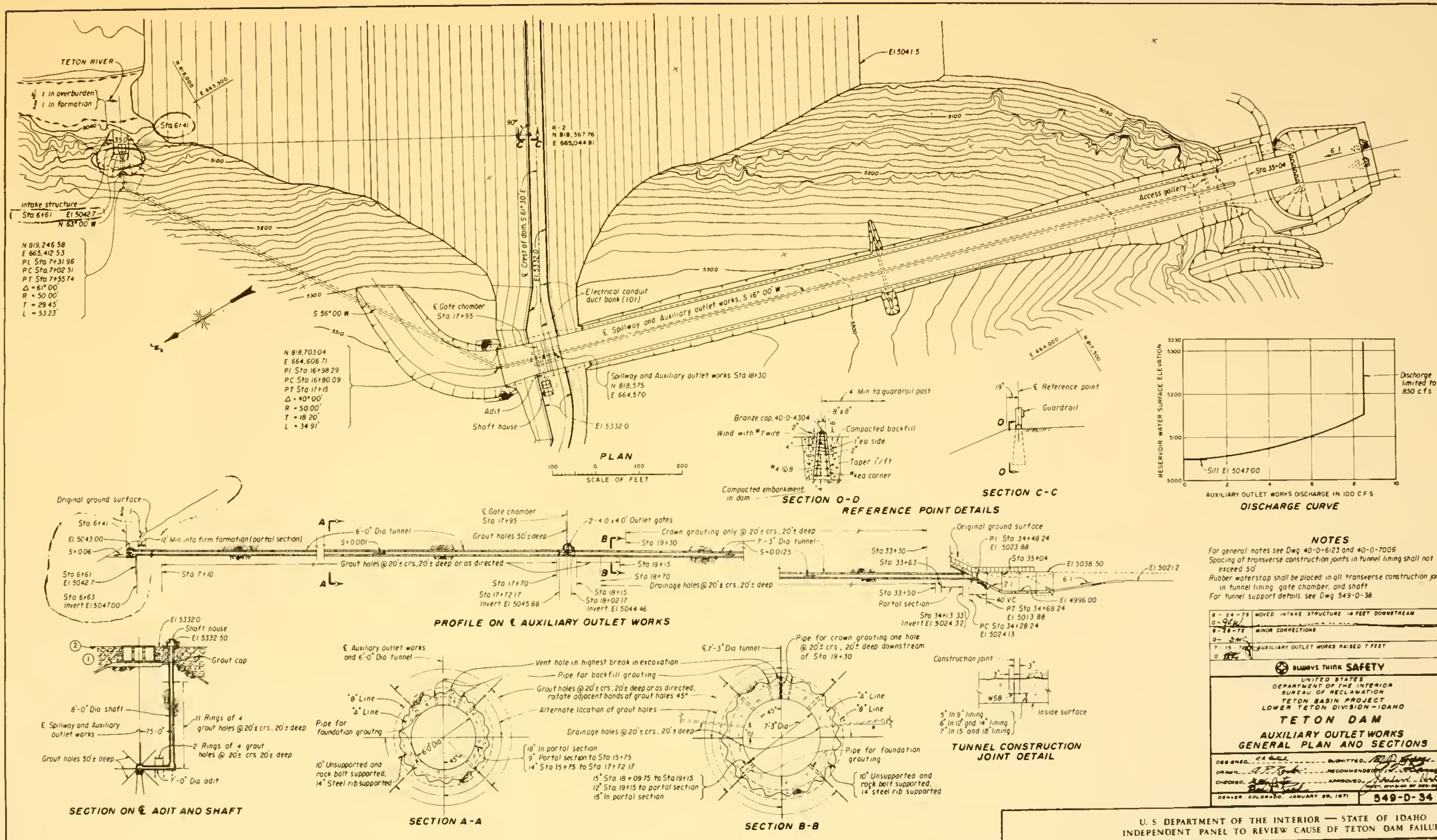


Figure 13.—Teton Dam—auxiliary outlet works.

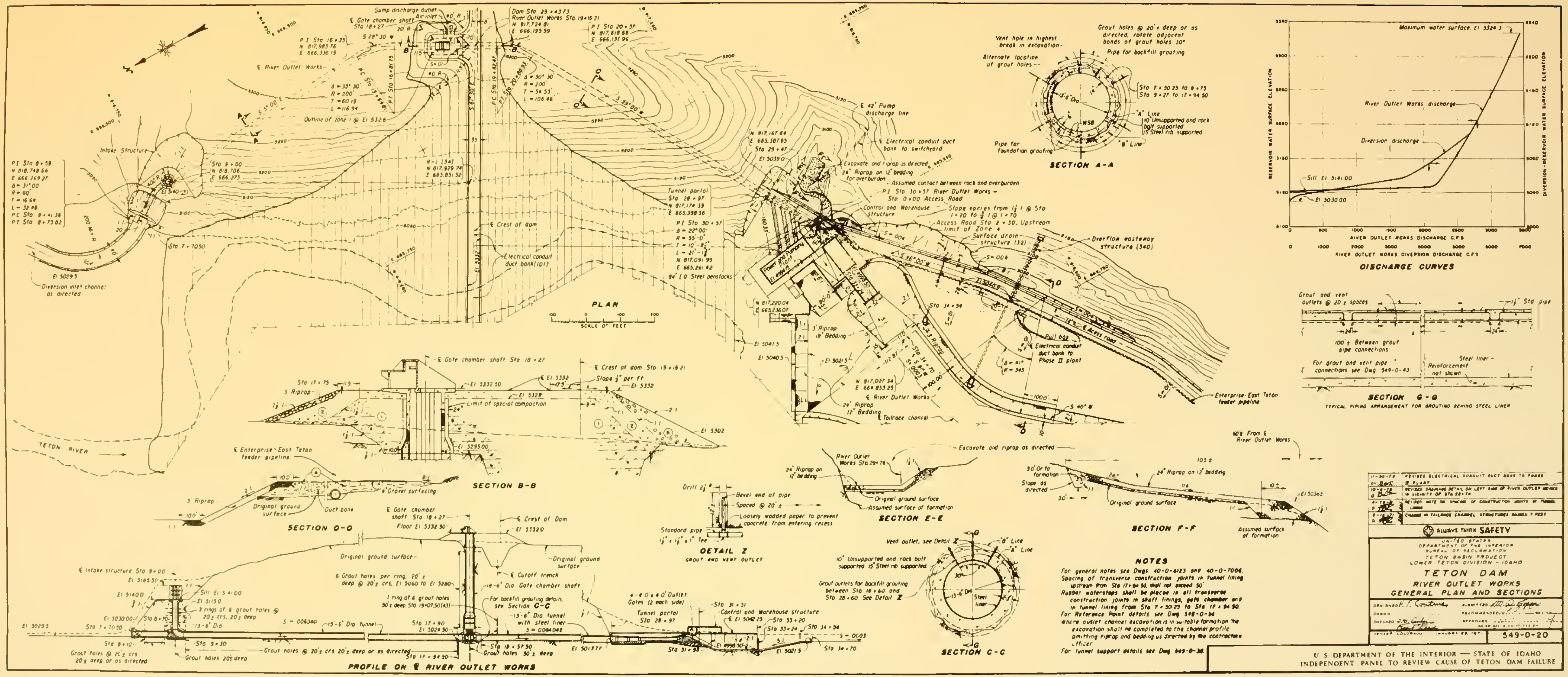


Figure 14.—Teton Dam—river outlet works

Diversion through the river outlet works began on June 8, 1973. During the 1973 construction season, the cutoff trench was completed, as was the grouting in the lower part of the foundation. In October 1973, the first zone 1 material was placed in the bottom of the cutoff trench. All embankment placement stopped in November because of cold weather. Embankment placement resumed in May 1974, and by the winter shutdown in November, all zones were completed to about El. 5147. In May 1975, embankment placement resumed and the dam was topped-out in November. During the 1975 construction season, approximately 5 million cubic yards of embankment material were placed, averaging 700,000 cubic yards per month.

Embankment Control.–Bureau inspectors were present at all times during embankment construction to assure that the placement specifications were met. Field and laboratory tests were made at frequent intervals on samples taken from the embankment.

Zone 1 material was compacted with 12 passes of standard tamping rollers to obtain 6-inch compacted layers. The zone 1 material was placed at an average moisture content of 1.1 percent dry of the optimum moisture content and at an average dry density of 98.3 percent of the Bureau's Proctor maximum dry density. Compaction was monitored using the Bureau of Reclamation Rapid Compaction Control Method. One in-place density test was performed for approximately each 1,900 cubic yards of zone 1 fill placed.

Specially compacted zone 1 earthfill was placed in areas where use of the specified tamping roller was impracticable. These areas included steep or irregular abutments, rough or irregular embankment foundations, and the sides of the key trenches. The material was placed at an average of 0.5 percent dry of optimum moisture content and compacted with pneumatic rams, plate tampers, and rubber-tired equipment. The specially compacted earthfill was compacted to an average dry density of 97.3 percent of the Bureau's Proctor maximum dry density. The density control monitoring was by the Rapid Compaction Control Method. One in-place density test was performed for approximately each 88 cubic yards of acceptable specially compacted earthfill placed.

Zone 2 material was compacted by heavy crawler-type tractors and vibratory rollers to obtain 12-inch compacted layers. Compaction control criteria were established with relative density tests. The average relative density obtained for zone 2 material was 94 percent.

Zone 3 material was placed in 12-inch compacted layers, compacted with 50-ton pneumatic-tired rollers, tamping rollers, loaded trucks, or crawler dozers. The average moisture content for fine-grained zone 3 material was 1.5 percent dry of optimum moisture content. Compaction control was based on Proctor density tests or relative density tests, depending on the material being placed. For fine-grained zone 3 material, the ratio of the average fill dry density to the Proctor maximum dry density was 97.5 percent. One in-place density test was performed for approximately each 3,700 cubic yards of zone 3 material.

Zone 4 was compacted similarly to zone 2 or zone 3, depending upon the nature of the material.

Zone 5 consisted of rock fragments placed in 3-foot layers, compacted by hauling equipment.

Further details related to embankment construction are contained in Appendix D.

Foundation Treatment.–The dam foundation treatment consisted basically of four items (see Figs. 10 and 11):

- 70-foot-deep key trenches in both the right and left abutments above El. 5100, except under the spillway
- A positive cutoff trench along the valley floor and in the abutments below El. 5100 which, as constructed, had a minimum bottom width of 80 feet
- A continuous grout curtain along the entire length of dam foundation

- Rock surface treatment under portions of zone 1

The entire embankment foundation was stripped of all material considered unsuitable.

Under zone 1, the rock surface was cleaned using high-pressure air and water jets. Some open joints and cracks in the bottom of the key trenches and the cutoff trench were treated by installing pipes and grouting. Between El. 5100 and 5205 gravity filling with a grout slurry or filling with specially compacted zone 1 material was used to treat open cracks on the key trench side slopes and adjacent abutment rock surfaces. Complete discussions of rock surface treatment are included in Appendixes C and D.

The grout curtain consisted of three lines of grout holes, except in certain reaches below El. 5100 where the rock was less permeable. A single line of grout holes was used between Stas. 16+00 and 19+90 and between Stas. 23+90 and 25+10.

Where three lines of grout holes were used, they were 10 feet apart. The center line was drilled through a 3-foot-wide by 4-foot-deep grout cap. Maximum depth of holes in all three lines was 260 feet, with the exception of the spillway area where the holes were deepened to 310 feet. Holes in the downstream line were vertical, while the upstream and center-line holes were angled 30° from the vertical into the abutment. The downstream line was grouted first, then the upstream line, and finally the center line. In addition to the curtain grouting, blanket grouting was performed in the spillway crest foundation and to treat open cracks as described above.

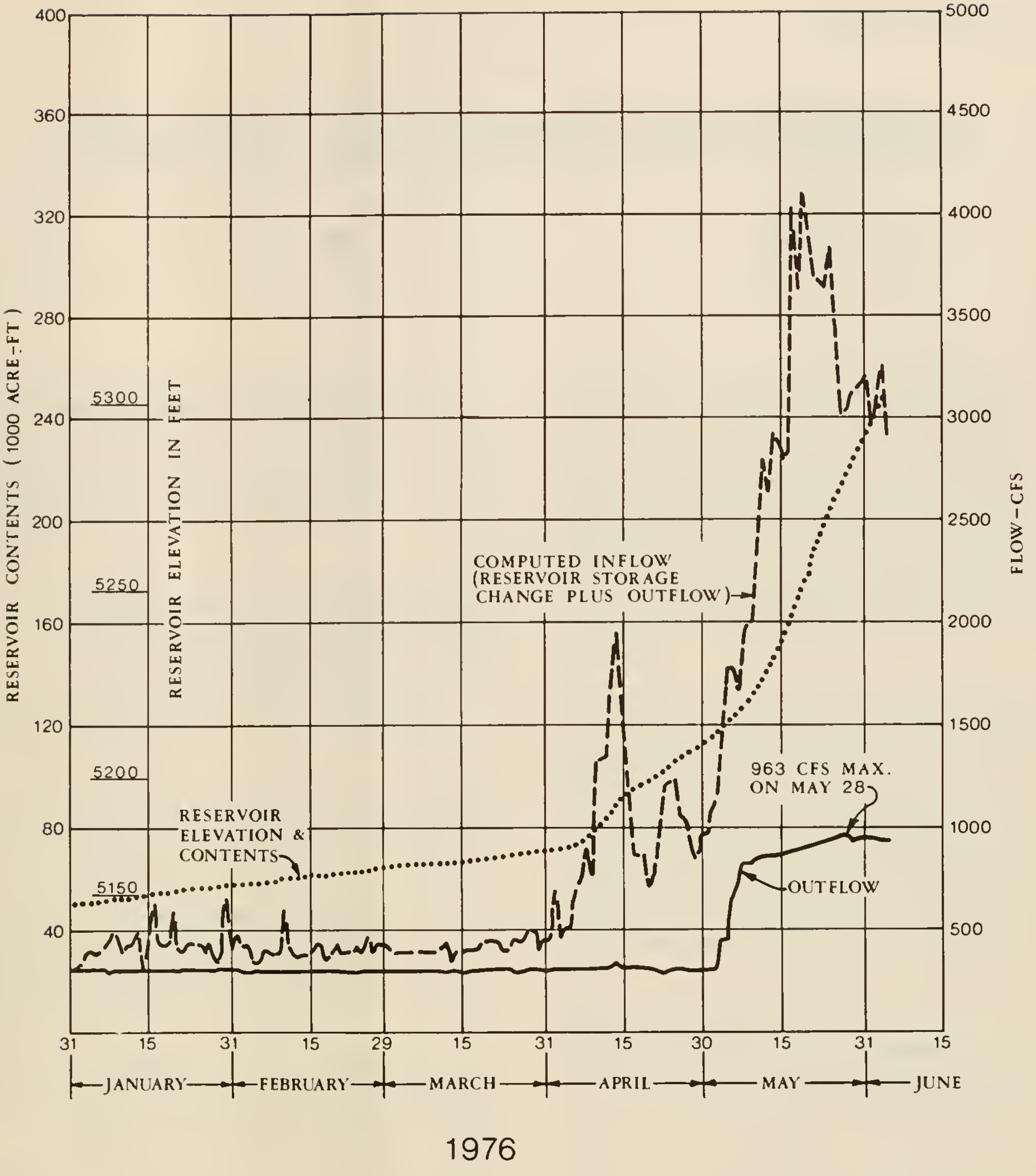
During the grouting operation, 496,515 cubic feet of cement, 82,364 cubic feet of sand, 132,000 pounds of bentonite, and 418,000 pounds of calcium chloride were injected into 118,179 lineal feet of drilled holes. A review of the records and reports of the grouting operation indicates that the work was performed in a methodical, workmanlike manner and in accordance with the plans and specifications. A complete discussion of the grouting operation can be found in Appendix C.

Reservoir Filling and Releases

Reservoir filling began on October 3, 1975, when the river outlet works were closed. During the period from October 3, 1975 to May 3, 1976, releases through the auxiliary outlet works were limited to required downstream flows, 300 cubic feet per second (cfs), and all inflows in excess of 300 cfs were stored in the reservoir. On May 3, 1976, the reservoir depth was 185 feet. From May 4 through May 11, 1976, the flow through the auxiliary outlet works was gradually increased to 850 cfs. From May 11, 1976 until the time of the failure of the dam, the auxiliary outlet works was operated at or above its design capacity of 850 cfs.

The design considerations required that above El. 5200 the reservoir was not to be filled faster than 1 foot per day. This arbitrary criterion has been used for many years by the Bureau for initial filling of reservoirs. The criterion allows the initial rate to be exceeded when the dam performs satisfactorily.

On March 3, 1976, the Construction Engineer requested of the Denver Office that the filling rate restriction be relaxed. At this time, the reservoir was 135 feet deep or at El. 5170. On March 23, 1976, the Construction Engineer was given permission by the Denver Office to increase the filling rate to 2 feet per day to accommodate the large reservoir inflows. The groundwater table adjacent to the reservoir appeared to be developing normally, and no springs had developed downstream of the dam. The normal groundwater table development and lack of spring development continued through May 13, 1976, and the decision was made by the Denver Office to fill the reservoir to the spillway crest, resulting in an average filling rate of approximately 3 feet per day with a maximum rate of 4.3. The water level had only reached the spillway approach channel (El. 5301.7) when failure occurred. Figure 15 graphically shows the reservoir filling schedule.



TETON RESERVOIR INFLOW, OUTFLOW,
AND CONTENTS, JANUARY 1 TO JUNE 5, 1976

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

Figure 15.—Teton reservoir inflow, outflow, and contents.

Chronology and Consequences of Failure

Chronology of Failure Events

Introduction.–The Teton Dam failure sequence has been reconstructed by the IRG from:

- Interviews of 41 persons and sworn testimony from 36 of them. Interviews were conducted by special agents, Office of Audit and Investigation, Office of the Secretary. Eyewitness accounts of the failure are contained in Appendix F

- Photographs taken by Bureau of Reclamation project personnel, contractor employees, and on-site visitors

- 8- and 16-mm movies taken by Bureau of Reclamation project personnel

- Verbal discussions with project personnel who witnessed the failure

- Statements given by Messrs. Robison, Aberle, Ringel, Parks, Isaacson, and Rogers before staff members of the Independent Panel on October 29, 1976

The exact timing of the events, excepting the final collapse of the dam crest, are not known but sufficient information is available to estimate approximate times. The failure chronology as determined by the IRG is substantially the same as that reported by the Independent Panel.

Events and Observations Prior to June 5, 1976.–On October 3, 1975, reservoir filling began; the reservoir was then at El. 5060. Filling continued at a relatively slow rate until April 10, 1976, when the reservoir level reached El. 5180. No problems of any type with the embankment were reported, nor were there any seeps or leakage reported. From April 10, 1976 until June 5, 1976, the reservoir filled at a faster rate because of high runoff of snowmelt. Prior to June 3, 1976, no problems of any type with the embankment were reported, and no seeps were reported.

On June 3, 1976, project personnel inspecting the right abutment found two small seeps located 1,300 and 1,500 feet downstream from the toe of the dam. These seeps were flowing clear water at approximately 60 and 40 gallons per minute (gpm), respectively. Photographs of these seeps are shown in Figures 17 and 18, and their location relative to the dam is shown on Figure 16. The reservoir elevation at this time was 5300.

On June 4, 1976, a small clear seep was found flowing approximately 20 gpm from the right abutment 150 to 200 feet downstream from the toe of the dam. This small seepage was noted by a number of individuals; its location is shown on Figure 16. No photographs of this seep exist.

No additional seepage locations were found as of 9:00 p.m. on June 4. Thus, it is established that as of 9:00 p.m. on June 4, 1976, no serious seepage or leaks were observed on the embankment or from the abutments of Teton Dam. A Bureau employee was at the damsite until 12:30 a.m. on June 5, 1976.

Events and Observations On June 5, 1976.–Among the first persons to arrive at the site on June 5 were Bureau of Reclamation project surveyors. The survey party observed a leak, at about 7:45 a.m., coming from the right abutment at El. 5045 near the toe of the dam. There were also references in the eyewitness reports to a leak higher in the dam at this same time. This leak is presumed by the IRG to have come from the abutment at El. 5200. The survey party immediately reported the leak to their supervisor, Mr. Ringel, who drove to the powerhouse and walked to the right abutment to inspect the leak. At 8:15 a.m. he estimated its flow to be 20 to 30 cfs. Ringel did not photograph the leak.

Contractor personnel (Gibbons and Reed Company) were most probably the first persons to notice water flowing from the abutment at El. 5200. Their first observations were made at approximately 7:00 a.m.

The Project Construction Engineer and Field Engineer were notified at about 8:20 a.m. and both arrived at the damsite at about 9:00 a.m. Robison and Aberle walked down the downstream face of the dam near the right abutment to the El. 5200 embankment-abutment contact. No leakage was observed from the crest of the dam down to El. 5200. About 9:10 a.m., at El. 5200, a slightly turbid leak of 2 cfs was observed exiting from the abutment. The water flowed for a short distance along the contact before entering the embankment's zone 5 rockfill near the contact. No photograph of this leak exists. Waterflow from this leak during the early morning hours of June 5 had evidently washed away finer material overlying the zone 5 rockfill along the entire embankment-abutment contact between El. 5200 and 5041.5. It is possible that this seepage was much larger than 2 cfs which Robison and Aberle observed. At this time, neither individual noted any wet spot on the embankment at El. 5200 nor at any location on the downstream face of the dam.

Robison and Aberle continued down the abutment contact to the lower leak at El. 5045. No other seepage from the abutment or embankment was noted between El. 5200 and 5045. The leak at El. 5045 was examined at approximately 9:30 a.m. and was estimated to be flowing 40 to 50 cfs.[1] Ringel joined then and took photographs of the leak, Figures 19 and 20. The erosion channel along the right embankment-abutment contact is apparent in the upper center portion of Figure 20. The leak at El. 5045 was located near crest centerline Sta. 17+25 and is shown on Figure 16. Robison and Aberle then returned to the Project Office and gave instructions to channelize the leakage that developed at El. 5045.

Between 10:00 a.m. and 10:30 a.m., a wet spot developed on the downstream face of the dam at El. 5200, and about 15 to 20 feet out from the right abutment. The wet spot quickly began to leak at a rate of 10 to 15 cfs, and eroded zone 5 material from the face of the embankment. At about 10:30 a.m., Robison, Aberle, and others noted a loud sound followed by the sound of rapidly running water. Robison comments in a supplemental statement:

> "This leak developed almost instantaneous at about 10:30 a.m. and let loose with a loud roar."

From the moment the embankment leak developed at El. 5200, the volume of water increased very rapidly, as did the erosion of embankment materials. Two dozers were sent to the right abutment at 10:40 a.m. to begin pushing rock into the eroding hole (see Fig. 21). While the dozers were working, Robison walked down the right abutment and looked directly into the eroding opening in the embankment. He observed a tunnel 30 to 40 feet long and roughly 6 feet in diameter extending into the embankment.

From the supplemental statement of Robison:

> "I examined this leak very carefully when the dozers were working on it and noted that the water was coming out of a circular hole through the embankment at right angles to the axis of the dam. The water was flowing

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[1] Although Ringel had estimated 20 to 30 cfs as coming from this location earlier, both estimates are only crude approximations. What does seem clear is that between 8:15 a.m., when Ringel first viewed the leak and 9:30 a.m., when Robison and Aberle viewed the leak no massive increase in flow had occurred.

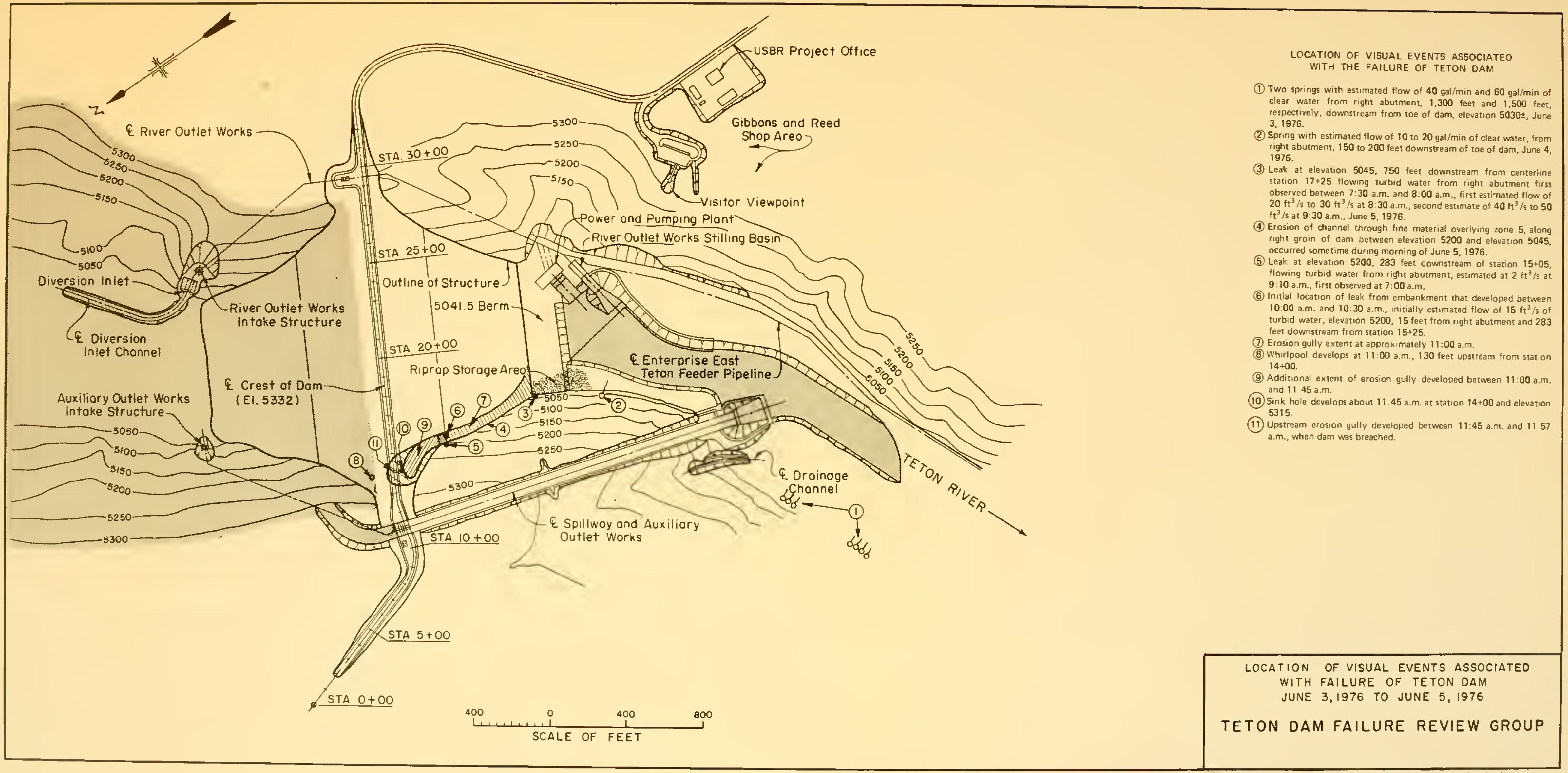


Figure 16.—Teton Oam—location of visual events associated with the failure.

Figure 17.—North canyon wall about 1300 feet downstream from Teton Dam. Clear water from several small seeps flowing about 60 gpm, June 3, 1976.

Figure 18.—North canyon wall about 1500 feet downstream from Teton Dam. Clear water flowing from rhyolite about 40 gpm, June 3, 1976.

Figure 19.—Muddy flow about El. 5045 at right downstream toe estimated 20 to 30 cfs, 8:30 a.m., June 5, 1976.

Figure 20.—Closeup of leak shown on Figure 19.

Figure 21.—Closeup of leak at El. 5200 near right abutment. About 10:30 a.m., June 5, 1976.

extremely muddy and exiting from the hole in the embankment about 15 to 20 feet from the abutment."

The dozers had worked 20 to 30 minutes when the one closest to the eroding hole could not attain sufficient traction to back away from the edge. The second dozer hooked onto the first and a futile effort was made to pull it back. At 11:30 a.m., the two dozers slid into the opening and were washed downstream.

At approximately 11:00 a.m., a whirlpool formed in the reservoir 15 to 20 feet upstream from the face of the dam near Sta. 13+75, approximately 150 feet from the right abutment-reservoir surface contact. The whirlpool diameter rapidly began to expand. No photographs of the whirlpool were taken. Efforts were made to push rock into the whirlpool with no apparent effect.

Between 11:40 a.m. and 11:50 a.m., a sinkhole located near Sta. 13+75 and El. 5315 developed in the downstream face of the embankment.

The embankment crest collapsed at 11:55 a.m. and the dam was breached at 11:57 a.m.

The erosion of the embankment subsequent to 11:30 a.m., and indeed the entire failure sequence, occurred with great rapidity. It took only 5 hours from the time of the first observed seepage in the immediate proximity of the dam until the dam failed. From the time at which the dam was last observed to have no visible leakage, 9:00 p.m. the previous night, only 15 hours were required to breach the dam.

The failure sequence is shown in Figures 22 to 28. It should be noted from the postfailure photographs that most of the significant flows of water from the right abutment were upstream of the grout curtain and key trench.

The main events of the failure sequence are summarized in Table 1.

Description of Damages and Losses

The failure of Teton Dam resulted in the loss of approximately one-third of the embankment material. The Teton power and pumping plant superstructure, switchyard, and warehouse were totally destroyed. The powerhouse substructure, turbines, generators, waterways, and associated electrical and mechanical equipment were covered by debris and sediment.

The spillway, river outlet works, and auxiliary outlet works were essentially undamaged. Access roads, land, recreation, and fish and wildlife facilities of the original plan and landscaping are either not usable or had not yet been constructed at the time of failure.

Of the total first phase Teton Basin Project cost of $85,676,000, approximately $40 million in facilities was lost or damaged. In addition, several million dollars has been spent to date investigating the failure of the dam.

Downstream from the damsite, accurate estimates of damage have not been completed. However, it is known that an area of approximately 300 square miles, extending 80 miles downstream on the Teton and Snake Rivers to the headwaters of American Falls Reservoir, was either fully or partially inundated. The peak floodflow was estimated to be roughly equivalent to that of the Mississippi River in flood stage. The tragic consequence was the loss of eleven lives and the disrupted lives of the 25,000 people who were left homeless as a result of the flooding.

Major damage occurred in the towns of Rexburg and Sugar City, and in the surrounding farmland. Between 16,000 and 20,000 head of livestock were lost and 32 miles of railroad were damaged or destroyed. Up to 100,000 acres of prime agricultural land were inundated, with all

Figure 22.—Flow increasing from leak at El. 5200.

Figure 23.—Dozers lost in leak at El. 5200.

Figure 24.—Approximately 11:30 a.m., June 5, 1976.

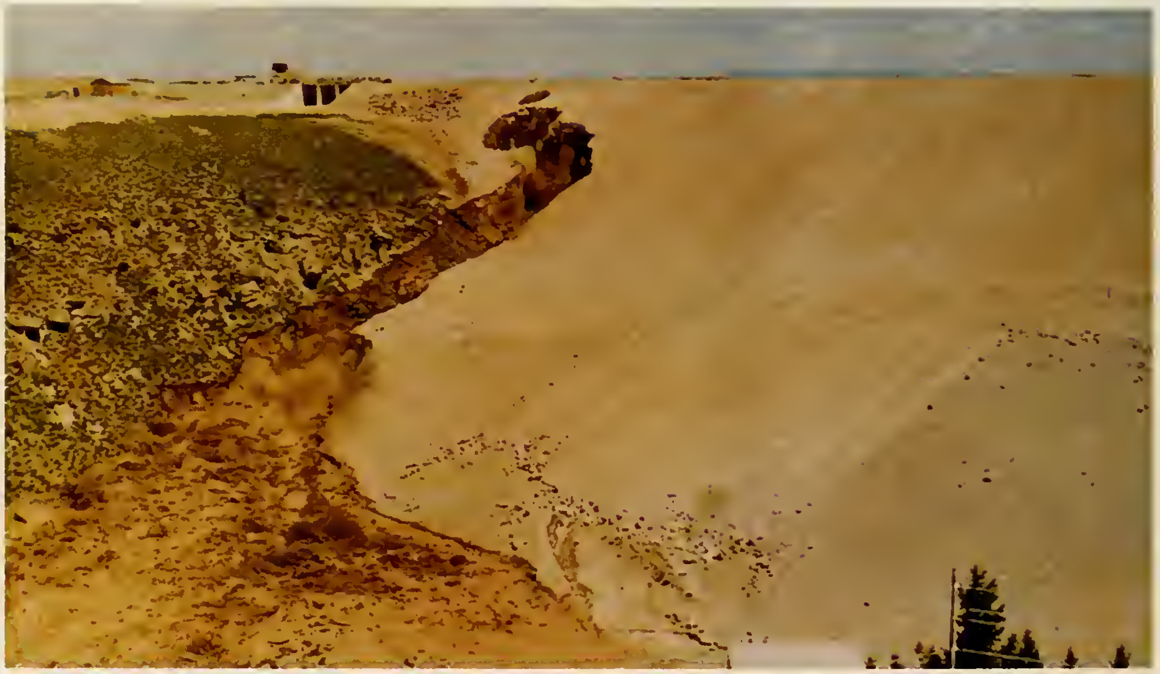


Figure 25.—Second hole in downstream slope of dam just after 11:30 a.m., June 5, 1976.

Figure 26.—About 11:50 a.m., June 5, 1976.

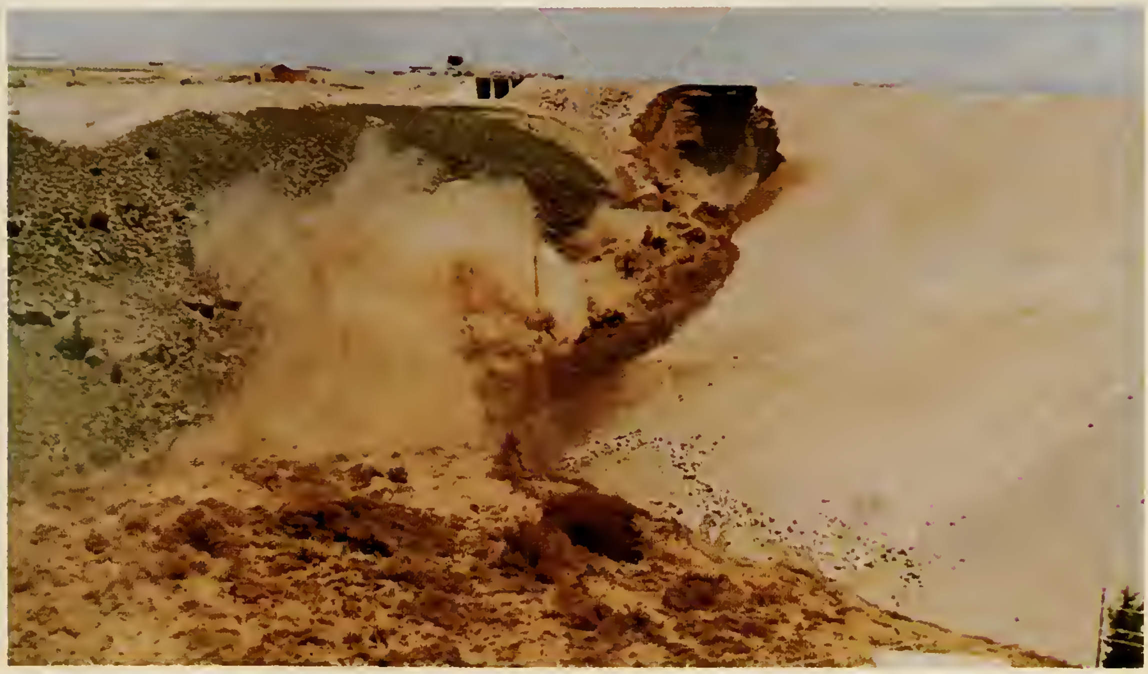


Figure 27.—Dam crest breaching 11:55 a.m., June 5, 1976.


Figure 28.—Early afternoon, June 5, 1976.

Table 1.–*CHRONOLOGY OF SIGNIFICANT HAPPENINGS AND OBSERVABLE EVENTS RELATED TO TETON DAM FAILURE*
(Times are approximate
Mountain Standard Time)

	Date	Time	Event
1.	June 3		Two small seeps noted 1,300 and 1,500 ft downstream from toe of dam on right abutment flowing 40 and 60 gpm, respectively
2.	June 4		One small seep noted 160 to 200 ft downstream from toe of dam on right abutment flowing 20 gpm
3.	June 5	7:00 a.m.	Slightly turbid leakage first noted near El. 5200 coming from right abutment
4.	June 5	7:30-8:00 a.m.	Turbid leakage first noted at El. 5045 coming from right abutment
5.	June 5	8:30 a.m.	Leakage at El. 5045, 750 ft downstream from centerline Sta. 17+30, examined by Ringel and estimated to be 20 to 30 cfs
6.	June 5	9:10 a.m.	Leakage at El. 5200, 280 ft downstream from centerline Sta. 15+25, examined by Robison and Aberle and estimated at 2 cfs. Erosion along right embankment-abutment contact noted
7.	June 5	9:20 a.m.	Leakage at El. 5045 examined by Robison, Aberle, and Ringel and estimated at 40 to 50 cfs
8.	June 5	10:15 a.m.	Wet spot at El. 5200, 280 ft downstream from centerline Sta. 15+25, and 15 to 20 ft from right abutment formed rapidly and began to leak and erode embankment
9.	June 5	10:30 a.m.	Loud noise heard by Aberle, Robison, and others
10.	June 5	10:40 a.m.	Two dozers begin to push material into hole at El. 5200
11.	June 5	10:43 a.m.	County sheriff called and notice given to begin evacuation of downstream areas
12.	June 5	11:00 a.m.	Whirlpool develops in reservoir near Sta. 13+75, 130 ft upstream from dam centerline (15 to 20 ft into the reservoir)
13.	June 5	11:00-11:10 a.m.	Efforts initiated to fill whirlpool
14.	June 5	11:10 a.m.	Dozer gets stuck at edge of downstream hole
15.	June 5	11:30 a.m.	Dozers slide into downstream hole
16.	June 5	11:45 a.m.	Sinkhole forms at Sta. 14+00 and El. 5315
17.	June 5	11:45 a.m.	Dozers that were attempting to fill whirlpool were removed from top of dam
18.	June 5	11:55 a.m.	Crest collapses
19.	June 5	11:57 a.m.	Embankment breached
20.	June 5	5:00-6:00 p.m.	Reservoir essentially empty

but 3,000 acres capable of being restored to full productivity.

Damage to private and public property was extensive. As of March 16, 1977, the Bureau of Reclamation had received 5,616 claims totaling more than $250 million. The Bureau has paid over $138 million on 4,938 claims. It is now estimated that the total claims to be paid by the Bureau of Reclamation will approach $400 million. This does not include damages to the dam and appurtenant structures nor damages paid from other sources. A total cost of damages is not yet available.

Scope of the IRG Review

Site Investigations by the IRG

The IRG made its initial visit to Teton Dam on June 16, 1976. The group inspected the right abutment and returned to Denver, Colorado, on June 17, 1976. On September 14, 1976, several members and alternates of the IRG made a field trip to the damsite and project office. During this field trip the main interest was in the excavation of the right abutment remnant where removal was approaching El. 5250 with trenches being excavated to El. 5245. After trenching, complete excavation proceeded in 5-foot lifts. Core holes were being drilled in the right spillway bay near the center line of the grout curtain.

On September 15 and 16, 1976, following the field trip, the IRG held meetings, which were open to the press and public in Idaho Falls, Idaho. On the afternoon of September 15, 1976, the group interviewed Professors Robert Curry[1] and Marshall Corbett.[2]

The IRG visited the worksite again on November 9, 1976, to make a detailed inspection of the right abutment. At this time, removal of the right abutment remnant and washing of the abutment had been completed.

[1] Dr. Robert R. Curry, Professor of Geology, University of Montana
[2] Dr. Marshall K. Corbett, Professor of Geology, Idaho State University

Interviews with Professors Robert Curry and Marshall Corbett

The Subcommittee on Conservation, Energy, and Natural Resources of the Government Operations Committee, U.S. House of Representatives, chaired by Representative Leo J. Ryan of California, held hearings on the Teton Dam disaster in Washington, D.C., on August 5, 6, and 31, 1976. Professors Curry and Corbett testified and presented two additional possible causes of the failure that the IRG had not considered up to that point.

The IRG arranged a hearing with Curry and Corbett in Idaho Falls, Idaho, on September 15, 1976. At this hearing, they made statements and answered questions concerning their theories of possible causes of failure. An unedited stenographic record of the IRG hearing is on file with the Department of the Interior.

Professor Corbett postulated that the failure could have resulted from hydraulic pressures deep in the foundation that produced differential lifting of the dam. Professor Curry suggested that consolidation of sediments underlying the volcanic rock foundations of the dam caused differential settlement and cracking of the dam. Both claimed that there was a paucity of knowledge of regional geology in the area of the Teton Dam and that the information on deep underlying foundation materials at the damsite was very limited. These hypotheses are explained more fully in Chapter 6, Discussion and Evaluation of Investigations.

Review of Postfailure Investigations

The IRG has had the opportunity to review data from all postfailure field investigations of the Teton Dam. These investigations included resurveys of monuments in the area, a program of drilling and water testing in the dam foundation to check the adequacy of the grouting program, and a program of hydraulic fracturing tests of the embankment remaining on the left abutment. In addition to this, that part of the embankment remaining on the valley slope of the right abutment was carefully removed, its conditions observed, and samples taken for laboratory testing. After removal of the embankment, the entire area of the key trench and adjacent areas were cleaned and detailed geologic maps and sections were prepared. In addition, topographic mapping and cross-sectioning of the right abutment, borehole camera photography of selected holes, and testing of transmissibility of selected rock joints were carried out. Postfailure laboratory testing was done to confirm the embankment design parameters, and to determine the nonlinear parameters needed for finite element analyses of the embankment. Finite element stress analyses were performed for transverse and longitudinal sections through the dam and for sections in the foundation. Arrangements for work in the field, laboratories, and offices were made in close cooperation with the staff of the Independent Panel to eliminate duplication of effort.

Data Supplied by the Bureau of Reclamation

The IRG received a detailed briefing at Reclamation's E&R Center on June 15, 1976. Basic background data such as the plans and specifications were distributed at this briefing. Subsequently, the remaining exhibits listed in Appendix E were given to all members of the IRG, their alternates, and members of the task groups as appropriate. Many of these documents were supplemented and expanded at the request of both the IRG and the Independent Panel. Also, the field data generated during the investigation were given to the IRG. These documents are compiled and maintained in two repositories: (1) the Bureau of Reclamation in Denver, Colorado, and (2) the Department of the Interior in Washington, D.C.

At the request of the Independent Panel and the IRG, two physical models of Teton Dam were prepared for the failure investigations: (1) a model of the entire dam and appurtenances, at a scale of 1:1000, showing both the foundation and the dam, and (2) a model of the right abutment and dam at a scale of 1:400, showing pre- and postfailure conditions.

Task Group Reviews

Three task groups were organized with staff members from the Federal agencies represented on the IRG. The responsibility of the groups was to obtain information that would assist the IRG in carrying out its charges. The scope of their activities are given below:

Geology.–The geology group made several visits to the damsite to examine the geologic conditions, held discussions with Bureau of Reclamation project and regional geologists, and reviewed project records. To obtain additional expertise, they consulted with Federal specialists concerning the regional geology of the Snake River Plain, volcanic geology, engineering geology of damsites in volcanic rocks, groundwater geology, and seismology.

They reviewed Bureau of Reclamation prefailure investigations of the site geology and recommended to the Bureau of Reclamation that further geologic studies be conducted.

They reviewed and evaluated the results of the recommended postfailure studies.

Appendix B is the report of this task group.

Grouting.–The grouting group evaluated the adequacy of the grout curtain to perform its design function, evaluated the adequacy of the rock surface treatment program, and determined the probable contribution, or lack thereof, of the rock treatment to the failure. During the course of its activities, the Grouting Task Group made two visits to Teton Dam to interview Bureau construction personnel, interview the grouting contractor, study Bureau photographs and inspection documents, and inspect the dam and abutments. One visit was also made to Denver to interview Bureau design personnel.

Appendix C is the report of this task group.

Embankment Construction Review.–The embankment construction group reviewed the construction aspects regarding the embankment and foundation, exclusive of the foundation pressure grouting. During the review made by the task group, a visit was made to the damsite, a number of construction inspectors were interviewed, and construction records and reports were reviewed. Analysis of soil tests was made to determine if design requirements were met and if construction control was obtained.

Appendix D is the report of this task group.

Questions and Answers Regarding Design and Construction

The IRG developed and sent to the Commissioner, Bureau of Reclamation, a number of questions concerning the design of the dam and the function and responsibilities of the Division of Design during the construction of the dam.

Appendix G contains these questions and the answers supplied by the Bureau. Also, in Appendix G, is a second set of questions posed by the Independent Panel and answers supplied by the Bureau of Reclamation and the construction contractor.

Procedures Review

The original charge to IRG directed that the cause of the failure be determined and that recommendations, if warranted by its findings, be provided to prevent recurrence of such failures. To respond appropriately to this charge, the IRG believes it is necessary to review documentation of the Bureau of Reclamation technical decisionmaking process and to conduct interviews with appropriate Bureau personnel. These reviews and interviews will be conducted so that the design procedures and design review and technical decisionmaking processes will be identified. Recommendations will be based on the review and interviews.

Possible Causes of Failure

Introduction

In the IRG Second Interim Report, October 21, 1976, six possible causes of the failure were presented in the order of their relative probability of occurrence. These six possible causes of failure are described below. The investigations and evaluations carried out to determine the most probable cause of failure and to explore the adequacy of design and construction of Teton Dam are discussed in subsequent chapters.

Cracking of Zone 1 Material from Differential Settlement or Hydraulic Fracturing

The steep abutment slope and key trench in contact with brittle zone 1 material may have led to cracking of this material because of differential settlement. Also, there is a possibility that hydraulic fracturing may have occurred because of the stresses introduced in zone 1 by the unusual abutment geometry. Arching in or above the key trench could pevent the full weight of overlying embankment material from bearing on the underlying material. This could permit hydraulic fracturing and provide a path for seepage through the zone 1 material. Water seeping through these cracks in an otherwise impervious zone may have led to piping through the embankment.

Piping Along the Interface Between the Zone 1 Material and the Rock Foundation

The zone 1 material, an erodible silt, was in direct contact with the open-jointed rock foundation in the right abutment. The differential head across the key trench was conducive to seepage along paths of any poorly compacted material and/or poorly treated surface joints. Water seeping along the contact could have caused erosion of zone 1 material.

Flow Through the Grout Curtain

The foundation rock in the right abutment is so open jointed that there was a possibility of major leakage through the grout curtain. If large flows passed through the grout curtain and came into contact with zone 1 material through downstream joints, the zone 1 material could have been sufficiently eroded to create a void which would have gradually enlarged through collapse of the adjacent material. This process eventually could have reached the upstream face of zone 1, resulting in failure of the embankment.

Flow Bypassing the Grout Curtain

Flow could have bypassed the grout curtain in either of two ways:

(1) *Flow around the end of the grout curtain.*–Water level measurements in observation wells during reservoir filling indicated high piezometric levels in the open-jointed rock of the right abutment at the end of the grout curtain prior to failure of the dam. Water from this area could have flowed along the downstream side of the grout curtain to zone 1 of the embankment through high-angle joints having suitable orientation. This flow eventually could have led to failure in somewhat the same manner as discussed in the section, Flow Through the Grout Curtain.

(2) *Flow under the grout curtain.*–The open-jointed rock in the right abutment provides possible entrances and exits for water to pass under the grout curtain. This flow could have entered the embankment, resulting in failure in somewhat the same manner as described in the section, Flow Through the Grout Curtain.

Cracking Due to Foundation Settlement[1]

The open-jointed rhyolitic tuff and the basalt flow upon which the dam is founded are underlain, at a depth as little as 170 feet beneath the rock floor of the canyon, by an unknown thickness of poorly to moderately indurated alluvial and/or lake sediments that are relatively weak compared to the overlying rock. Little is known about the thickness or properties of these sediments at the damsite. It has been postulated that they consolidated under the weight of the dam and reservoir, thus leading to settlement of the bedrock foundation and of the dam itself.

[1] This possible cause of failure and its description have been developed by the group based on its interpretation of statements made by Professor Curry before the Conservation, Energy and Natural Resources Subcommittee of the Committee on Government Operations, U.S. House of Representatives, Washington, D.C., August 5, 1976, and before the U.S. Department of the Interior Teton Dam Failure Review Group in Idaho Falls, Idaho, September 15, 1976.

With considerable local variation in thickness of the sediments, the amount of settlement due to their consolidation could differ from one part of the foundation to another. Such differential settlement could have opened cracks in the right abutment and through the grout curtain within the right abutment, at the contact between the right abutment and the embankment and/or within the embankment. Seepage through these cracks could have led to failure of the embankment by piping.

Cracking Due to Hydraulic Uplift[2]

This possible cause has been suggested by Corbett who summarized the hypothesis as follows:

> " * * * , that the dam could have floated. This in a sense is a simplification. But, what I mean is that the ground water pressures in the sediments and the loose unconsolidated material below the rhyolite flow, and therefore, the dam itself, may have been subject to quick loading or some sort of stress which would cause other pressures to build up in the ground water system. This would be a hydraulic system; therefore, the pressures would be hydrostatic and equal in all directions. And certainly a certain degree of lifting might take place."

[2] Quotation from Professor Corbett's statement before the Conservation, Energy and Natural Resources Subcommittee of the Government Operations Committee, U.S. House of Representatives, Washington, D.C., August 5, 1976.

Discussion and Evaluation of Investigations

Introduction

This chapter describes the postfailure investigations undertaken to determine the cause of the failure of Teton Dam and evaluates their results. Many of these investigations were monitored by the IRG Task Groups. Detailed descriptions of the investigations and the results therefrom are contained in the task group reports (Appendixes B, C, and D).

Geologic Studies

As stated in Chapter 4, Scope of the IRG Review, the Geology Task Group performed studies to determine the adequacy of geologic investigations of the Teton damsite prior to and during construction, and to review the geologic conditions extant at Teton Dam and their possible influence on the failure. The regional and site geologic conditions have been summarized in Chapter 2, General Project Description. The detailed description of geologic studies, presentation of the results of the studies, and discussion of their significance is contained in Appendix B. The following discussion summarizes the investigations and their results concerning the adequacy of the Bureau of Reclamation geologic investigations and the relationship of existing geologic conditions to the failure.

Adequacy of Geologic Investigations

This study was performed by examining available reports, drawings, boring logs, and other geologic records of the Bureau of Reclamation. The descriptions of prefailure geologic investigations and of parts of the site geology were mainly obtained from the unpublished report, "Teton Dam Geology," June 1976, by the Geology and Geotechnology Branch, Bureau of Reclamation. The evaluation of construction geologic investigations was developed mainly from examination of the construction geologic maps. Observation well readings were examined to determine the type of groundwater flow pattern that could have developed.

Preconstruction Investigations.–Considerable geologic investigations of alternate sites were performed during the site selection for Teton Dam (see Chapter 2, General Project Description). The acquisition of geologic information was given high priority.

Subsurface explorations at the site eventually selected were started in 1957 with the drilling of two core borings by the Corp of Engineers. In all, 102 core borings, totaling 17,824 lineal feet, were drilled prior to construction to develop foundation and potential seepage loss information. This included 10 borings to evaluate a pilot grouting program conducted in the canyon and on the left abutment. A summary of preconstruction core drilling is included in Table 2.

Some of the foundation borings were surveyed with a downhole TV camera to develop information on joint patterns. Index tests (such as specific gravity, unconfined compressive strength, and absorption) were performed on selected core samples from the welded tuff and basalt.

Areal geologic maps of the damsite and large-scale detailed outcrop maps of the abutments were prepared.

All the information relating to the preconstruction geologic investigations program through early 1971, including logs, maps, sections, etc., is in the Preconstruction Geologic Report.

The preconstruction geologic investigations were sufficient to identify the important geologic features that needed to be considered for design. The IRG is concerned that the geologic data may not have been documented in a manner which effectively displayed the critical features to the designers. This subject will be studied in further investigations of Bureau procedures.

Construction Investigations.–Investigations and additional explorations during construction consisted of the development of as-built geologic foundation and tunnel maps, core borings, auger borings, and test trenches.

As-built geologic maps were made of the cutoff and key trenches and of the spillway excavation. The maps of the cutoff and key trenches also depict the adjacent parts of the canyon walls that were to be covered by zone 1 material. Geologic materials were identified and joints were mapped and partially described. Much of the bottom of the left key trench and the extreme right part of the right key trench were not mapped.

Inclined core borings were drilled in the upper portion of the right abutment to explore an area where fissures and cavities had been discovered in the key trench excavation. A similar deep, inclined core boring was drilled for the same reason in the upper portion of the left abutment. Some core borings in the right abutment were kept open for use as observation wells. Additional borings were drilled for use as groundwater observation wells, one on each abutment and one near the outlet of the river outlet works tunnel. Table 2 summarizes the construction core drilling.

During excavation of the cutoff trench in the canyon section, auger holes were bored and test trenches were excavated to define the limit and extent of silt and clay layers encountered near the bottom of the excavation.

In general, the construction geologic investigations in the right abutment and associated appurtenant works were adequate to verify the basic geologic conditions determined by the preconstruction investigations. However, in the area of the failure, the construction geologic mapping was deficient. The significant open brecciated contact between the middle and lower units of the welded tuff was not mapped. The postfailure mapping of the key trench recorded the presence of several throughgoing high-angle joints that were not mapped during construction; one of these joints is as much as 2-1/2 inches in width. Numerous low-angle joints were not recorded on the construction geologic map. The information on joint openings was inadequate; information was not provided on joint openness unless the opening was at least 0.1 foot (more than 1 inch).

Postfailure Geologic Studies

Geologic conditions that have been hypothesized as being important to the failure are active faults, rock foundation cracking due to consolidaton of deep-seated sedimentary materials, and the permeability characteristics of the right abutment rock foundation. The Geology Task Group's postfailure geologic

Table 2.–*Summary of preconstruction and construction core drilling–Teton Dam*

[From U.S. Bureau of Reclamation, 1976]

SUMMARY OF PRECONSTRUCTION CORE DRILLING

Location	Number of holes	Depth Minimum	Depth Maximum	Total feet	Purpose
Left abutment	26*	19.6	556.0	6,569.8	Deep and shallow foundation conditions, permeabilities (seepage loss potential), and effectiveness of pilot grouting program
Valley floor	9	58.5	505.8	2,228.8	
Right abutment	9	296.8	698.0	3,683.8	
Appurtenant structures	56	26.0	303.7	4,337.0	Deep and shallow foundation conditions, some permeabilities, and water table locations
Reservoir	2	498.8	506.0	1,004.8	Reservoir seepage loss potential
TOTAL	102			17,824.2	
SUMMARY OF CORE DRILLING DURING CONSTRUCTION ("500" series, drilled 1974-75)					
Left abutment	2	343.7	397.1	740.8	Fissure and void exploration and ground-water observation
Right abutment	4	399.8	646.0	2,285.1	
River outlet works area	1			372.0	Ground-water observation
TOTAL	7			3,397.9	

*Includes 10 core holes for pilot grouting investigation.

studies were aimed mainly at determining the relationship between these geologic conditions and the failure of the dam.

If a dam were located on or in the immediate vicinity of an active fault, its safety would be jeopardized for two reasons. The first is the shaking that could result from an earthquake originating on the fault, and the second is the actual physical displacement of the foundation and the dam due to the fault movement. The subject of faulting is addressed in Chapter 2, General Project Description and, in detail, in Appendix B. Some inferred northwest-trending faults have been mapped 3 to 4 miles north-northeast of the damsite. If these faults exist, they may be part of a collapse structure along the margin of a caldera. A well-defined lineament was observed on aerial photos to parallel the Teton River about 500 to 800 feet west of the right abutment. This lineament was tentatively considered by the U.S. Geological Survey to be an extension of an inferred northeast-trending fault. Reexamination of available data led to relocating this inferred fault to a position as much as 3 miles northwest of the lineament observed on the aerial photos. Well-defined, northwest-trending faults have been observed within 11 miles southwest of the dam and to within 8 miles east of the dam. Well-defined, northeast-trending faults have been mapped no closer than 7 miles northeast of the dam and 10 miles south of the dam.

No active faults were identified at or near the dam.

Professor Curry hypothesized that the sediments underlying the welded tuff at the dam consolidated under the weight of the dam and reservoir. Because the thickness of these sediments varies, the amount of settlement due to consolidation would vary from point to point. This differential settlement could have opened cracks in the right abutment and in the grout curtain. Seepage through these cracks could have led to piping. The geologic history of the damsite area provides an insight into the likelihood of occurrence of these events.

After deposition of the welded tuff and before the canyon of Teton River was eroded into the tuff, the weight of these volcanic rocks on the underlying sediments was considerably greater than the more recently applied load of the dam and reservoir. The underlying sediments were consolidated under this heavy load of rock over a period of hundreds of thousands of years. Such preconsolidated materials will not further consolidate any significant amount unless the original load is exceeded. Thus, there is little likelihood that the sediments consolidated under the weight of the dam and reservoir. This subject is treated further in Chapter 6, under Analysis of Postfailure Embankment Remnants.

The permeability of the rock mass comprising the right abutment is considered to be the most important geologic condition associated with the failure of Teton Dam. As a consequence, extensive postfailure investigations of rock mass permeability and associated subjects were performed. These investigations included: (1) postfailure geologic mapping of the right abutment, (2) evaluation of the information obtained from postfailure core borings, (3) evaluation of groundwater observations made on the right abutment, and (4) study of the cause of rock jointing in the welded tuff.

The geologic mapping of the right abutment rock surface was performed to locate and describe all visible geologic features (joints, foliation, breccia zones, etc.) that could have been involved in the failure.

With the key trench and adjacent areas cleaned, the entire abutment was photographed, using oblique views. Enlargements of these photographs were used as a base on which postfailure geologic mapping of the abutment was performed. Also, a strip map of the bottom of the key trench was prepared. This map differs from the abutment maps in that features were projected into a horizontal plane. The strip map accurately portrays feature locations in plan view; the abutment maps do not. All mapped joints (fractures) were assigned identifying numbers and located on the photo overlays. A tabulation was made describing, for each joint, the attitude, width of opening, extent of filling, nature of the filling, and other properties. Geologic sections were prepared along the upstream and downstream lines of grout holes and 150 feet upstream and 100 feet downstream of the centerline of the dam.

The rock core drilling and water pressure testing were performed to check the effectiveness of the grout curtain and to determine the physical characteristics of the sediments underlying the welded tuff. The first area investigated was the spillway channel upstream of the ogee, where both blanket and curtain grouting under the structure had been performed and where the key trench was omitted. Eleven holes were drilled and water pressure tests were performed in each.

The second area of the right abutment that was investigated was beneath the embankment north of the spillway structure. Large open fissures had been encountered during construction in this area, and remedial work had included placing of concrete, as well as establishing a deep grout curtain. Three holes were drilled and pressure tested to a depth of several hundred feet. One of these holes, drill hole 651B, was extended vertically to a depth of 890 feet to develop information about the underlying sediments. Initially, drill hole 651, an NX diamond drill hole, went to a depth of 622 feet; at this depth it was decided that a large-size hole would better develop information on the strata. Due to drilling difficulties in drill hole 651A, it was replaced by drill hole 651B, which was rock bitted to 622 feet and core drilled to the final depth.

The third area of the right abutment that was investigated by core drilling and water pressure testing was the key trench riverward of the spillway structure. In all, 24 holes were completed in this area to check the adequacy of the grout curtain.

The locations of all borings, all postfailure boring logs that have been prepared, and the results of the water pressure testing are included in Appendix C-5 of the Grouting Task Group Report. The rock cores from all borings have been photographed for record purposes. In addition, five holes have been photographed in color, using a borehole camera developed by the Corps of Engineers. Ten additional holes have been left open to permit borehole photography in the spring of 1977.

The principal data concerning hydrologic conditions in the right abutment lie in the observations of water levels in drill holes that were monitored for this purpose. The Geology Task Group analyzed the water level measurements made in right abutment drill holes 5, 6, 504, 501, and 506(2). The locations of these holes are shown on Figure 29. Analyses were performed on variations of water levels with time, variation of rates of water level change with time, and variation of differences in water level between selected locations with time.

The permeability of the right abutment foundation rock (welded tuff) is almost wholly controlled by spacing, openness, and continuity of fractures. The preconstruction foundation core drilling and water testing indicated high fracture permeabilities near the surface, which decreased somewhat with depth. The preconstruction pilot grouting program showed that the near-surface rock in the abutment was so permeable that curtain grouting could not be effectively accomplished. This condition resulted in a decision to use the 70-foot-deep key trenches in the abutments.

An attempt was made to analyze behavior of the groundwater in the area of the right abutment during the filling of the reservoir. This analysis was made difficult by the low number of groundwater observation wells, the absence of wells in locations critical to the analysis, and the lack of frequent readings. However, the analyses that were performed tend to confirm the permeability of the welded tuff. The rates at which the water levels in the observation wells followed the rise of reservoir water varied over the filling period. This may be interpreted to mean that zones of relatively higher permeability were being encountered by the rising water. Figure 30 shows the relative positions of the water levels in the observation wells and in the reservoir with time.

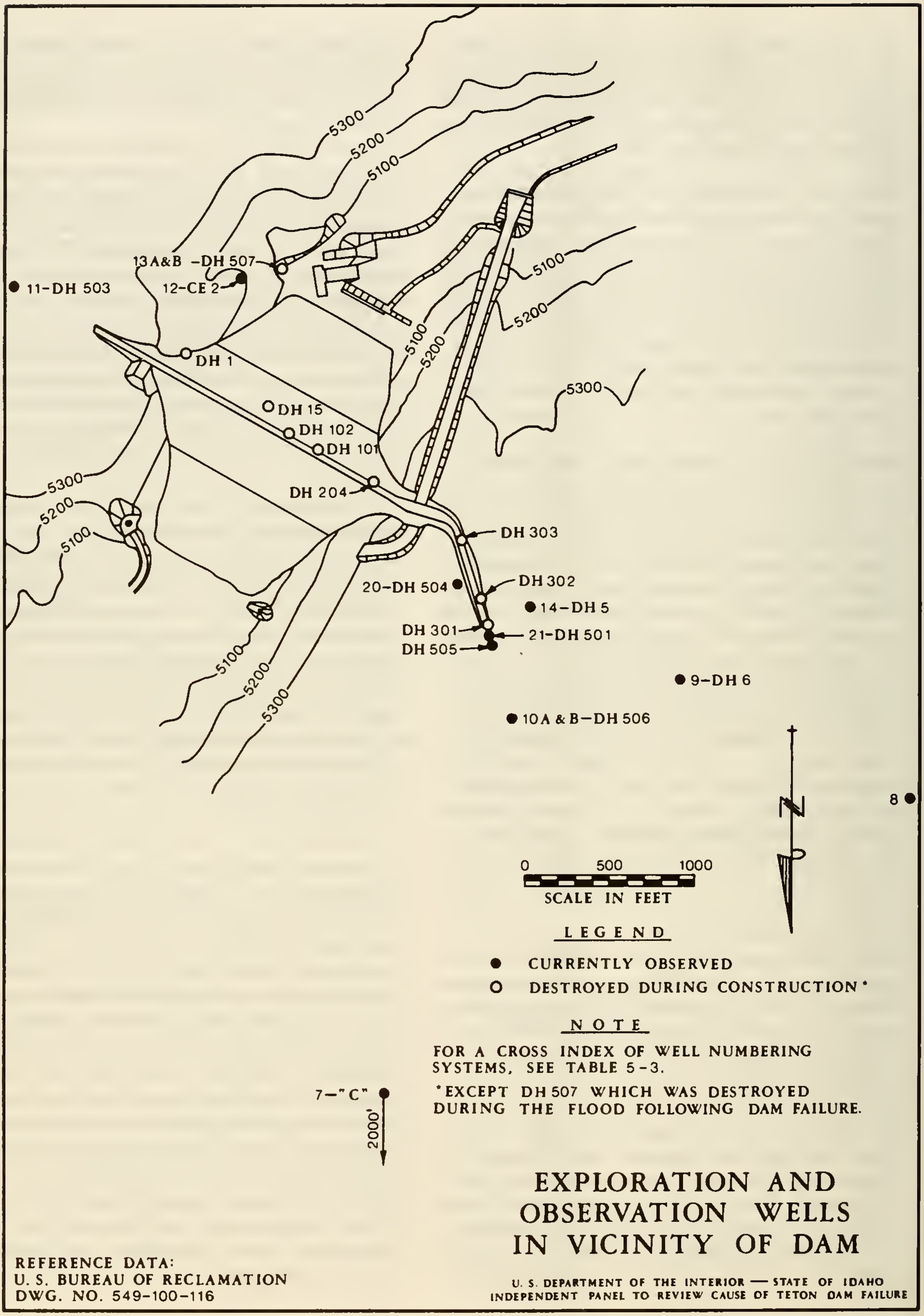
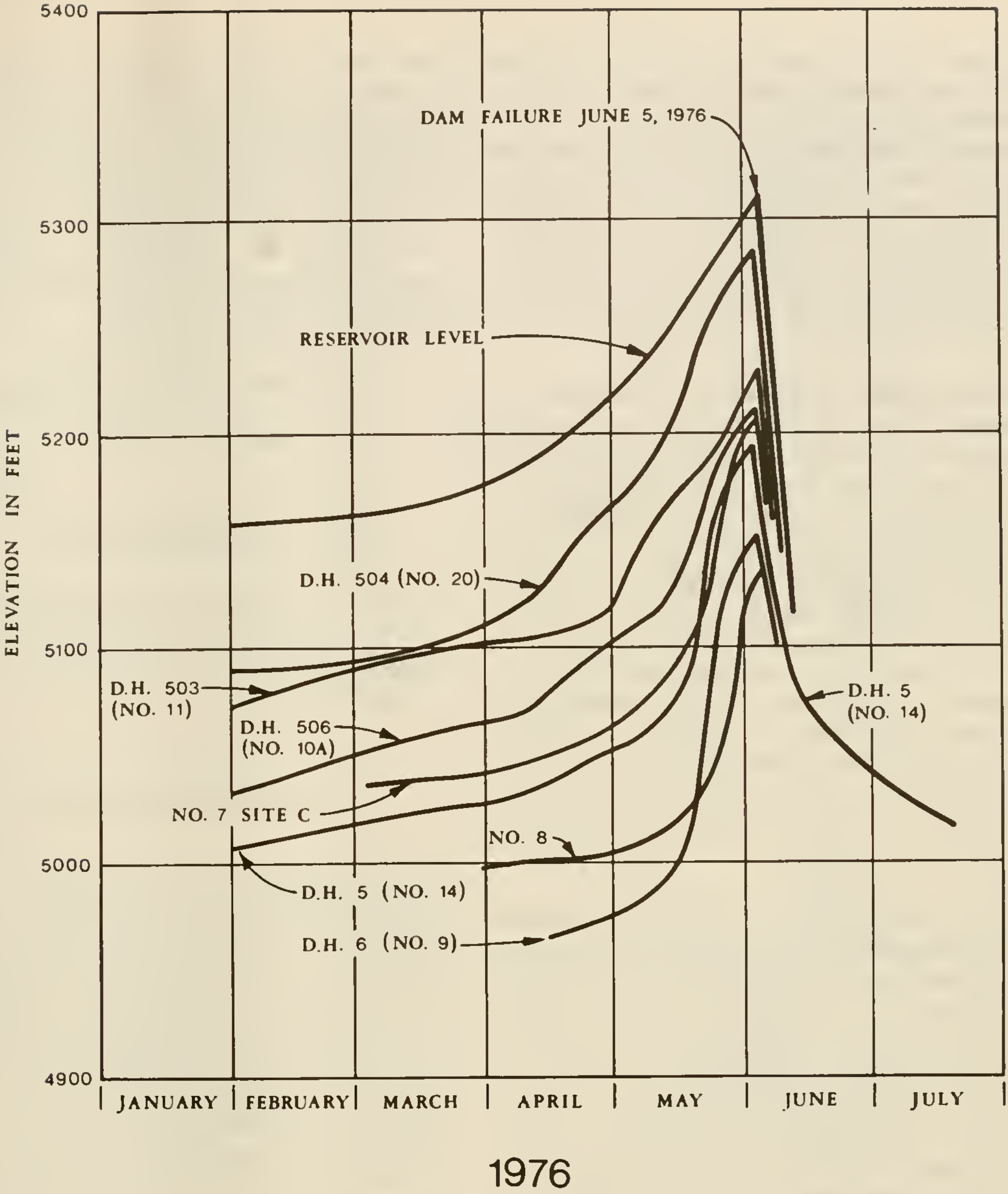


Figure 29.—Location of exploration and observation wells in the vicinity of Teton Dam.



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

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

Figure 30.—Water level elevations in observation wells.

Contours of the groundwater surface at various times were drawn from water levels in three observation wells, drill holes 5, 6, and 506. A sequence of selected contour maps, covering the period of time from reservoir filling to just after failure is presented on Figure 31. These contours indicate apparently normal behavior of reservoir water flowing through permeable abutment rocks along and around a less permeable curtain of grouted rock.

The postfailure geologic mapping confirmed the presence of open fractures in the welded tuff, along which considerable quantities of water could have flowed. Some high-angle open joints were traced for distances up to 280 feet. Upstream from the key trench the contact between the middle and lower units of the welded tuff was characterized by an open brecciated zone. Many throughgoing low-angle joints, or zones of joints were recorded, paralleling the foliation. A coincidence of major, throughgoing joints occurs along the key trench between grout cap centerline Stas. 13+00 and 14+00, the interval that includes the whirlpool that developed in the reservoir about an hour before the dam was breached.

Although the main purpose of the postfailure core drilling program was to check the adequacy of the grout curtain, much valuable information was obtained on the condition of the abutment rock. The locations of these borings are shown on Figure 32. The boring logs and water pressure data are contained in Appendix C. The postfailure drilling indicated zones of high permeability in the vicinity of Sta. 13+50, even though curtain grouting had been performed. However, it is not known if this permeable zone occurred as a result of loosening of jointed rock during the failure. One of the postfailure borings in the vicinity of Stas. 3+00 to 5+00 (drill hole 650) was oriented so that it penetrated rock outside from the grout curtain. Water pressure tests in this boring confirmed the high permeabilities observed during the preconstruction investigations.

As stated in Chapter 2, General Project Description, the foliation and associated low-angle joints in the welded tuff appear to be the result of accumulation, compaction, and welding of the ash-flow deposits. The high-angle joints probably resulted from tensional stresses caused by cooling of the rock after it solidified. The high-angle joints appear to have opened further after the rock mass cooled, thus increasing its permeability. Three main possibilities have been advanced for this further opening of high-angle joints:

- horizontal tectonic extension
- gravitational creep toward the present canyon
- relief of horizontal stresses

Of these possibilities, horizontal tectonic extension appears to be the most likely. The logic supporting this conclusion is presented in Appendix B-1.

Foundation Treatment

Condition of Foundation.–During the investigation of the damsite and the development of the design, considerable evidence was compiled which demonstrated that the dam was to be placed on a foundation of highly fractured and permeable rock. Groundwater studies generally showed the water table to be at or below river elevation; return water was uncommon during exploratory drilling; water tests during exploratory drilling showed high permeabilities, and rock outcrops in the side walls of the canyon contained well-developed patterns of numerous open joints.

Curtain Grouting.–A pilot grouting program was instituted in 1969 to determine the groutability of the basalt in the canyon floor, the alluvial material below the basalt, and the welded tuff in the abutments. Bids were let for the injection of an estimated 260,000 cubic feet of grout in the testing program. More than twice that amount was pumped during the course of the pilot grouting program.

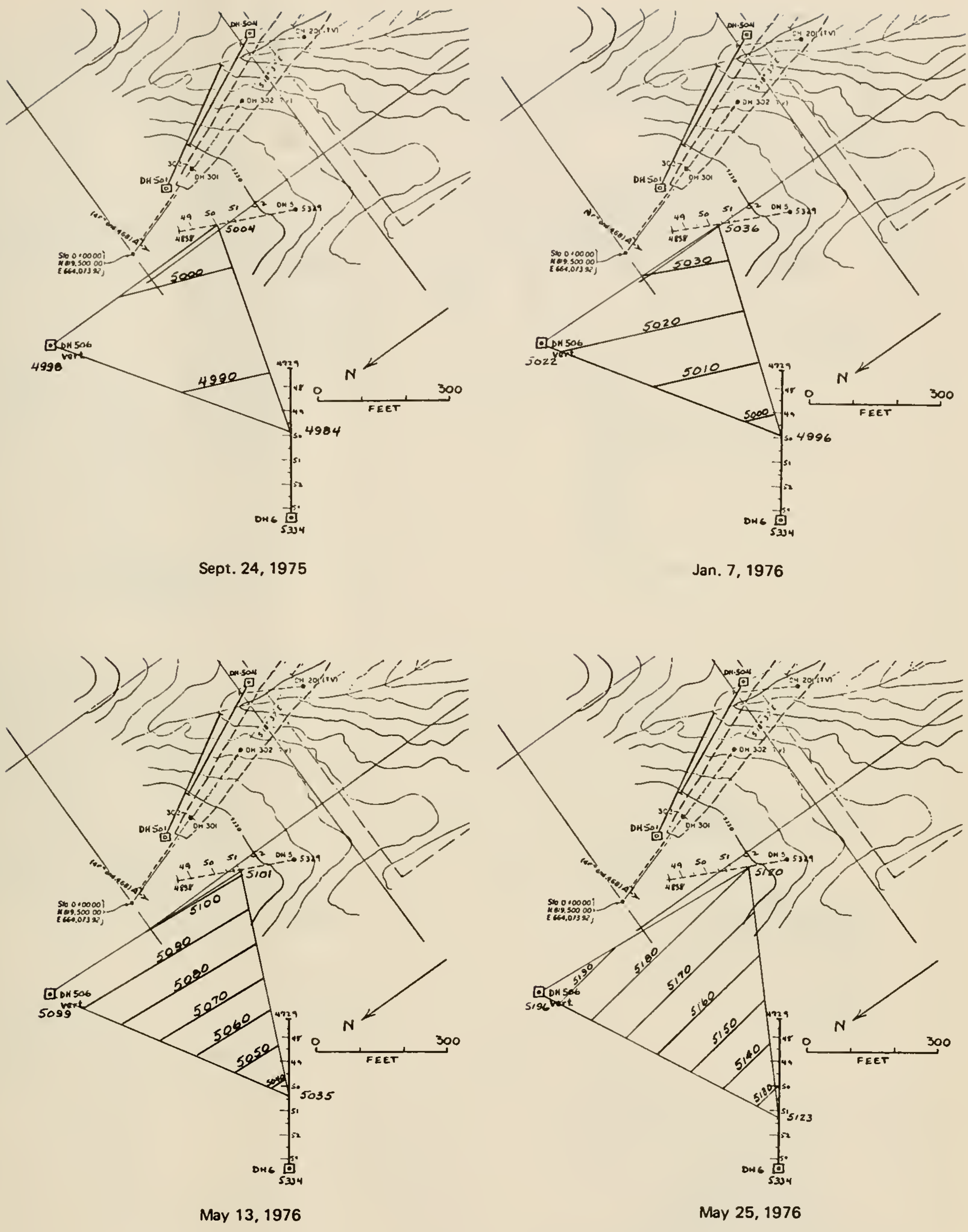
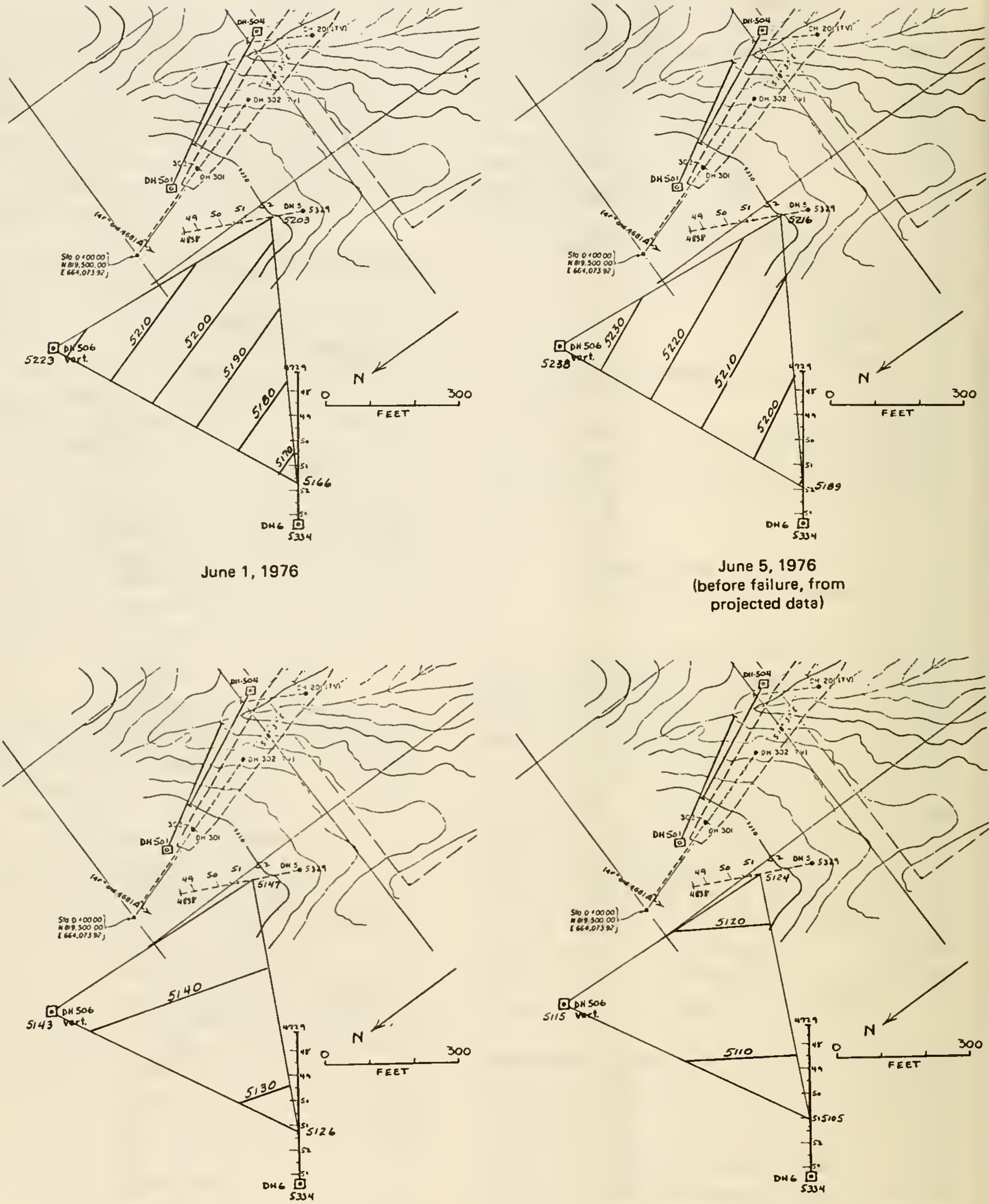


Figure 31.—Contours on groundwater surface adjacent to Teton Dam. (Sheet 1 of 2)



June 1, 1976

June 5, 1976
(before failure, from projected data)

June 9, 1976

June 13, 1976

Figure 31.—Contours on groundwater surface adjacent to Teton Dam. (Sheet 2 of 2)

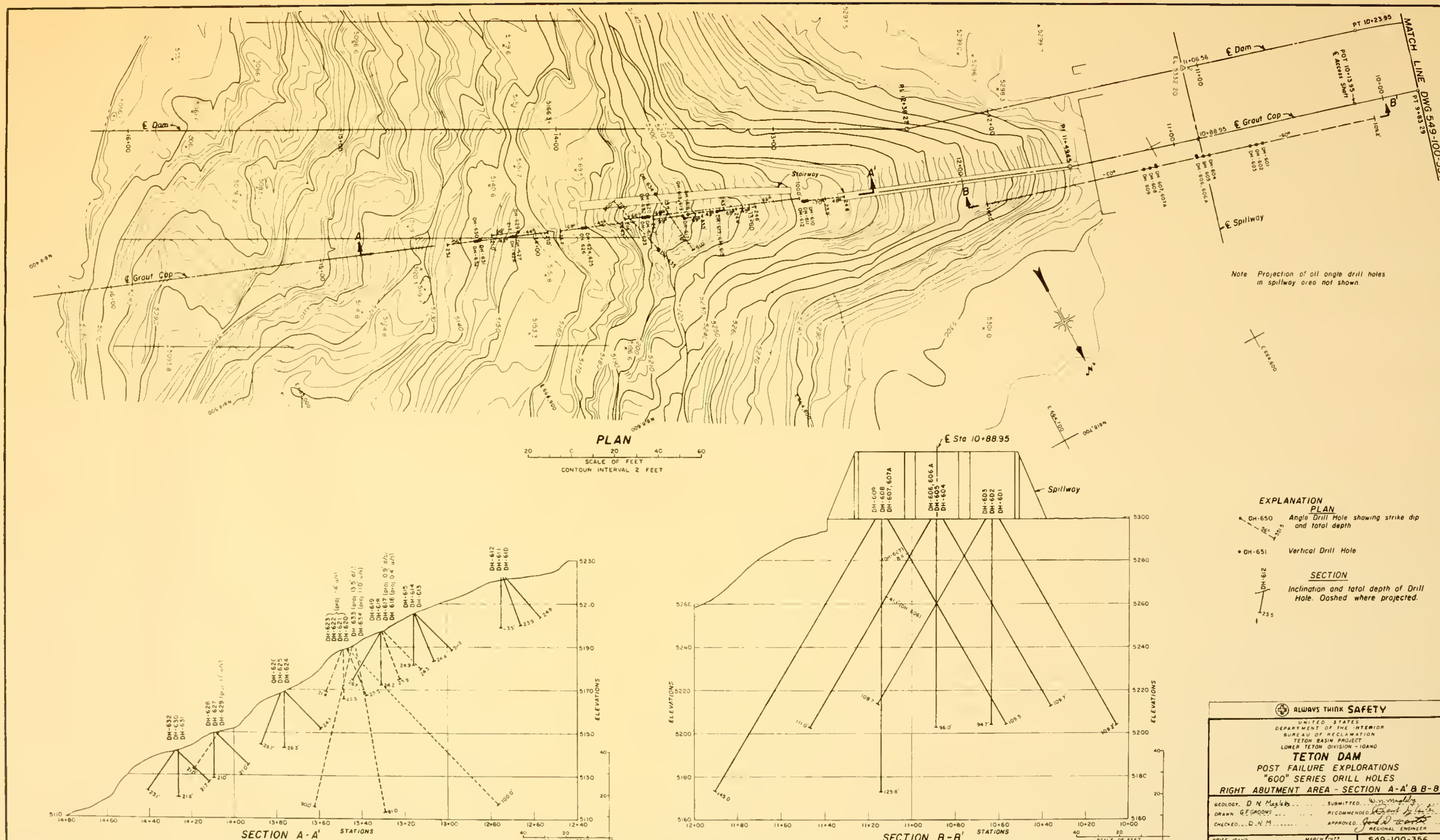


Figure 32.—Locations of postfailure core borings (sheet 1 of 2).

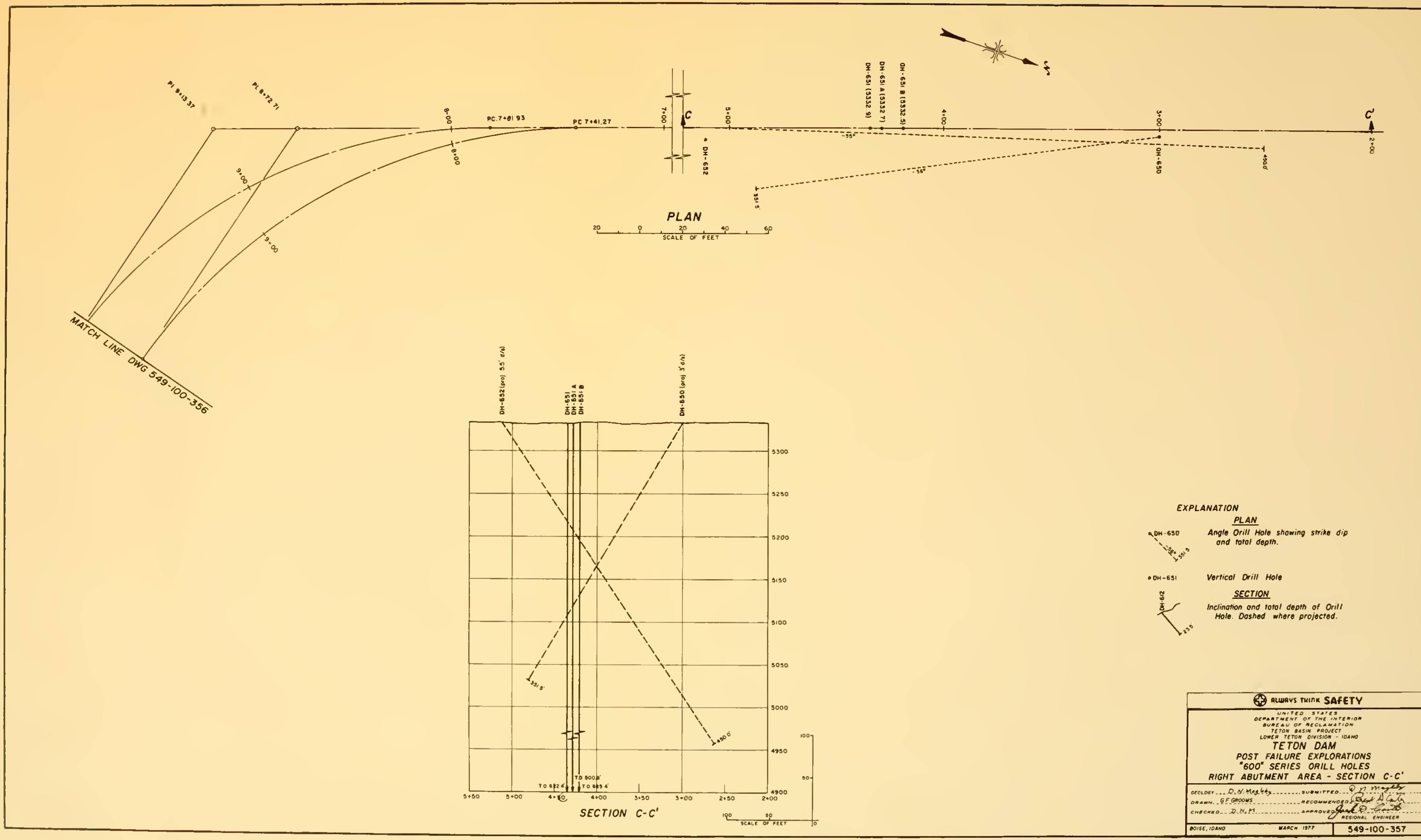


Figure 32.–Locations of postfailure core borings (sheet 2 of 2).

High grout takes were common in the abutment holes. The combined takes of just two of the pilot holes were almost 16,000 sacks of cement and 18,000 cubic feet of sand. Several holes could not be grouted to refusal at depths of less than 70 feet. Grout travel greater than 300 feet was observed. In the canyon floor, the basalt was so fractured that drilling of angled holes was difficult, but the basalt was found to be easily grouted. The alluvial material under the basalt was found to be groutable using three lines of holes, with the two outer lines grouted first.

In 1970, 10 exploratory holes were drilled to observe the condition of areas grouted during the pilot grouting tests. The results of the pilot grouting and subsequent exploration showed that in the abutments above El. 5100, the upper 70 feet of tuff was so permeable that a satisfactory grout curtain could not be economically constructed. As a consequence, a decision was made to excavate a 70-foot-deep key trench into each abutment above El. 5100. Blanket grouting was called for to locally replace the key trench under the spillway. A grout curtain would then be constructed with three rows of holes to a depth of 260 feet below the bottom of the key trench. Below El. 5100, where the rock was tighter, a single-line grout curtain was called for, except through the basalt and lower alluvium in the left half of the canyon floor. Here, three rows of holes were again used.

The details of the curtain grouting program are shown on Figure 11. Where a three-line curtain was used, the upstream and downstream lines were not grouted to closure, but were considered complete with holes grouted on 20-foot centers. The outer two rows were grouted first to act as backups for the center row. The center row was then grouted to refusal, using required split spacings occasionally closer than 5 feet. The reason for this technique was to limit grout travel and to concentrate refusal grouting to within the confines of the upstream and downstream rows. In reviewing the records for grout takes, it appears that this method of construction for a three-row grout curtain served the intended purpose of limiting grout travel.

With the upstream and downstream rows containing holes on 20-foot centers, the three-line grout curtain below the key trench at Teton Dam should be considered to function as a single-line curtain. It has been the practice of other Government agencies to install multi-line curtains in permeable rock. This lessens the possibility of windows in the grout curtain. More importantly, closing all lines increases the effective width of the grout curtain and decreases the hydraulic gradient across the curtain, thus decreasing the possibility of erosion and/or leaching of the grout over long periods of time.

In the areas where one row of holes was used, the rock was thought to be tight enough that excessive grout travel would not occur from closure grouting with one row of holes. The grout records generally substantiated this approach.

For the entire length of the grout curtain, the single row and center row holes were drilled through a concrete grout cap. The grout cap was installed in a 3-foot-wide by 4-foot-deep trench excavated in the rock foundation.

Where three rows of grout holes were used, $CaCl_2$ (calcium chloride) was added to the grout mix used in the upstream and downstream rows for two purposes: (1) to quicken the set time of the grout mix and inhibit long distance grout travel, and (2) to raise the temperature of the grout mix in cold weather. The $CaCl_2$ was not used in single-row grouting or in the center row of the three-line curtains. Between Stas. 11+50 and 16+00, including the area where the failure occurred, grouting was done during the summer months and $CaCl_2$ was not used in significant amounts. Elsewhere, the two outer rows of holes were injected with grout mixes containing up to 8 percent $CaCl_2$ (by weight of cement). The industry normally specifies 2 to 3 percent $CaCl_2$ as the maximum amount that can be used. Whether or not the use of $CaCl_2$ in excess of 2 to 3 percent $CaCl_2$ would cause adverse effects is unknown. The use of $CaCl_2$ in smaller amounts is common for hastening set time, but its use for controlling grout temperature is unknown to the other Government agencies involved in this investigation. The Bureau is currently conducting tests at the request of the IRG to determine if the use of $CaCl_2$ in unprecedented amounts could have affected the integrity of the grout curtain over either the short or long term.

After removal of the embankment remnant from the right abutment key trench, the condition of the grout cap was observed. The cap had been badly battered and partially removed in the area of the failure, but it appeared tight and well constructed where it had been exhumed in the key trench.

A number of postfailure core holes was drilled for water testing within the center line of the grout curtain. Figure 33 shows the arrays of holes drilled between Stas. 12+50 and 14+25. On this figure, postfailure grout curtain permeabilities are indicated by water takes in the core holes. Construction grout takes during construction for the center row of grout holes are also shown on the figure. To evaluate grout curtain permeability, the Grouting Task Group identified water takes ranging from 0.1 gpm/ft to 0.5 gpm/ft as being marginally significant and takes greater than 0.5 gpm/ft as significant. As shown on the figure, several significant takes were recorded near the top of rock.

In addition to the core drilling, 19 joint transmissibility tests were conducted in the bottom of the key trench in the area of the failure between Stas. 12+93 and 13+50. Water was ponded over exposed joints which ranged from 1.2 to 8.0 feet in length, with maximum ponded water depths of from 0.4 to 1.0 foot. Water losses ranged from 0 to 1.1 gpm. Water communication to the surface occurred within a few minutes along fissures outside the ponded areas. These tests should be considered qualitative.

Postfailure testing indicates a shallow zone of moderate permeability in the vicinity of the failure between Stas. 13+00 and 14+00. It is likely that this zone was disturbed during the failure and that observed permeabilities were greater that those which existed before the failure. The IRG considers the seepage quantities observed after the failure to be within tolerable limits and that they should be considered normal for a grout curtain in a permeable rock mass.

Treatment of Rock Foundation Surfaces under Zone 1 Material.–Detailed analyses of foundation preparation under the zone 1 embankment area are presented in Appendixes C and D. Appendix D also discusses foundation preparation under the remaining zones. Because foundation preparation was generally adequate outside the area where zone 1 contacted the foundation, only those portions of the foundation associated with zone 1 are discussed here. The entire question of foundation preparation ties closely with the design concepts and assumptions discussed later in Chapter 7, Adequacy of Design and Construction.

No formal written procedures detailing the surface treatment were developed within either the Bureau's design or field construction organizations. Information concerning surface treatment has been obtained from field records and from task group interviews with design and construction personnel.

Concerns for the protection of pipable zone 1 fill material placed on a highly fractured foundation were expressed by the principal designer in a collection of design notes and draft material dated from March 1967 to January 16, 1970. These concerns were not, however, expressed in the document, "Design Considerations for Teton Dam, October 1971." This document, which was prepared by design personnel to familiarize field construction personnel with important design considerations and to further describe the construction specifications, does not address itself to the details of a surface treatment program outside the area of the bottom of the cutoff and key trenches. In fact, the statement "Erosive seepage under the embankment will be eliminated by injecting the foundation with a grout mix" seems to imply that the "tight grout curtain" was the major defense relied upon by Design to assure that piping was under control.

No provision was made in the contract specifications for general surface treatment of open joints except for blanket grouting which was to be used on "an individual or small area

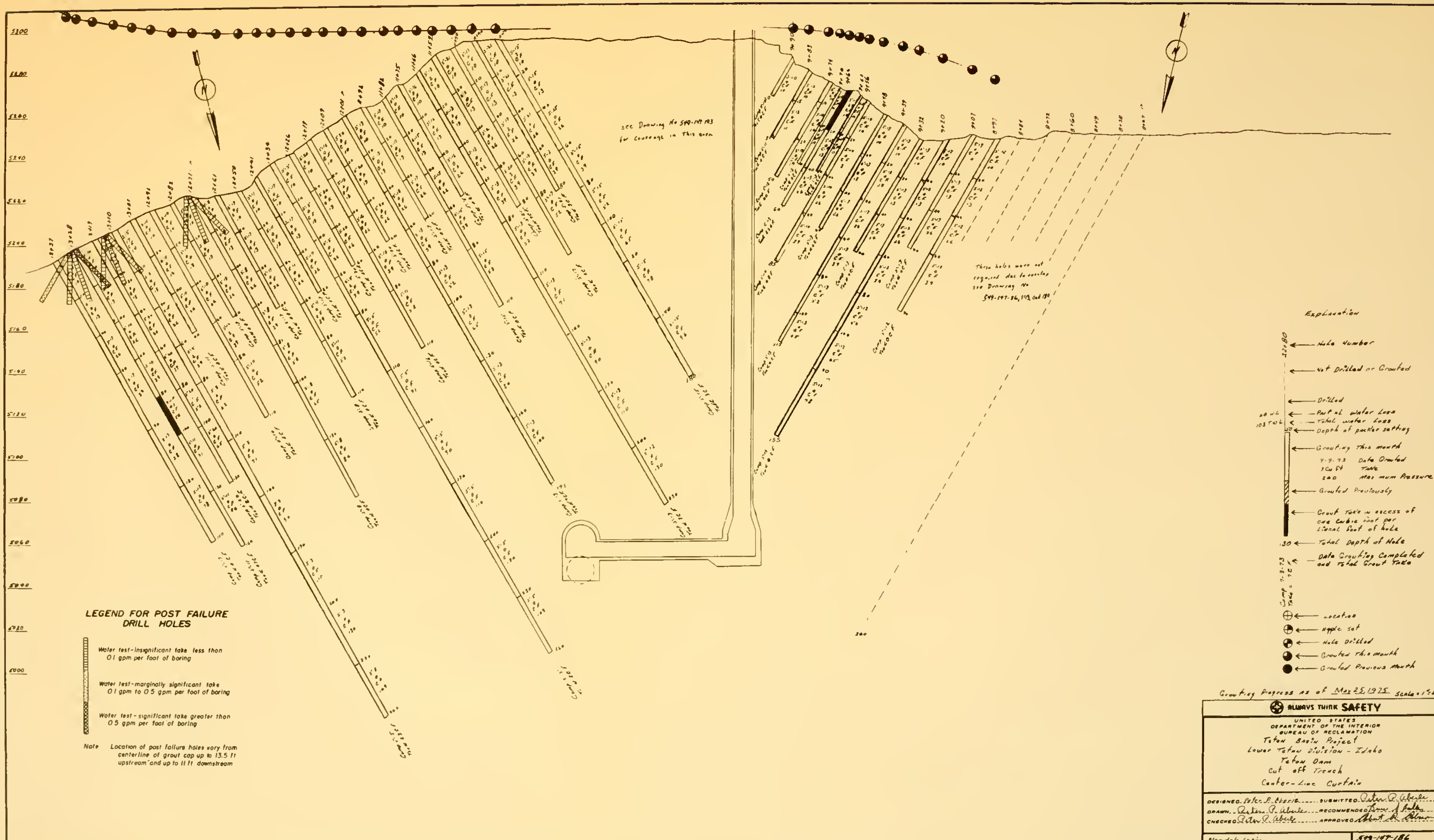


Figure 33.—Details of the right abutment centerline grout curtain including postfailure core borings (sheet 1 of 3).

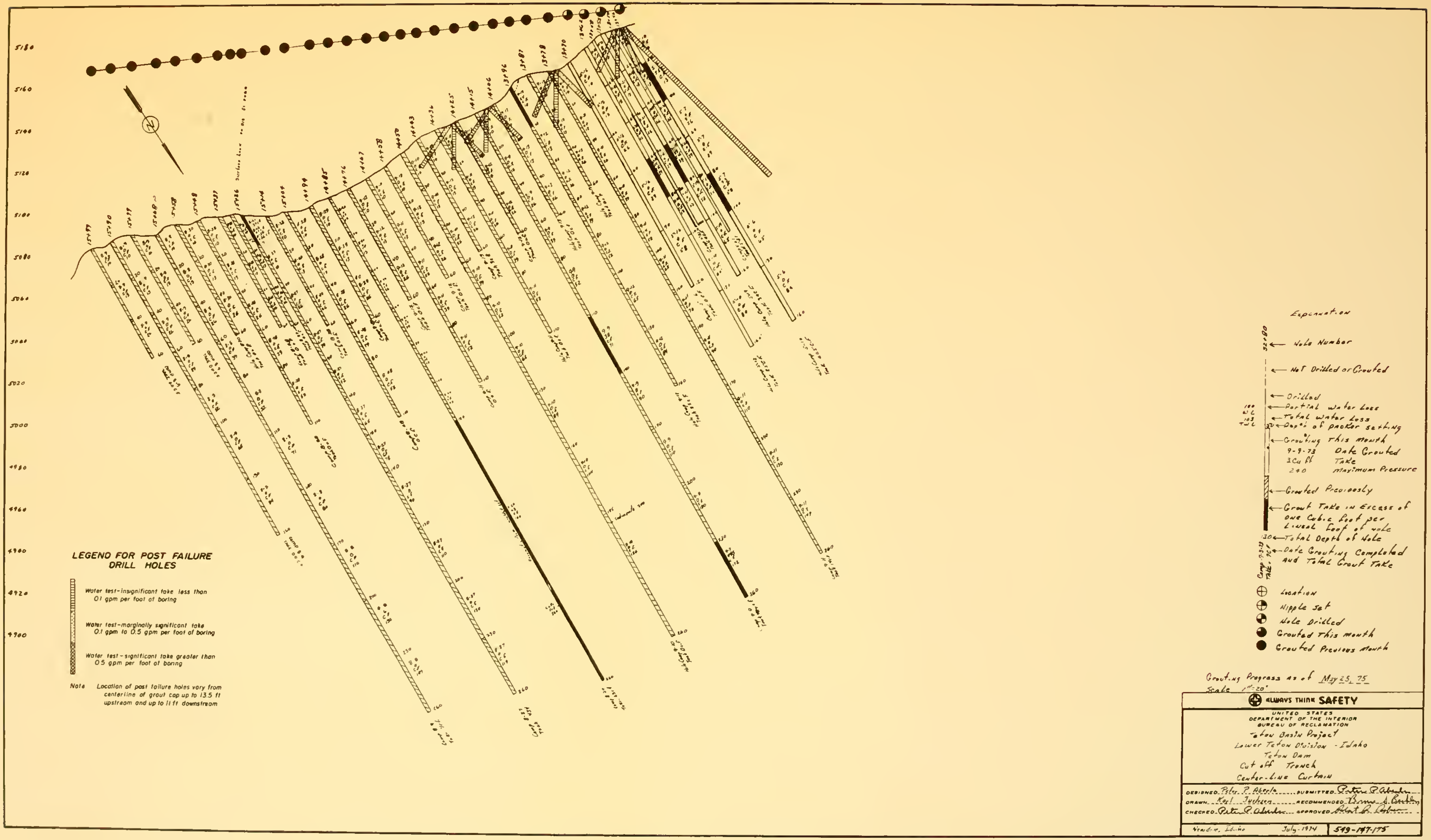


Figure 33.—Details of the right abutment centerline grout curtain including postfailure core borings (sheet 2 of 3).

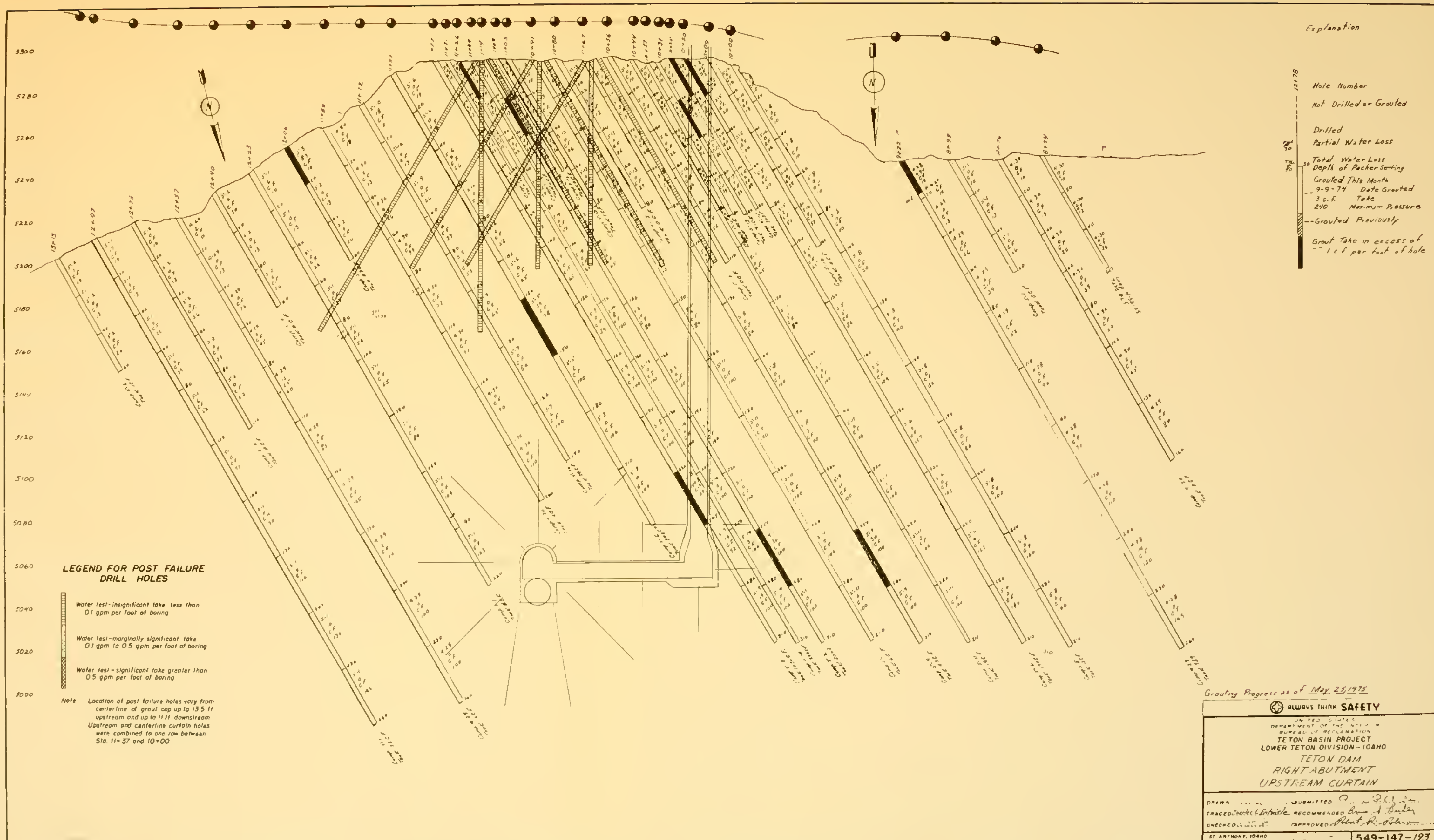


Figure 33.—Details of the right abutment centerline grout curtain including postfailure core borings (sheet 3 of 3).

basis." It was the feeling of the designers that surface grouting should be treated as a field problem to be negotiated with the contractor.

In late 1973 and early 1974, design, geology, and construction liaison personnel visited the site, at the Project Construction Engineer's request, to look at the cutoff and key trench excavations prior to placement of fill. The trip reports, which are presented in Appendix G, are not directed toward the full scope of a surface treatment program. Interviews with Messrs. Bock, Harber, Aberle, and Gebhart (see Appendix C) indicate that during these site visits surface treatment was discussed. After deciding jointly at that time that surface treatment would be done by "bucket grouting," field construction personnel developed a surface treatment procedure, apparently without further detailed design and geological assistance. At one time, a memorandum was prepared by the principal designer stating that consideration was being given to a surface treatment program using shotcrete. However, after telephone conversations with the Teton Project Construction Engineer, it was felt that the studies necessary for the development of a shotcrete program were premature at that time. These communications are apparently the last time shotcreting was considered for surface treatment.

Field construction personnel developed surface treatment procedures to allay their concerns for compaction of zone 1 embankment material over large open joints. The embankment and surface grouting inspectors would designate which cracks were to be filled. The cracks were then filled by surface grouting using a 0.7:1 neat cement grout or a sand-cement grout, depending upon the size of the opening. The surface treatment was performed in a narrow cleaned-up strip of rock just ahead of the advancing fill. Because of the stiff grout mixes used, and because the concerns of construction personnel were generally for compaction control and not the percolation of water next to the pipable zone 1 material, joints and cracks less than 1/4 to 1/2 inch in width were ignored. In the right abutment, surface grouting was performed under zone 1 material from El. 5075 to 5205.

Figure 34 shows the location of features which were surface-grouted and the amount of grout poured into each. This information is superimposed on the geologic map of the abutment drawn during construction. The total surface treatment operation on both abutments used somewhat over 1,800 cubic yards of poured grout, 1,325 cubic yards of which were used in the right abutment.

It was also the intent of design personnel that surface grouting be done in the side walls of the key trench. The field construction inspectors have indicated that very little surface grouting was done in the side walls of the key trench. However, inspection of Figure 34 shows that there was some surface grouting in the side walls. Postfailure inspection of the key trench revealed a number of ungrouted joints and cracks, some open as much as 6 inches. Most of the area outside the key trench, where surface treatment was done on the right abutment, was affected by the failure. In situ rock up to 70 feet in depth was stripped by the flood. As a result, postfailure observations of as-built conditions were not possible.

Surface grouting stopped at El. 5205. Neither the designers nor the liaison engineer were aware of the decision to stop surface grouting until after the failure of Teton Dam. According to field personnel, the field geologists also played no part in the decision to stop surface grouting.

Conflicting reports as to the openness of the rock surface above El. 5205 have been presented to the IRG. According to the field engineer, above El. 5205 the rock was more slabby. While there were no large fissures riverward of the spillway, there were "hundreds" of 1/4- to 1/2-inch-wide fissures which were left "untreated." One of the shift supervisors stated that there did not appear to be a change in the fracturing characteristics above El. 5205, and that above that elevation no attempt was made to remove native soil from cracks to replace it with zone 1 material. He also said the native crack filling material appeared moist and was probably a silt of low plasticity similar to zone 1 material.

Analysis of Postfailure Embankment Remnants

The portion of the embankment of prime interest was destroyed by the failure of the dam. The remaining embankment remnants outside the breach area were investigated after the failure because they were representative of the failure area.

Investigation of Right Abutment Remnant.–Exploratory work was performed to determine the physical characteristics of the remnant embankment and to allow examination of the embankment and rock surfaces for evidence of unusual conditions. This work was done under the direction of representatives of the Independent Panel. Pertinent descriptions of the conditions encountered, along with illustrations, are extracted from the Independent Panel Report and are included as Appendix H. The IRG and/or task group members observed conditions at several stages of the work.

The embankment remnant was carefully removed, using hand excavaton, supplemented by small mechanical equipment. As removal took place, notes, supplemented by photographs, were made of the embankment conditions, such as fracturing, unusual moisture conditions, and contact of the core and filters with bedrock. Undisturbed cube samples of the core material were taken for laboratory testing. Numerous in-place density tests were made, as were tests for in situ moisture. When all embankment material had been removed, the entire key trench was washed clean with water. Areas on the lower slope adjacent to and underneath the key trench that had been deeply eroded during failure were also washed clean.

Laboratory tests on samples from the right abutment embankment remnant were made to confirm material properties used in the design, to determine variability of as-constructed physical properties, and to determine stress-deformation properties for in situ stress analyses. Ninety-two undisturbed, hand-cut 9-inch cube samples, and 47 Shelby tube drive samples, 3 inches in diameter by 36 inches long, together with 10-pound bag samples were obtained at locations shown on Figure 35. Selected block samples were sent to various laboratories for testing.

The dispersive characteristics of zone 1 material were investigated by pinhole dispersion tests at the U.S. Army Engineer Waterways Experiment Station. The erodibility characteristics were investigated by flume tests and rotating cylinder tests at the University of California at Davis.

The stress-strain properties of the zone 1 material were investigated by drained triaxial compression tests at both placement and saturated moisture contents by Northern Testing Laboratories, Billings, Montana, and by the Bureau of Reclamation laboratories in Denver. The results are summarized in Table 3. Unconfined compression tests at varying moisture contents were also made by the Bureau of Reclamation.

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

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. These results are summarized in Table 4.

The excavation and examination of zone 1 material on the right abutment showed nothing that could definitely be interpreted as a clue to the failure. In general, the fill in the key trench appeared to be well constructed.

At El. 5265, near-vertical cracks roughly paralleling the breach face were encountered in the fill approximately 12 feet from the breach face. The cracks were as much as 2 inches wide and some were filled with water-borne sediment. A crack and sheared zone adjacent to the breach face extended to the bottom of the key trench as the excavation progressed. It was concluded that this disturbance was the result of postfailure movement; however, it did demonstrate the cracking potential of the fill.

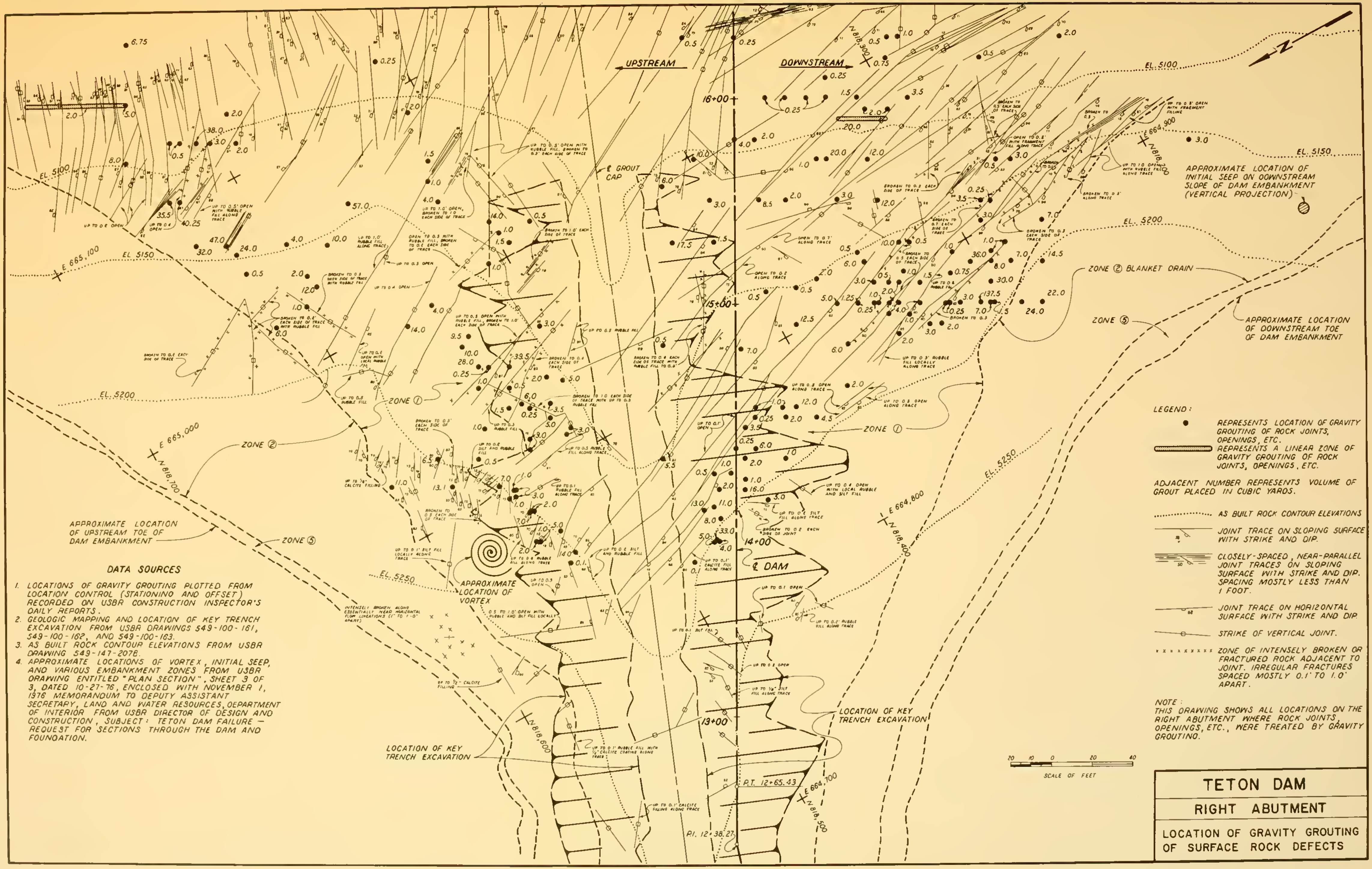


Figure 34.—Location of gravity grouting of surface rock defects.

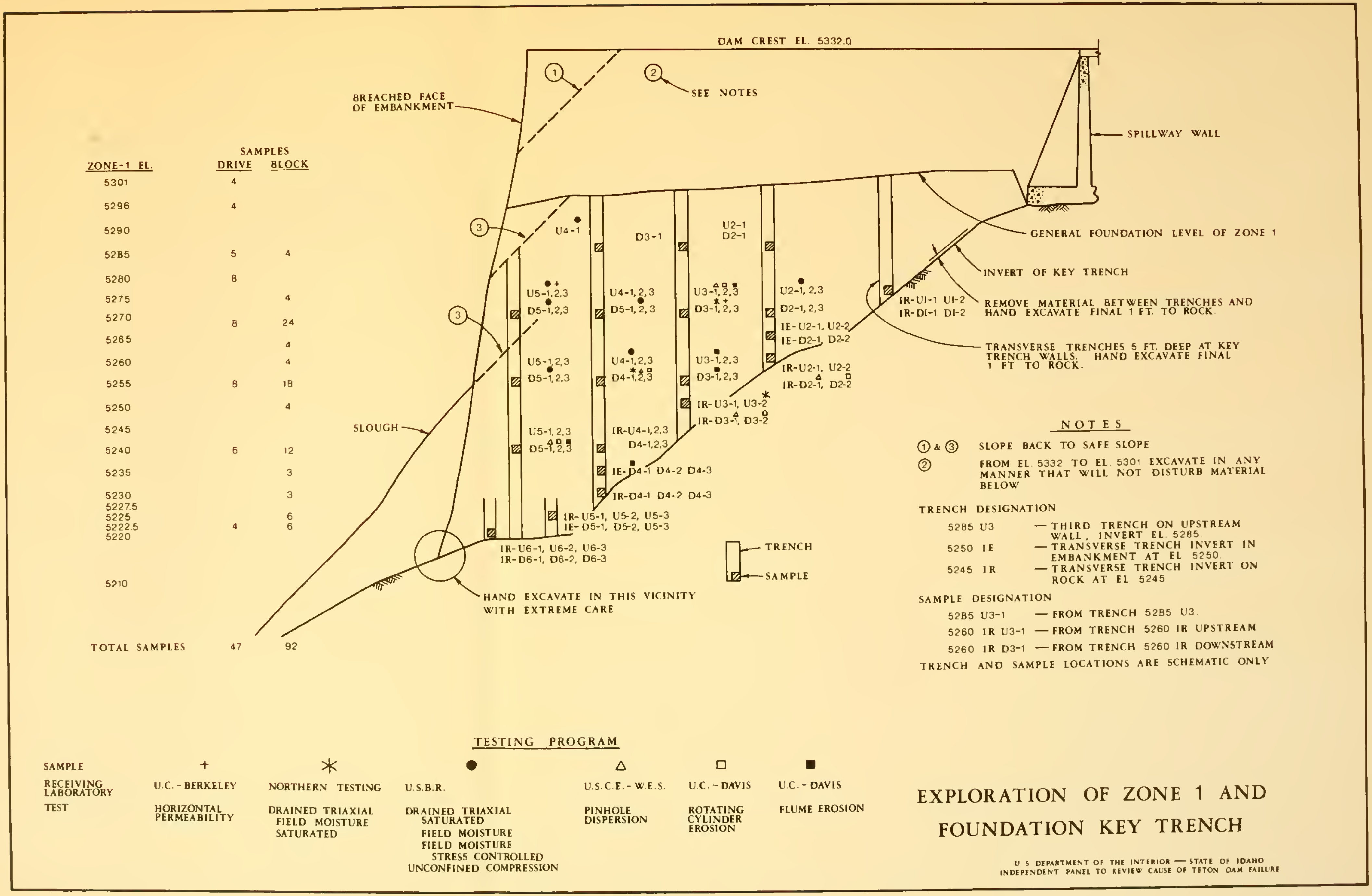


Figure 35.—Exploration of zone 1 and foundation key trench.

Table 3.–*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 ε (in./in.)	K_o E (psi)	K_o μ	USBR Exhibit (Appendix A)
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 reproduced from the Independent Panel Report on "Failure of Teton Dam," December 1976, table 7-2, p. 7-11.

Table 4.—*Summary of classification test data, samples from remnant of key-trench fill right abutment*†

Property	Mean	Approx. Std. Deviation	Usual Range
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
† Table reproduced from the Independent Panel Report on "Failure of Teton Dam," December 1976, table 7-4, p. 7-13

Zone 1 compaction at the contact with foundation rock surfaces was generally equivalent to that within the body of the fill. Joints and fractures in the key trench walls were numerous, with openings up to 6 inches wide. There was little evidence of surface grouting in joints exposed in the key trench walls.

The laboratory tests, in addition to providing input parameters for the finite element analysis, confirmed the highly erodible nature of the zone 1 material and its brittle characteristics when compacted dry of optimum moisture content.

Investigation of Left Abutment and Embankment Remnant.–There is much similarity between conditions on the right and left abutments. Each had a key trench having the same configuration and dimension, and embankment zoning was identical. The bedrock on the left abutment is reported to be somewhat less jointed and fractured than on the right abutment. Thus, conditions in the left abutment can be considered nearly the same as those in the right abutment. After failure of the dam, it was noted that a number of transverse cracks were present in the crest of the remaining left embankment. These cracks were mapped. Limited investigations have been made and extensive future examination is planned.

Because of the similarity of the two abutments, in situ stress measurements were considered desirable to evaluate the role of cracking and/or hydraulic fracturing in the zone 1 material during failure. An attempt was made to make direct measurements of the in situ state of stress by using an experimental self-boring pressure meter, PAFSOR, being evaluated by the California Department of Transportation under a contract with the Federal Highway Administration. The PAFSOR (Pressiometre Auto Foreur Sols Sous on Raide) is a French device designed to be used in medium stiff to stiff soils. The technique of self-boring consists of inserting a thin-walled metallic cylinder with an expandable membrane into the ground with minimal disturbance of the soil surrounding the hole. The investigation at the site was conducted by the California Department of Transportation under arrangements with the Bureau of Reclamation. The results were furnished in a report to the Bureau, dated January 21, 1977, titled "In Situ Stress Measurement at Teton Dam, Idaho, Using the French Probe PAFSOR."

The investigation was not successful in providing meaningful data on in situ stresses at the locations of interest. Equipment difficulties limited the depth of probe penetration to 34 feet below the crest of the dam. Tests were made at 24 and 34 feet, but the results at these shallow depths are considered insignificant in evaluating stress conditions near the base of the embankment.

At the request of the Independent Panel, hydraulic fracturing tests were performed at Stas. 26+00, 26+25, and 27+00 to determine the hydraulic head that would cause fracturing of zone 1 materials within the key trench. Each test consisted of drilling a large-diameter hole to the depth to be tested, setting a small-diameter casing at this depth and casting a concrete seal around its base. Then, the test section was drilled as a 3-inch hole through the casing below the concrete seal. Testing consisted of applying various heads of water and measuring the seepage rate at each head. Before fracturing, the seepage rates were approximately proportional to applied head; after fracture, the rate of seepage increased rapidly.

The details of the tests and the results are given in Appendix I. Results were somewhat erratic, partly because of installation difficulties. At Sta. 26+00, fracturing occurred during drilling at a depth and head of 101.3 feet. At Sta. 26+25, the installation was unsuccessful and a test was not made. A test run at Sta. 27+00 did not indicate hydraulic fracturing under an average head at 119 feet (to the top of the hole).

Another test was run at Sta. 26+00. The results indicated hydraulic fracture at a head ot approximately 145 feet (see Appendix I).

Finite Element Analysis of the Embankment.–The IRG felt it was necessary to investigate in situ stress conditions in the embankment. In the first IRG interim report dated July 14, 1976, investigations were requested to evaluate in situ stress conditions with finite element analysis, to perform water pressure testing in the remaining left embankment, and to make in situ pressure measurements in the left embankment. In accordance with agreements to avoid duplication of investigations being carried on by the Independent Panel, the IRG deferred work on the finite element analysis of the embankment and water pressure tests in the left embankment until results of the Independent Panel's investigations were available. The results of the Independent Panel finite element investigation are included as Appendix D in its report. This information is included herein as a portion of Appendix I.

The finite element studies clearly show the possibility of hydraulic fracturing in the key trench. These studies cannot be considered conclusive since the input data are subject to considerable variation. Further, the results are based on a two-dimensional analysis, while the complex geometry of the abutment and key trench suggests considerable influence by the stress distribution along the dam axis.

Foundation Deformation

The hypothesis of failure advanced by Curry involved differential settlement of the bedrock foundation, with resultant cracking in the dam. This hypothesis has been discussed previously in the Geologic Studies portion of this chapter. To obtain qualitative insight into the magnitude of foundation deformations, a finite element analysis was made to model the effect of the load of the dam and reservoir on the rock foundation. These studies indicate a maximum settlement at the base of the dam of about 1/4 foot, with a uniformly distributed differential settlement between the base and top of the abutments ranging from 0.10 to 0.15 foot. No large local differential settlements were found. Rebound on the order of 0.1 foot is indicated for the postfailure condition. These studies were consistent with earlier opinions that the magnitude of settlement in the rock foundation was well within that experienced by many projects, and that foundation deformation was not responsible for cracking within the embankment.

To further investigate the foundation movement hypothesis, a study was made of permanent movement that might have taken place in the dam area. This study constituted a resurvey of all available points that could be compared with known prefailure locations, both at the damsite and in the immediate area. Excavation of the right abutment remnant provided access to bench marks set on the grout cap. Of six readings, the maximum vertical movement was 0.021 foot downward and the average was 0.008 foot downward.

The investigation of area movement involved remeasurement and releveling of monuments adjacent to the dam, referencing them to an established bench mark several miles away. Both trilateration and leveling showed no systematic movement. Within instrument accuracy, only two monuments showed movement–one attributable to construction disturbance and the other attributable to local sliding of the reservoir bank during rapid drawdown.

These studies of foundation movement indicate nothing that can be interpreted as lending credence to the theory that at-depth bedrock behavior was instrumental in the failure.

Rate of Reservoir Filling

The impact of the rate of filling on the failure cannot be analyzed. It is conceivable that a more rapid increase in load from the reservoir could induce cracking in a core which might deform without cracking under slower application of stress. The difference in rate between 1 foot per day and the maximum of 4.3 feet per day is not believed to have affected Teton Dam.

Adequacy of Design and Construction

Design

Early in its investigation, the IRG noted some design features that were judged to be pertinent to the failure. An investigation was made to determine the design concepts that led to those design features. The Bureau of Reclamation was provided with a list of questions relating to many of the design features. The Independent Panel also submitted questions concerning the design of the project. Both sets of questions and the responses are included in Appendix G. Other information on some of the design considerations is included in the task group findings appended to this report. The IRG has also gained insight into some of the design concepts from discussions with Bureau of Reclamation personnel during the working sessions.

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

Obviously, the plans and specifications issued for construction received the concurrence of technical, supervisory, and management personnel. However, because of the lack of documented rationale to bridge the gap between early design notes and the final design, it is not clear to what extent these personnel were involved during the design process.

Most of the problems related to the failure can be associated with basic design assumptions and procedures. Design procedures will be the focus of a forthcoming IRG investigation.

General Design of Embankment.–Design of the embankment relied heavily on past Bureau experience at other dams. At Teton Dam, the development of new information to establish design parameters for embankment materials was minimal. For example, during the design only two laboratory shear strength tests were performed on zone 1 embankment material and no permeability tests were performed on zone 2 material.

Even with this minimum amount of test information, the IRG agrees that a safe dam was designed from the point of view of its structural stability. As discussed below, the problems that led to the failure were not associated with the structural stability of the dam.

The impervious core was constructed of a wind-blown silt of fairly uniform grain size. Of paramount importance was the protection of this silt from piping and erosion. This protection had to be provided to prevent migration of silt particles as water percolated through it (piping) and next to it (erosion). Safe dams have been constructed using this material, and the basic concepts for a defensive design were well within the state-of-the-art during the design of Teton Dam.

The protection of cohesionless, or nearly cohesionless, core material is usually provided by means of impermeable barriers and/or filters. Rock surfaces are sealed by treating them with concrete, blanket grouting, dental concrete, surface grouting, etc. Filters allow free drainage of seepage but prevent migration of the material to be protected as water seeps through it. Specific guidelines have been developed by the industry for the design of filters based on theoretical analyses, laboratory tests, and experience.

At Teton Dam a defensive design for adequate protection of zone 1 embankment material was not provided. This is perhaps the single most important finding of the IRG investigation.

The designers appear to have recognized from the beginning that zone 1 must be protected from erosive seepage through the permeable rock foundation. However, except for the cutoff and key trench bottoms, adequate sealing of rock surfaces under zone 1 was not provided. The only rock treatment specified was blanket grouting to be used in the bottom of the cutoff and key trenches and in small localized areas identified during construction. The designer assumed that specified compaction of zone 1 material would achieve a degree of resistance that could prevent its migration into openings in the foundation as seepage passed through it. The IRG disagrees with these design concepts.

It is doubtful that special compaction of any kind could substantially inhibit migration of silt particles into open foundation cracks. In addition, the basic function of a grout curtain is to reduce seepage under a dam to tolerable limits. This serves to enhance embankment stability and to inhibit the loss of valuable reservoir water through the foundation. A grout curtain should never be considered capable of eliminating seepage through the foundation. Seepage in varying quantities will exist along the entire length of a grout curtain, especially in a highly jointed foundation such as that at Teton Dam. Whether or not a grout curtain is used, the embankment must be protected from this seepage to the degree dictated by the properties of the embankment materials and the foundation.

The details of the rock surface treatment program, beyond local blanket grouting, were handled informally after construction began. The responsibility for developing the scope and details of the surface treatment program was left to the field construction personnel, with no written instructions from design personnel concerning basic design concepts. Such written instructions would have been necessary to ensure a full understanding that surface treatment was fundamental to the ultimate integrity of the dam.

Surface treatment was discontinuous. Where treatment was used, it did not tie into the grout curtain, and it was not applied to a significant abutment area above El. 5205.

Without a proper defensive design, the integrity of zone 1 was in jeopardy from percolation of water through it and along its contact with the rock foundation. The design and construction of the defensive measures necessary to protect zone 1 were well within the state-of-the-art at the time Teton Dam was constructed.

Zone 2.–The zone 2 was included to protect zone 1 from the effects of throughgoing seepage. No laboratory permeability tests were performed on zone 2 material during design. The designers assumed that this material would have sufficient water-carrying capacity to handle all normal seepage passing through zone 1 and also prevent zone 3 from becoming saturated. Zone 2 was not designed for, and could not handle, the large flows that were associated with the initiation of the failure. The designers judged that because of the grain size of the material and the width of zone 2, adequate drainage capacity and filtration for normal seepage quantities would be provided.

Tests performed on zone 2 material after completion of the design are presented on Figures 36 and 37. Figure 36 shows the range of gradation curves for zone 1 material; limits of gradation for zone 2 as required by Corps of Engineers, Soil Conservation Service, and Bureau of Reclamation design standards; and the

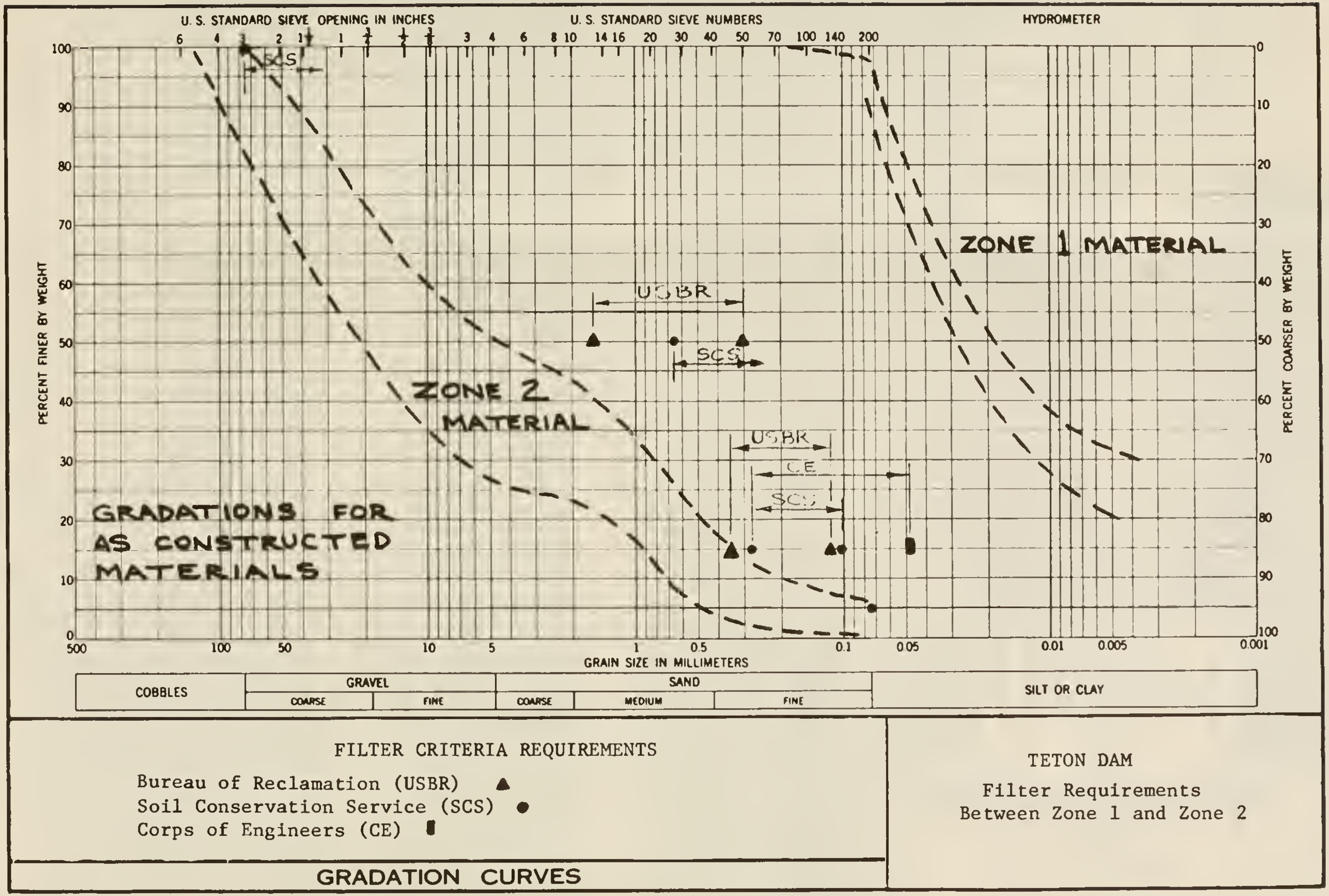


Figure 36.—Filter requirements between zone 1 and zone 2.

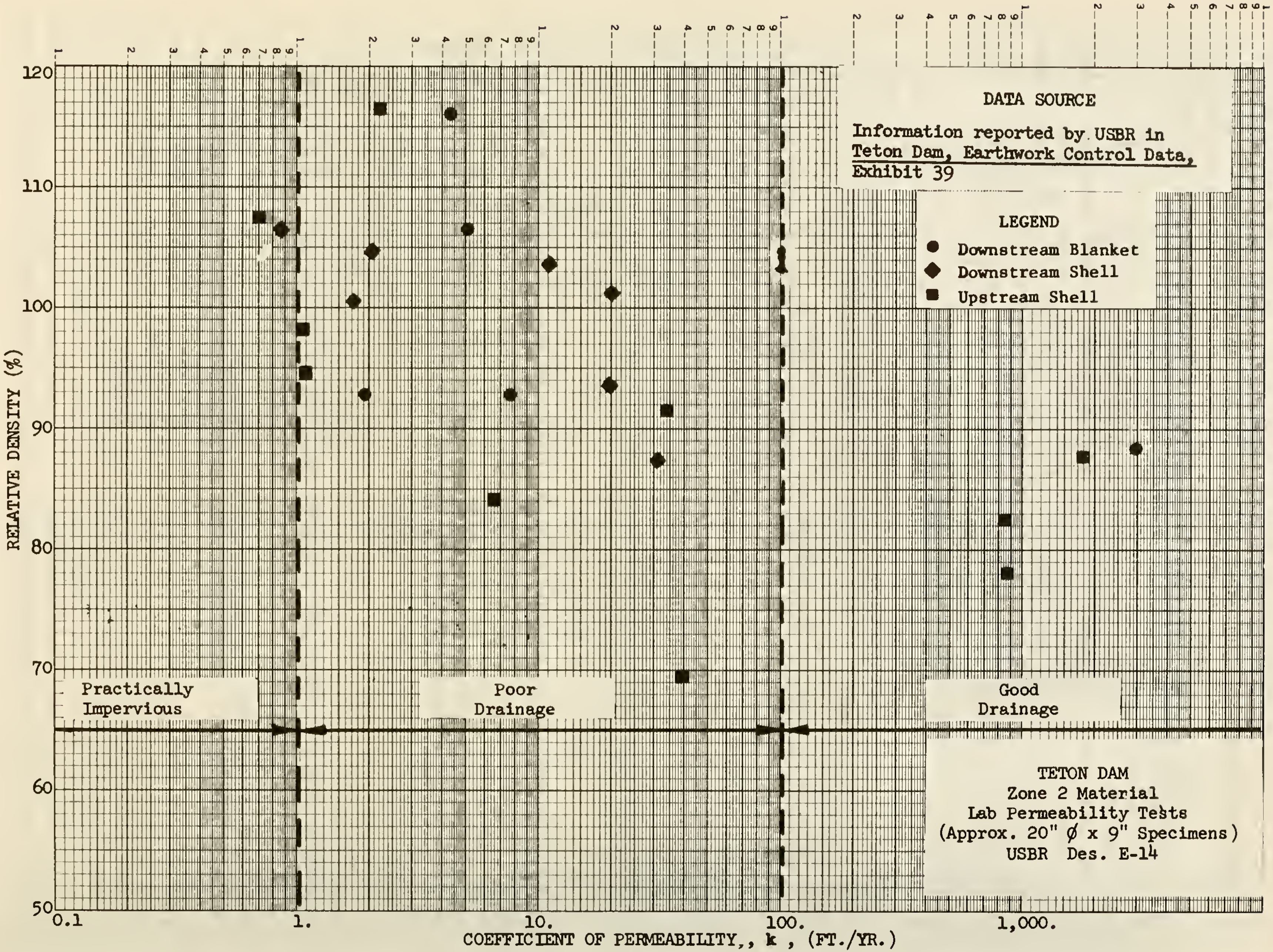


Figure 37.—Zone 2 material laboratory permeability tests.

range of as-constructed gradation curves for zone 2 material. Figure 37 shows the results of permeability tests on zone 2 material. Permeabilities are grouped into three categories ranging from "practically impervious" to "good drainage."

Test results generally indicate (1) that zone 2 was too coarse to meet design standards for filtration of zone 1, and (2) that zone 2 did not provide free drainage. Zone 2 was probably capable of functioning according to the intent of the designers, but only marginally so.

Design of Key Trenches.–It was recognized early in the design that the silty core material had a high potential for cracking. It was also recognized that cracking would be aggravated by the geometry of the foundation. The following are direct quotes from design notes by the principal designer:

> *March 1967*–"A wide flat sloped cutoff trench should be provided across the valley to minimize cracking of the core from differential settlement."
>
> " . . . it undoubtedly also has the negative characteristics of a silt, namely low resistance to erosion, susceptibility to cracking . . . "

The design of the key trench came later, during the period for which no design documentation is available. A profile of the right key trench is shown on Figure 38.

Four observations can be made:

(1) Sufficient information was available within the Bureau to understand, at least quantitatively, the impact of abutment geometry on stress distribution and cracking potential within the core. Technical information on the general subject was also readily available in the literature.

(2) The abutments and key trenches included steep side slopes.

(3) The deep, narrow key trenches and abutment slope changes tended to aggravate the cracking problem. The design of the key trench did not take this into consideration. The ability to minimize the impact of the foundation geometry on cracking in the core was within the state-of-the-art at the time of the design.

(4) That the narrow key trench created a steep hydraulic gradient that was not adequately provided for in the design.

Design Review.–The design was not subjected to an independent review. The IRG feels that such reviews are warranted.

Instrumentation.–The designers judged that their experiences at other dams were sufficient to adequately predict the performance of Teton Dam and that installation of instruments to measure foundation and embankment settlement, lateral movements, and piezometric pressures was unnecessary.

In the opinion of the IRG, if state-of-the-art instrumentation had been installed it would not have been capable of predicting this failure.

Construction

Investigations by the IRG and the task groups have found that in all respects, with the possible exception of inconsequential scheduling problems, Teton Dam was constructed in accordance with the intent of the designers and in agreement with the plans and specifications. The field organization, under the direction of the Project Construction Engineer, was well staffed, well organized, and functionally efficient. The field laboratory was adequately equipped and the scope and quality of tests performed were within industry standards.

Of major concern to the IRG is the level of active participation by designers during the construction phase of the project. The designers visited the site at two critical construction stages. They were satisfied that field conditions were compatible with design assumptions. Considerable written information, including weekly and monthly progress reports, was transmitted from Construction to Design. However, there is little documentation of the flow of information from Design to Construction during the construction period.

A liaison engineer acted to some degree as a line of communication between Design and Construction. However, from early in 1972 until the failure in 1976, he made only six visits to the site. The last documented visit was in March 1974. During the same period, only two visits were made by the design engineers.

The IRG believes that designers should make frequent field visits to prove to their satisfaction that the assumptions made during design sufficiently portray actual field conditions. Such trips also give the designer the valuable experience necessary to make his next design and specification technically and economically more competent than the last.

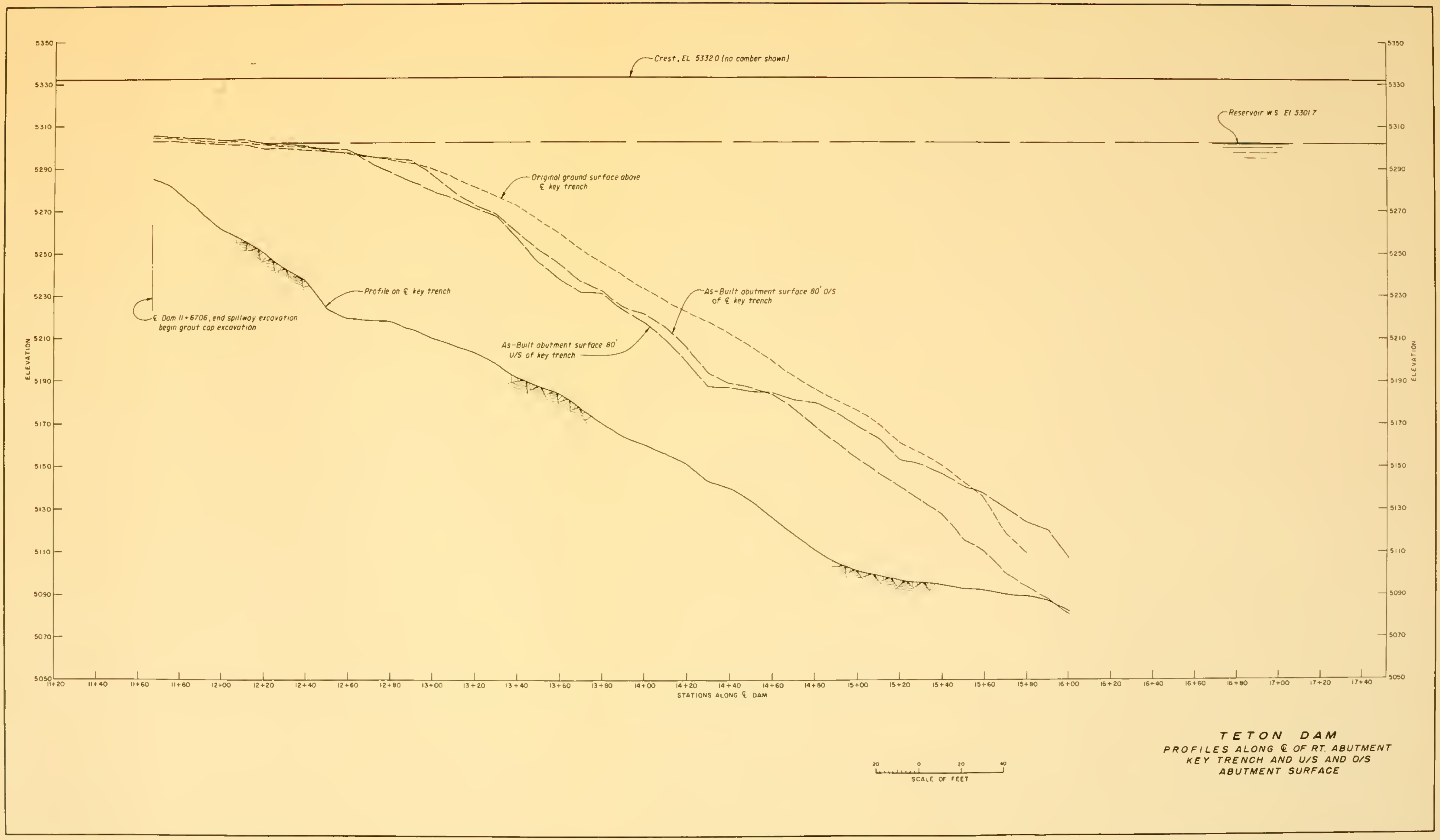


Figure 38.—Profiles along ℄ of Rt. abutment key trench and U/S and D/S abutment surfaces.

Physical Cause of Failure

Unsupportable Causes of Failure

After evaluating all available information, the IRG judges that the hypotheses of Curry and Corbett, foundation cracking due to foundation settlement and cracking due to hydraulic uplift, are not supported by the physical conditions that existed prior to and during the failure.

The hypotheses dealing with flow bypassing the grout curtain are unlikely. There is no evidence available to indicate flows under or around the grout curtain would have been enough to cause the rapid erosion of large amounts of zone 1 materials needed to result in the rapid breaching of the dam.

The Independent Panel has concluded that the initial seepage that led to eventual failure could well have occurred through the grout curtain. The bases for their conclusion are the results of the joint ponding tests and the results of water pressure tests in the postfailure borings. The IRG recognizes the possibility that these measured permeabilities may have been caused by the failure-induced loosening of the rock mass. As a consequence, the IRG believes that failure due to initial seepage through the grout curtain is much less probable than failure due to initial seepage through cracks in the zone 1 material or to initial seepage along the contact between the zone 1 material and the rock foundation.

Probable Causes of Failure

The IRG concludes that the geometry of the abutment and key trench, coupled with the physical properties of the constructed zone 1, were conducive to cracking because of differential settlement or because of hydraulic fracturing. The IRG believes that one, or both, of these modes of cracking occurred, allowing the initial seepage to pass through the zone 1 material. Water under full reservoir head had access to the upstream end of these fractures through open joints in the rock mass. These joints had a sufficient volume of voids so that the initially eroded zone 1 material could have been carried downstream.

Somewhat less probable is the concept that the damaging seepage started at the contact between the zone 1 (impervious core) material and the rock surface. The IRG believes that if zone 1 cracking had not occurred, failure would have been initiated by seepage between zone 1 and the foundation.

"Hydraulic Fracturing and Its Possible Role in the Teton Dam Failure," included in Appendix I, provides an excellent discussion of hydraulic fracturing and its possible role in the failure of the dam. After demonstrating that fracturing could have occurred in the zone 1 key trench fill, the authors of this paper developed a scenario of the progression of piping leading to the failure. This scenario 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. 39[1] 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. 40).

"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 Sta. 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 Figure 41(a), with the seepage flowing under 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 Figure 41(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 Figure 41(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 Figure 41(d).

"Beyond this point the progressive formation of sinkholes, both upstream and downstream, as illustrated in Figure 41(e), provided an ever-accelerating mechanism for internal erosion, finally leading to complete breaching of the dam as illustrated in Figure 41(f)."

The IRG believes that the scenario of the development of failure is logical and consistent with all of the available information and photographs of the failure.

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[1] Illustrations are identical to those appearing in the Independeot Panel Report; however, the figure numbers have been changed.

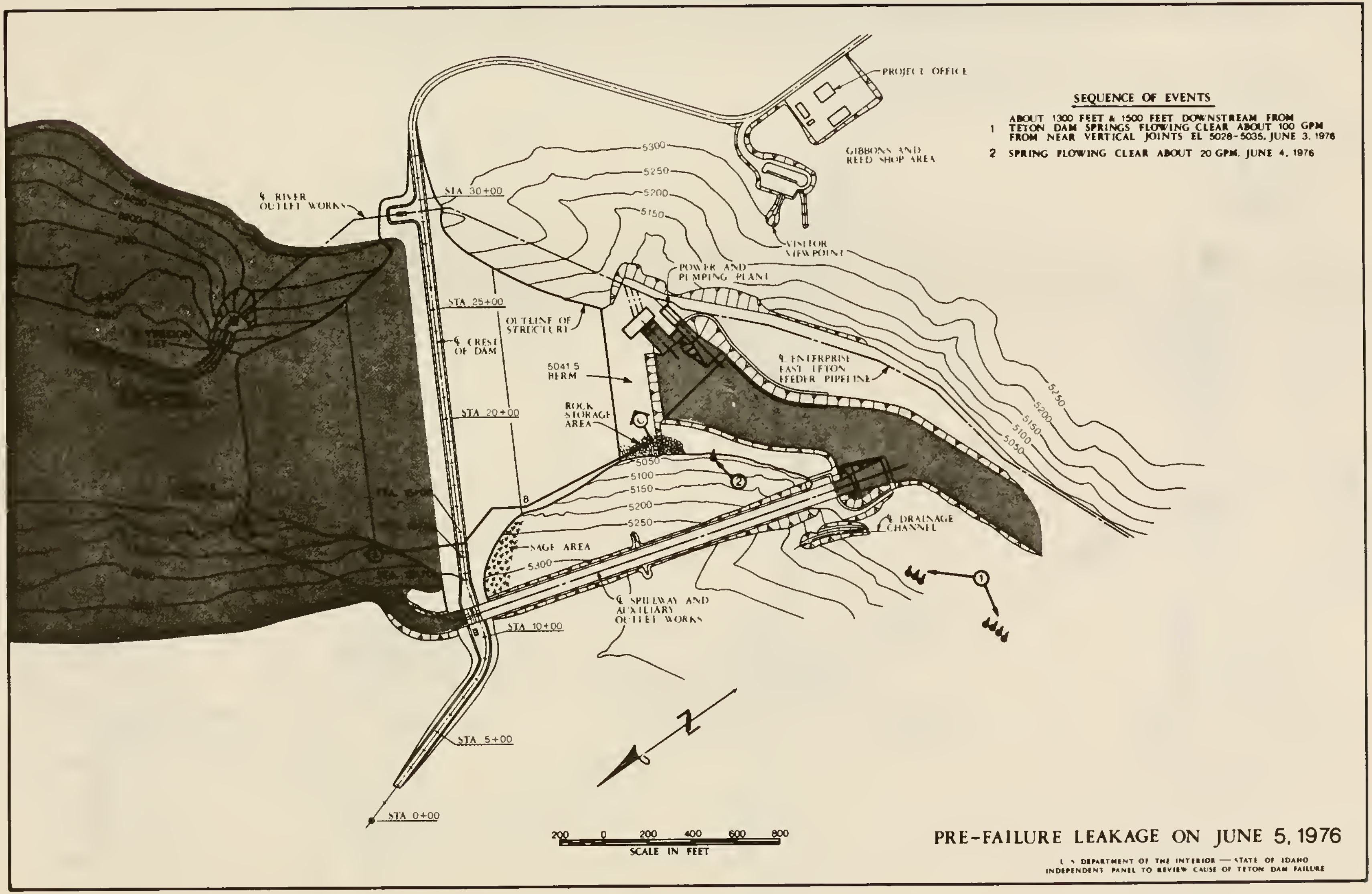
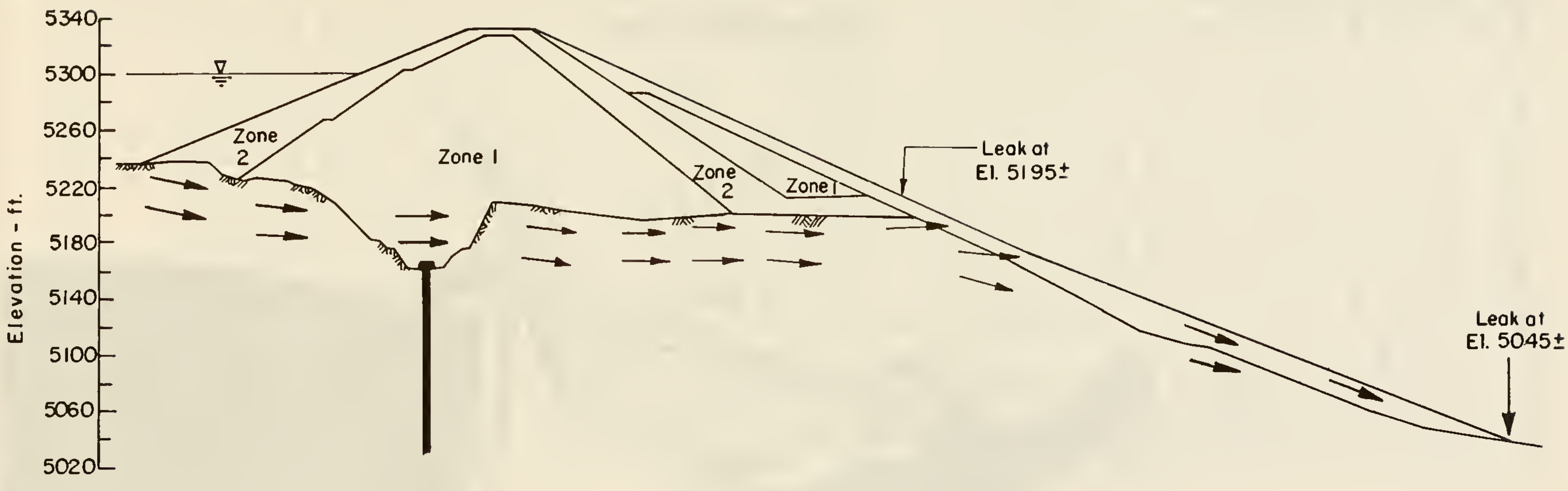


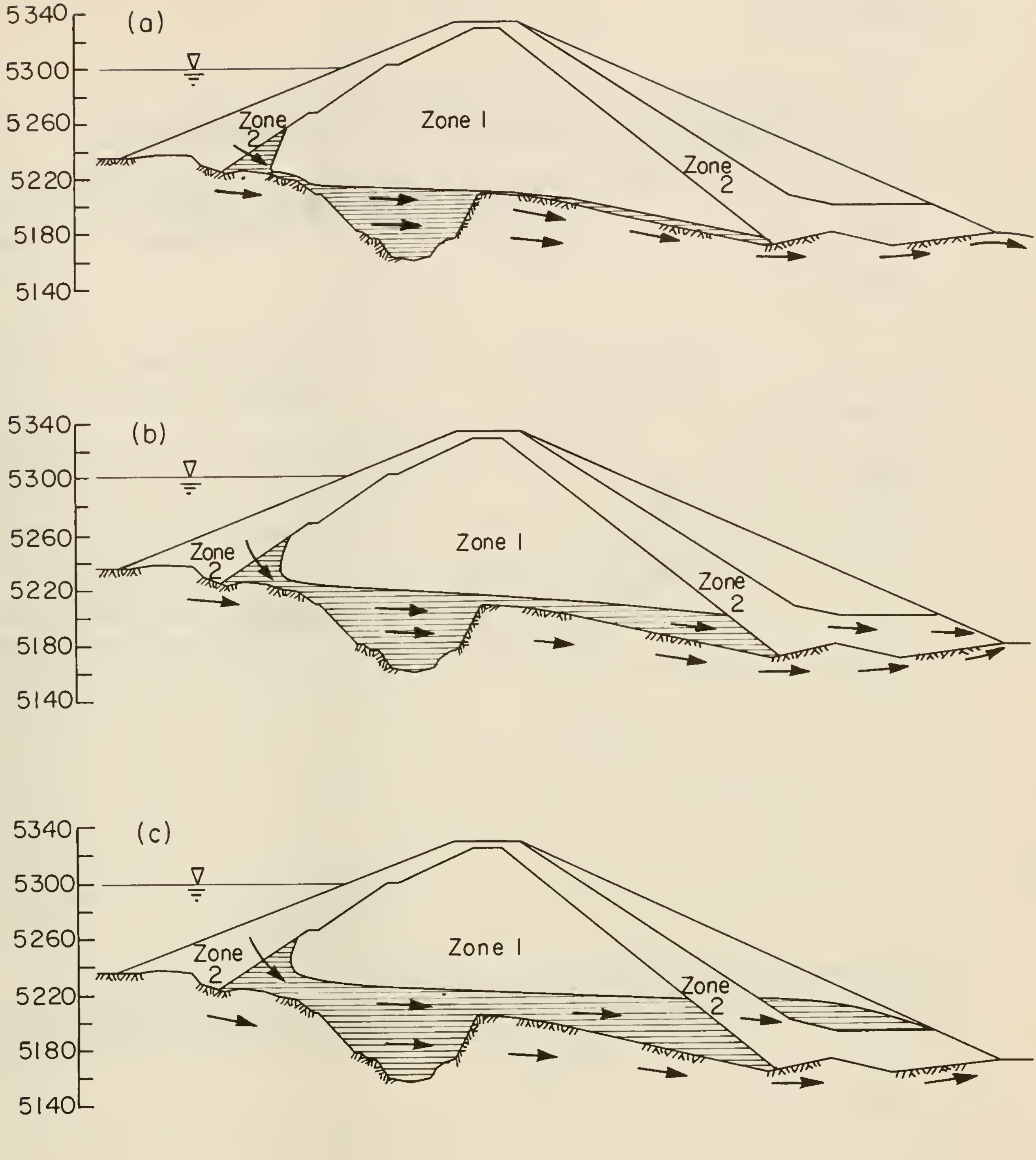
Figure 39.—Pre-failure leakage June 5, 1976.



PROBABLE PATH OF WATER IN EARLY STAGES OF LEAKAGE

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

Figure 40.–Probable path of water in early stages of leakage.



CONCEPTUAL MECHANISM OF PROGRESSIVE FAILURE ALONG SECTION A-B-C

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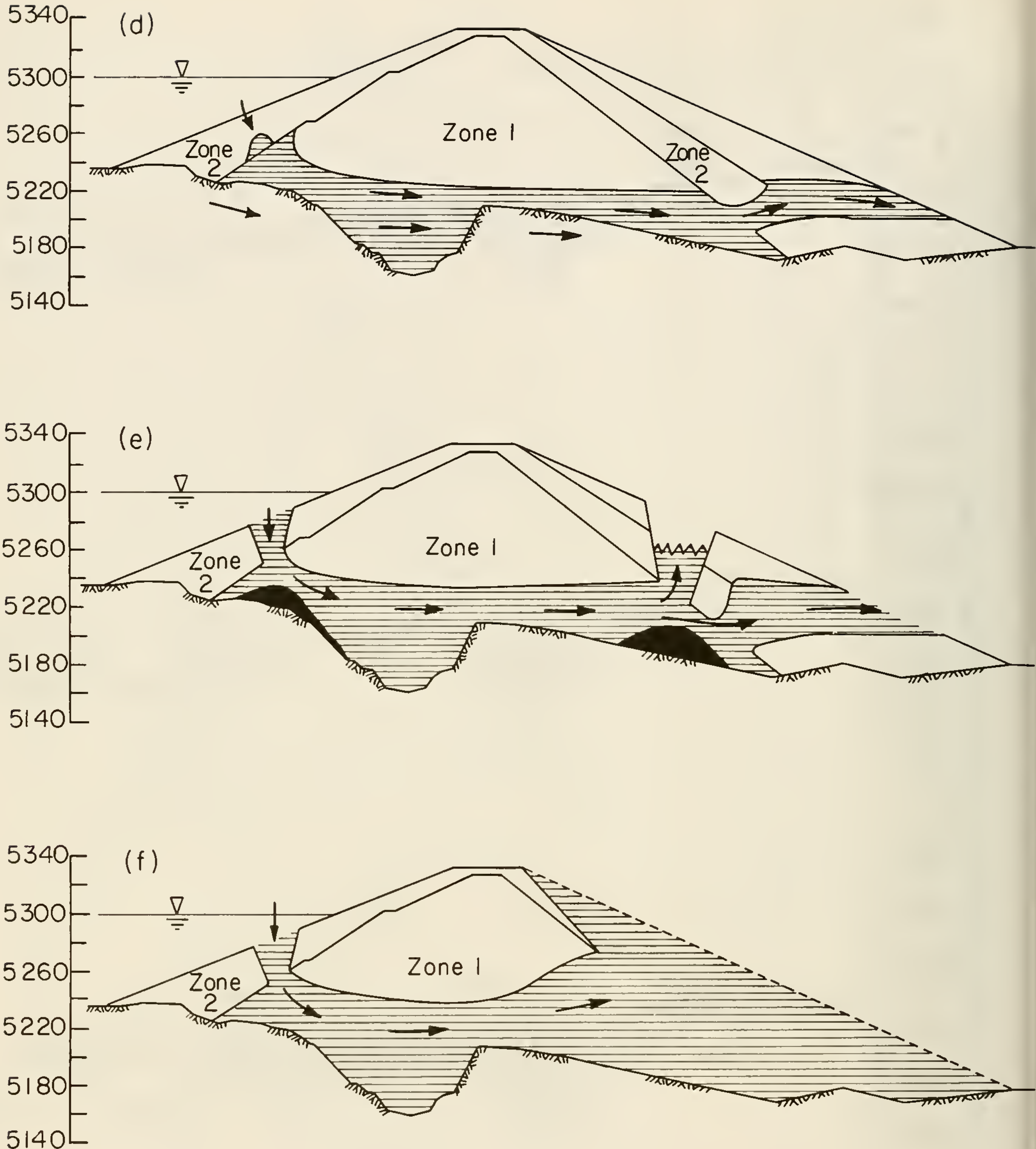
Figure 41.—Conceptual mechanism of progressive failure along section A-B-C (sheet 1 of 2).



CONCEPTUAL MECHANISM OF PROGRESSIVE FAILURE ALONG SECTION A-B-C

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

Figure 41.–Conceptual mechanism of progressive failure along section A-B-C (sheet 2 of 2).

Consideration of Bureau Procedures

Consideration of Bureau Procedures

The original charge to the IRG directed that the cause of the failure be determined and *that recommendations, if warranted by its findings, be provided to prevent a recurrence of such failures.* (See Appendix A.)

During its study of the cause of failure, the IRG identified areas where it appears that procedures and documentation, or a lack of them, may have played a part in decisions that ultimately led to the failure of Teton Dam. The IRG has determined that a study of the specific procedures followed in the design and construction is needed to identify deficient or absent procedures and appropriate documentation. The study made by the Interior Department's Office of Administrative and Management Policy, "Review of Bureau of Reclamation's Dam Building Procedures" (December 1976), outlines general Bureau procedures. The report does not include the specific procedures followed during the design and construction of Teton Dam.

The Bureau has prepared an annotated index of documents related to important geologic, design, and construction aspects of Teton Dam. Key personnel involved in the decisionmaking process will be interviewed so that any procedures in need of correction may be determined and improved procedures recommended. Some areas of concern of the IRG are:

- key trench design
- analysis of abutment and key trench geometry
- considerations of cracking potential
- treatment of rock surface in contact with zone 1
- development of specifications
- design involvement in construction
- technical specialists' role in design and construction
- role of construction liaison personnel
- evaluation of instrumentation
- consideration of the need for a review board

The recommendations or suggestions developed by the IRG following the interviews will be incorporated in a report that will be submitted no later than June 1977.

Conclusions

Two classes of conclusions are presented: (1) the primary conclusions concerning the failure of Teton Dam, and (2) other conclusions concerning site selection, design, construction, and operation of the dam.

Primary Conclusions

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

The Bureau had the necessary information available to develop an adequate defensive design. A safe dam could have been built at the site utilizing design concepts that were known at that time.

Other Conclusions

Site Selection.–Site selection procedures were adequate; the site selected was the best of the available alternative sites for the desired purposes of the project.

Design.–Enough geologic data were obtained for a proper design of the dam.

Reliance on the grout curtain for total control of damaging seepage inhibited adoption of other design features that could have prevented the failure.

The design failed to provide a defense against both flow through embankment cracks and erosion of the zone 1 impervious core at rock surfaces. Defensive measures, such as rock surface sealing and adequate filters, were well within the state-of-the-art at the time Teton Dam was designed and should have been used.

The design incorporated a feature, key trenches in the abutments, that significantly departed from past Bureau of Reclamation practices.

The geometry of the abutment key trenches was conducive to developing stress patterns that could have allowed cracking of the impervious core.

The narrow width of the sealed foundation rock at the bottom of the key trench, combined with the high permeability of the rock foundation on either side of the key trench, produced steep hydraulic gradients across the trench.

Recognition of the potential for occurrence of hydraulic fracturing was not in the general state-of-the-art of dam design at the time Teton Dam was designed. However, the potential for cracking from differential settlement near the steep abutments and key trenches within the abutments has been recognized by dam designers in the past.

An independent review of the design might have identified the design deficiencies.

Structural instability of the embankment was not a factor in the failure.

Instrumentation of the embankment and foundation would have been desirable to monitor the performance of the dam; however, there is no indication that such instrumentation would have furnished information that could have been used to prevent this failure.

Construction.–The grout curtain was satisfactory to control seepage within normal limits.

The foundation preparation for the embankment placement was in accordance with the contract plans and specifications.

The placement methods used to construct the embankment were in agreement with the general practices and procedures followed in earth dam construction. Construction control testing was adequate and followed generally accepted procedures.

Operation.–The relation of the high rate of filling of the reservoir to the failure is indeterminate. The fact that the main river outlet works was not available to assist in controlling rate of storage is judged to have had no significant effect on the ultimate failure of Teton Dam.

Additional Investigations

Further investigations are desirable to assure that all possible factors that might have influenced the failure have been considered. Appropriate reports will be issued. The investigations are as follows.

Grout Curtain Investigations

The original program for drilling and testing of the effectiveness of the grout curtain was not completed because of winter shutdown. Additional holes will be drilled in the right abutment to complete the program that was interrupted by shutdown. This will extend the present investigation both laterally and in depth.

Left Abutment and Embankment Investigations

Physical conditions on the left abutment are very similar to those of the right abutment. An investigation of the embankment and embankment foundation contact surface will be made by a means that will permit visual inspection.

The investigation will be primarily to search for cracks in the remaining left embankment and to find evidences of erosion channels through the core or at the contact of the zone 1 and the rock surface. The area of concern, in the vicinity of the key trench, is far enough from the failure breach to be influenced only slightly, if at all, by postfailure stress release. Further in situ stress investigations are planned.

Finite Element Studies

Additional finite element studies are planned. These will include studies of the left embankment, in addition to some parametric studies, to determine the influence of key trench shape on stress distribution within the embankment. Conditions encountered in the left embankment may suggest further analyses.

Recommendations

IRG Recommendations

In the charge to the IRG, the Under Secretary of the Interior directed that the group make recommendations to prevent the recurrence of failures such as that at Teton Dam. Therefore, it is recommended that:

- An independent board of review be convened for each major dam project. This board should review both design and construction at appropriate intervals
- Design decisions be formally documented
- Design personnel remain actively involved with a project during construction including frequent scheduled site visits
- Major dams and their foundations include an instrumentation program to monitor construction and postconstruction behavior. Instrumentation data should be promptly interpreted and evaluated

These recommendations may be supplemented when all IRG investigations are completed.

Charge and Directions to the IRG

United States Department of the Interior

OFFICE OF THE SECRETARY
WASHINGTON, D.C. 20240

June 8, 1976

Memorandum

TO : Assistant Secretary--Land & Water Resources
Assistant Secretary--Energy & Minerals
Director--Geological Survey
Commissioner--Bureau of Reclamation

FROM : Under Secretary

SUBJECT: Teton Dam Failure Review Group

I am establishing today a review group to examine the causes of the Teton Dam failure. The review group should examine, among other matters relevant to the causes of the failure, the following:

- Geologic aspects
- Engineering design factor
- Construction details
- Hydrologic factors
- Pertinent background information and testimony

If the findings warrant, the group should also make recommendations, as appropriate, designed to prevent recurrence of failures.

The review group will be chaired by Deputy Assistant Secretary Dennis Sachs. The Federal subject matter experts of the group will be drawn from:

- Department of the Interior
  - Bureau of Reclamation
  - U.S. Geological Survey
- Tennessee Valley Authority
- Corps of Engineers (Department of the Army)
- Soil Conservation Service (Department of Agriculture)

Save Energy and You Serve America!

The review group should provide to me an interim report by July 12 and a final report by July 30. This review effort is not a substitute for the "Blue Ribbon" panel that will conduct an investigation of the failure independent of the Department. Information developed by the review group will be made available to the panel.

Bill Lyons, for the Under Secretary's Office, will be the focal point for coordination with the Blue Ribbon panel. Dennis Sachs will serve as the contact point for all matters related to the Department-led review.

cc: Solicitor
Assistant Secretary/Program Development & Budget

United States Department of the Interior

OFFICE OF THE SECRETARY
WASHINGTON, D.C. 20240

JAN 17 1977

Memorandum

To: Chairman, Teton Dam Failure Interior Review Group

From: Secretary of the Interior

Subject: Review of Procedures Used in Specific Features of Teton Dam

To fulfill completely the original charge made to your group, you are requested to study the relationship of those physical aspects of Teton Dam that may have contributed to its failure to the specific procedures followed in the Bureau of Reclamation that gave rise to those aspects. This will provide the Interior Review Group with more complete information upon which to base recommendations designed to prevent recurrence of such failures.

The investigation will involve those specific features of Teton Dam which either the Interior Review Group or the Independent Panel have identified as being related to the failure. It will center on the evolution and development of each feature with the identification and documentation of decision points and the delineation of any divergence from "normal" procedures. Examples (not an inclusive list) of aspects of the dam and its failure that especially warrant this type of investigation are: the design of the key trench; the surface grouting program; the rate of filling of the reservoir; the selection of embankment materials; and the instrumentation deficiencies of the surveillance program.

The Interior Review Group will administer this program with assistance from the Assistant Secretary--Administration and Management and the Bureau of Reclamation. In response to a set of specific requests to be prepared immediately by the Interior Review Group, Administration and Management will gather detailed information on the procedures followed. The Bureau of Reclamation will provide related documentation, with particular emphasis on those points where critical decisions were made.

The Interior Review Group will be responsible for the synthesis and interpretation of as much of this information as possible for inclusion in the Group's report to the Secretary currently scheduled for February 1977. An addendum to that report, reporting all further information gathered from this special procedures review, should be submitted no later than June 1977.

/s/ Thomas S. Kleppe

Thomas S. Kleppe

AMERICAN REVOLUTION BICENTENNIAL 1776-1976

Report of the Geology Task Group

U.S. Department of the Interior
Teton Dam Failure Review Group

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ABSTRACT

The mission of the Geology Task Group was to review the geologic aspects of the Teton damsite and evaluate their possible significance in failure of the dam. This investigation was conducted through review of Bureau of Reclamation pre-failure and post-failure geologic investigations, site observations, and interviews of Bureau of Reclamation geologists. In order to avail itself of additional expertise, as needed, the Task Group has consulted from time to time with Federal specialists in regional geology of the Snake River Plain, volcanic geology, engineering geology of damsites in volcanic rocks, ground-water geology, and seismology.

Teton Dam is located in a steep-walled canyon that was eroded by the Teton River into a gently rolling, eolian silt-covered volcanic upland adjoining the eastern Snake River Plain. The rock that forms the canyon walls and which constitutes the major bedrock formation throughout the damsite and reservoir area is a rhyolite welded ash-flow tuff. At the damsite, as well as elsewhere along the Teton River, this welded tuff is characterized by the presence of prominent and abundant open joints and fissures. The high natural permeability of the rock in the right abutment resulting from the presence of these interconnected joints and fissures, especially in the upper part of the abutment, was a major factor in the dam failure. Of particular importance was the contact of open-jointed rock with zone 1 materials both upstream and downstream of the grout curtain.

A coincidence of major, throughgoing joints and other rock defects occurs along the right key trench between grout cap centerline stations approximately 13+00 and 14+00, an interval that includes the whirlpool that developed about 80 feet upstream in the reservoir shortly before the breaching of the dam. These geologic features may have contributed to the failure.

The closest positively identified faults are about 7 to 8 miles east of the damsite and they are not known to be active. The records of the seismic monitoring network centered on Teton Dam and Reservoir demonstrate that failure of the dam was not the result of seismic activity.

Surface and subsurface preconstruction geologic investigations generally appear to have been well conceived and executed and the data obtained to have been adequate to establish the basic geologic features that should have been considered in the design of the dam. In addition, construction geologic investigations in the right abutment and associated appurtenant works were generally adequate to verify the basic geologic conditions determined by the preconstruction investigations. However, the Geology Task Group feels that the development of a more complete and better integrated three-dimensional picture of the geology of the right abutment would have helped to focus even greater engineering attention on the critical problem of open joints that would be in contact with the embankment materials.

An adequate system was not provided for monitoring ground-water levels to evaluate the performance of the key trenches and grout curtain during filling of the reservoir.

INTRODUCTION

Purpose and Scope

In establishing the Teton Dam Failure Review Group, the Under Secretary of the Interior emphasized several factors that should be examined in determining the cause or causes of failure of the dam. Listed first among these factors was the geologic aspects. In order to facilitate this part of the review, the Geology Task Group was formed. The specific mission of this task group is to review the geologic aspects and evaluate their possible significance in the failure.

Participants

The members of the Geology Task Group are:

- Robert L. Schuster, Chief, Engineering Geology Branch, U.S. Geological Survey
- John T. McGill, Geologist, Engineering Geology Branch, U.S. Geological Survey
- David J. Varnes, Geologist, Engineering Geology Branch, U.S. Geological Survey
- Lloyd B. Underwood, Chief, Geology Section, Engineering Division, U.S. Army, Office Chief of Engineers

In order to avail itself of additional expertise, as needed, the Task Group has consulted from time to time with Federal specialists in regional geology of the Snake River Plain, volcanic geology, engineering geology of damsites in volcanic rocks, ground-water geology, and seismology.

Activities

Members of the Geology Task Group have performed the following principal functions as related to the activities of the Review Group:

(1) Made repeated visits to the damsite to (a) examine the geologic conditions, (b) hold discussions with Bureau of Reclamation project and regional geologists, and (c) review project records.

(2) Reviewed Bureau of Reclamation pre-failure investigations of the site geology.

(3) Recommended further geologic investigations which would aid in determining the cause of failure.

(4) Reviewed and evaluated the results of post-failure geologic investigations.

Sources of Information

The principal sources of geologic information concerning the damsite are the extensive geologic reports, drawings, drill-hole logs, and other records of the Bureau of Reclamation. Copies of these documents have been distributed to the members of the Task Group and the Review Group or otherwise made readily available to them. Other sources of information referred to or included in this report are cited at the appropriate places.

Descriptions of pre-failure investigations and of parts of the site geology are based mainly on the unpublished report, "Teton Dam geology," prepared in June 1976 by the Geology and Geotechnology Branch of the Bureau of Reclamation. That report presents a summary of the geologic conditions in the various parts of the dam foundation and associated appurtenant works as known to the Bureau of Reclamation at the time of completion of the major construction work shortly before the dam failure. It also presents a comprehensive annotated listing of all documentary materials which provided the basis for that geologic summary.

REGIONAL GEOLOGIC SETTING [1]

The Lower Teton Division of the Teton Basin Project is located in and adjacent to the eastern Snake River Plain, a 50-mile-wide, northeast-trending volcanic-filled tectonic depression that was formed by downwarping and downfaulting in late Cenozoic time (fig. B1). Volcanism has been concurrent with the tectonic subsidence, so that the older volcanic rocks, mainly Pliocene and Pleistocene rhyolite ash-flow and air-fall tuffs, are now exposed primarily along the margins of the plain, as in the Rexburg Bench[2] area, and dip gently beneath younger basalt lava flows that form the plain itself. The northeastern end of the plain seems to be the youngest and most active part, both tectonically and volcanically. The mountainous terrain that bounds the eastern Snake River Plain on the northwest and southeast is typical Basin-Range structure formed concurrently with the plain. The mountain blocks are composed mainly of highly deformed Paleozoic and Mesozoic sedimentary rocks (included in map unit 6 in fig. B1).

In most of the Rexburg Bench area (fig. B2), bedrock consists of several sheets of rhyolite welded ash-flow tuff interlayered with air-fall tuff and detrital sediments and locally overlain by one or more thin basalt flows. The uppermost ash-flow sheet is the middle member of the Huckleberry Ridge Tuff (Christiansen and Blank, 1972; and Christiansen, unpublished manuscript); it is this rock in which the canyon of the Teton River is incised. For convenience, this rhyolite welded ash-flow tuff will be referred to hereafter simply as welded tuff. This widespread unit ranges in thickness from more than 500 feet in the northeastern part of the Rexburg Bench to 150 feet or less near the southern edge, but locally the thickness varies greatly because of the irregular topographic surface on which the ash flow was deposited. This unit is believed to have been erupted from the Island Park area from a caldera (see fig. B1) whose southern rim is Big Bend Ridge, located about 8 miles north of Ashton, Idaho. The age of the Huckleberry Ridge Tuff has been determined (by the potassium-argon method) as about 1.9 million years and thus falls nearly at the Pliocene-Pleistocene boundary.

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[1] This discussion is based mainly on two reports of the U.S. Geological Survey (Oriel and others, 1973; Prostka and Hackman, 1974) and presentations made by USGS regional geologists S. S. Oriel to the Review Group on June 15, 1976, and P. L. Williams to the Independent Panel on June 29, 1976. These materials contain references to the primary sources of geologic information on the region.

[2] The Rexburg Bench (fig. B2) is a roughly triangular area of mostly smooth to gently undulating topography extending between Byrne (about 9 miles south-southeast of Rexburg), Rexburg, and the vicinity of Teton Dam. It rises above the flood plains of the Snake and Teton Rivers along a more or less rectilinear to somewhat irregular and steep to gently sloping escarpment that is as much as 400 feet high at Rexburg.

The basalt flows that overlie the welded tuff were erupted from local sources, such as several small lava domes on the Rexburg Bench. A sample of basalt collected by G. M. Richmond, U.S. Geological Survey (oral communication, December 1976), from an outcrop near the bridge where Idaho State Highway 33 crosses Canyon Creek, about 6 miles southeast of Teton Dam, has been dated by Minze Stuiver, Univ. of Washington; it is interpreted to be between 225,000 and 500,000 years old. This basalt is probably equivalent to or younger than the basalt exposed along the rims of the canyons of Teton River and Canyon Creek near their confluence and probably equivalent to or older than the remnants of intracanyon basalt in the bottom of the canyon of Teton River at and downstream from the dam.

Wind-deposited silt, or loess, mantles much of the northern Rexburg Bench and adjacent volcanic upland near the canyon of Teton River. Its source area was the river flood plains on the adjacent Snake River Plain. The loess is of Pleistocene age and may be less than 70,000 years old.

Most of the faults in the region tend to be roughly parallel and perpendicular to the northeast trend of the eastern Snake River Plain. Northwest-trending normal faults are common along the southeastern margin of the plain, where some of them bound northwest-trending mountain ranges and intervening valleys or basins that were blocked out principally by displacements during middle and late Cenozoic northeast-southwest crustal extension of the region. A set of well-defined faults with northwest trend has been mapped in and near the upper canyon of the Teton River to within about 8 miles east of Teton Dam (fig. B2; Magleby and Sweeney, 1963). These faults probably are related to late Pliocene and Quaternary uplift of the Big Hole Mountains (see fig. B1 for location); they offset the welded tuff and overlying basalt as much as 80 feet vertically. The loess does not appear to be displaced by these faults, but suitable exposures are lacking for a definitive determination. Other well-defined northwest-trending faults have been mapped to within about 11 miles southwest of the dam, where they cut basalt flows at the ground surface (fig. B2). These faults and a line of basaltic vents adjacent to them lie along the northwest projection of the Grand Valley fault (fig. B1). Some inferred faults with northwest trend in Huckleberry Ridge Tuff, 3 to 4 miles north-northeast of the dam in the area labeled Hog Hollow on published topographic maps, may be part of a collapse structure along the southwest margin of a caldera (H. J. Prostka, oral communication, August 1976).

Northeast-trending high-angle faults in the Snake River Plain and its margins presumably developed during subsidence of the plain accompanying general northwest-southeast crustal extension but they have been largely buried by lavas and sediments. Well-defined faults with northeast trend have been mapped cutting the welded tuff no closer than about 7 miles east-northeast (fig. B2) and about 10 miles south (Haskett, 1972) of the dam.

Northeast-trending lineaments located closer than the well-defined faults to the dam are conspicuous on aerial photographs and large-scale topographic maps of the area. Structural control seems likely for those lineaments having considerable length or topographic relief, but none has been confirmed as of fault origin. One such prominent lineament was noted on the upland about 500 to 800 feet west of and roughly parallel to the segment of the canyon in which the dam is located. On the basis of this lineament and interpretation of geologic cross sections in a Bureau of Reclamation report by Haskett (1972) on ground-water geology of the Rexburg Bench, regional geologists of the U.S. Geological Survey inferred the existence of a northeast-trending fault extending about 18 miles from south of Rexburg to and beyond the immediate vicinity of the right abutment of the dam. This was communicated to the Bureau of Reclamation in a meeting in March 1973 and then in a draft preliminary report (Oriel and others, 1973). Subsequent fieldwork by the USGS revealed no evidence of faulting of welded tuff or other surface rocks along the trace of the inferred fault; therefore, on a preliminary geologic map of the region (see fig. B2) the feature is shown as a concealed subsurface fault, inferred from well data, that is of pre-Huckleberry Ridge Tuff age. After the Teton Dam failure, a replotting of map locations of the fault interpreted from Haskett's geologic sections in the southern part of the Rexburg Bench indicates that the corrected map trace of the inferred fault, projected northeast, would fall as much as 3 miles northwest of the prominent, northeast-trending topographic lineament adjacent to the right end of the dam.

In a Bureau of Reclamation memorandum dated April 1, 1976, prepared by Haskett, the geologic conditions downstream from the dam are briefly described and the possibility is suggested of a fault along a line trending northeast from the town of Newdale. The production of wells per foot of penetration of welded tuff increases manyfold about at this line (Haskett, 1972, p. 9, 11), which also coincides with a narrow zone of hot water wells. This line probably would cross the northwest-trending segment of the canyon of

Teton River approximately 2 miles downstream from the dam, possibly within one-half mile of the replotted position of the inferred subsurface fault beneath the Rexburg Bench and about on the projection of two short, northeast-trending topographic lineaments located between Newdale and the canyon. These lineaments are shown on the USGS preliminary geologic map (see fig. B2) as concealed normal faults; they also are inferred faults.

The closest positively identified faults are about 7 to 8 miles east of the damsite, and they are not known to be active.

SEISMICITY

Seismicity believed to be associated with prominent faults is characteristic of the mountain ranges bordering the northern, eastern, and southern sides of the eastern Snake River Plain. Several earthquakes with maximum Modified Mercalli intensities of VIII have been experienced in the region. For this reason, southeastern Idaho has been included in U.S. Seismic Risk Zone 3 (Algermissen, 1969). However, in the area of the Snake River Plain and its margins, although youthful and active deformation is indicated by geologic, geodetic, and geothermal conditions, the level of locally generated historic seismicity is low. This is known from the historical record of 1915 to 1974 and, since June 1974, from a cooperative Bureau of Reclamation-Geological Survey program to study the seismicity of the Teton Dam and Reservoir area, in particular to investigate any seismic effects of reservoir filling and evidence of fault activity within about a 25-mile radius. The cooperative program involved the installation of a monitoring network of three seismic stations located about 18 miles north, east, and southeast of Teton Dam. The following conclusions have been drawn from this short-term monitoring of the seismicity (Navarro and others, 1976):

(1) During the period June 16, 1974, to March 31, 1976, no seismic event of Richter magnitude 2.2 M_L or greater was observed within an 18-mile radius of Teton Dam. In addition, all events within 12 miles of the dam have been confirmed as having been caused by blasting.

(2) During the period April 1, 1976, to June 9, 1976, no seismic event was observed within an 18-mile radius of the dam, with the exception of identified blasts. The closest and largest earthquake, which had a magnitude of 1.7 M_L, was located southwest of Victor, Idaho, about 37 miles southeast of the dam.

(3) No increase in seismic activity near the dam was recorded while the reservoir was being filled.

(4) For at least 4-1/2 hours, beginning within 1 minute of 11:47 MDT, June 5, 1976, the seismic monitoring network recorded ground motion generated by the breakup of the dam and release of the reservoir water. Maximum amplitude of the ground motion was reached within half an hour and continued for about an hour.

These observations demonstrate that the failure of Teton Dam was not the result of seismic activity.

GEOLOGIC INVESTIGATIONS BY THE BUREAU OF RECLAMATION[3]

Preconstruction Investigations

Prior to selection of the site of Teton Dam in 1964, various potential sites were investigated along the Teton River and its tributaries. In 1946 two sites on Canyon Creek, a tributary of the lower Teton River, were studied. In 1956, a low diversion damsite was investigated 5 miles downstream from the present site. In early 1961, five alternate sites were studied upstream of Teton Dam in the stretch of the river from Linderman Draw to the mouth of North Fork. (For information on the results of these investigations and selection of the site, see the section titled "Project Siting" in the IRG report.)

At the Teton damsite the initial geologic investigations consisted of two core holes by the Corps of Engineers in 1957. Subsequent planning and preconstruction drilling programs were undertaken by the Bureau of Reclamation in 1961, 1962, and 1967 through 1970. These programs consisted of a total of 102 core drill holes, ranging in depth from 20 to 698 feet. Many holes, most on the order of 300 feet deep, were drilled along the sites of the proposed cutoff and key trenches. Nearly all of these holes were drilled at an angle of 30° to the vertical and directed to intersect high-angle joints. The core from all the drill holes was

[3] The summary descriptions of preconstruction and construction investigations are based mainly on an unpublished report prepared in June 1976 by the Geology and Geotechnology Branch of the Bureau of Reclamation (U.S. Bureau of Reclamation, 1976). The locations of preconstruction and construction core drill holes are shown on the geologic map of the damsite (fig. B3) and those near the axis of the dam are shown on the geologic section through the damsite (fig. B4) in this Task Group report.

logged, and a down-hole television camera was used to examine some of the holes. A summary of preconstruction core drilling is presented in table B1.

Twenty-six of the holes were drilled in the left abutment, 9 in the valley floor, 9 in the right abutment, and 56 at sites of appurtenant structures. A special drilling program of 10 holes was included to evaluate the effectiveness of the pilot grouting program that was conducted in a blanket grouting zone in the valley floor and a curtain grouting area in the left abutment. Long-term pump-in permeability tests were conducted in five holes in the right abutment. Twelve holes (six in each abutment) were examined with a down-hole television camera to analyze the joint system. Two holes also were drilled about 10 miles upstream from the site as part of the evaluation of the reservoir water-holding capability.

Cores from five drill holes in the welded tuff and three holes in the intracanyon basalt were tested to determine the following physical and mechanical properties: modulus of elasticity, Poisson's ratio, compressive strength, absorption, and specific gravity. In addition, the same cores were subjected to petrographic examination both before and after the physical and mechanical tests were conducted. The "before" examinations determined the lithology, vesicularity, and degree of weathering of the rock in the cores. The "after" descriptions and measurements were made on the specimens tested to failure to determine their mode and angle of failure, and the nature of the materials associated with the failure.

Complementing the drilling programs was a geologic field-mapping program which produced maps at scales of 1"=100' and 1"=50' of the areal geology of the damsite and maps at a scale of 1"=20' of the joints in the outcrops of welded tuff on both abutments. Information obtained from the drilling and geologic field-mapping programs was used to compile geologic cross sections at various scales and degree of detail. A preliminary geologic report in 1963 included a reconnaissance geologic map of the reservoir area at a scale of 1"=1,000'.

An outcrop of intracanyon basalt on the left canyon side about 1 miles downstream from the damsite was explored as a possible source of riprap for the dam (see fig. B5). Three core holes were drilled and a sample geologic map was made of the left canyon side between the outcrop and the damsite.

Magnetometer surveys were made in the canyon bottom in the area of the proposed cutoff trench in 1967 and in the power plant area in 1969 to supplement drilling information regarding the extent and configuration of the buried intracanyon basalt.

All of the information relating to the preconstruction geologic investigations program at the damsite through early 1971, including logs, maps, sections, etc., is included in the Preconstruction Geologic Report (U.S. Bureau of Reclamation, 1971).

Construction Investigations

During the course of construction, which began in 1972, the various site excavations were examined in detail and "as built" drawings prepared showing the geologic features exposed at the final excavated surfaces. Major emphasis was placed on the mapping of joints in the welded tuff.

The geologic map of the cutoff trench and key trenches includes adjacent parts of the canyon walls that also were subsequently covered with zone 1 material. The map, at a scale of 1"=20', identifies the geologic materials in a very general way and shows joint traces and attitudes in the welded tuff; notations are provided as to widths, fillings, and coatings only for joints more than 0.1 foot in width. As indicated on the map, much of the bottom of the left key trench and the extreme right part of the right key trench were not mapped. Rose diagrams of the joints in selected areas of the right abutment recently were added to the map. Supplementing the geologic map is a series of panoramic color photographs with geologic overlays showing prominent joint traces in construction exposures of the walls of the key trenches and adjacent parts of the right and left abutments. As of early February 1977, two of four volumes of photographs with overlays had been completed. They cover most of the right abutment and key trench above elevation approximately 5150. The scale of the overlays is about 1"=10' to 1"=15'.

The geologic map of the spillway excavation, at a scale of 1"=10', also is mainly a map of joints in welded tuff. A geologic cross section along the spillway site, at a scale of 1"=50', shows the several geologic units present from the original ground surface to an average depth of about 60 feet. A final report has been drafted.

Joint maps, at a scale of 1"=10', and brief geologic reports were prepared for the river outlet works tunnel, gate chamber access shaft, and intake shaft, and for the auxiliary outlet works tunnel, gate chamber access shaft, and adit. These engineering structures are located entirely within the welded tuff.

Table B1.—*Summary of preconstruction and construction core drilling*

[From U.S. Bureau of Reclamation, 1976]

SUMMARY OF PRECONSTRUCTION CORE DRILLING

Location	Number of holes	Depth		Total feet	Purpose
		Minimum	Maximum		
Left abutment	26*	19.6	556.0	6,569.8	Deep and shallow foundation conditions, permeabilities (seepage loss potential), and effectiveness of pilot grouting program
Valley floor	9	58.5	505.8	2,228.8	
Right abutment	9	296.8	698.0	3,683.8	
Appurtenant structures	56	26.0	303.7	4,337.0	Deep and shallow foundation conditions, some permeabilities, and water table locations
Reservoir	2	498.8	506.0	1,004.8	Reservoir seepage loss potential
TOTAL	102			17,824.2	
SUMMARY OF CORE DRILLING DURING CONSTRUCTION ("500" series, drilled 1974-75)					
Left abutment	2	343.7	397.1	740.8	Fissure and void exploration and ground-water observation
Right abutment	4	399.8	646.0	2,285.1	
River outlet works area	1			372.0	Ground-water observation
TOTAL	7			3,397.9	

*Includes 10 core holes for pilot grouting investigation.

In October 1975 a map was made which locates and briefly described a series of major fissures discovered along the haul road on the upper part of the right canyon wall 650 to 1,450 feet upstream from the centerline of the dam.

In addition to the geologic mapping, three deep exploratory core holes were slant-drilled in the outer portion of the right abutment and one in the outer portion of the left abutment to investigate at depth the areas where fissures and cavities had been discovered in the key trench excavations. Two of these holes in the right abutment were kept open for use as ground-water observation wells. Also, one deep vertical core hole was drilled farther out on each abutment to service as an observation well in the ground-water monitoring program. An additional core hole for ground-water observation was drilled near the outlet of the river outlet works tunnel, adjacent to the power plant. Table B1 summarizes the core drilling during the construction stage.

During the excavation of the cutoff trench, six auger holes were bored to help define the limit and extent of silt and clay layers that were present in the alluvium near the bottom of the excavation, and three auger holes were bored to define the surface of the welded tuff adjacent to and below an overhang that had been encountered. As the cutoff trench was deepened and completed, 14 test trenches were excavated to further investigate the silt and clay layers.

In 1972 an electromagnetic subsurface profiling survey of a research experimental nature was conducted by Geophysical Survey Systems, Inc., in an effort to delineate possible near-surface voids in bedrock beneath the eolian silt just beyond the end of the left abutment.

In 1973 a cooperative program was initiated between the Bureau of Reclamation and the Geological Survey to monitor the seismicity of the Teton project area and to provide evidence for evaluation of the activity of the known and inferred faults (see Seismicity).

Post-failure Investigations

Post-failure geologic investigations and related studies have been conducted or supervised by the Bureau of Reclamation to help determine the cause of failure. These investigations and studies through January 1977 can be summarized as follows:

(1) *Initial geologic mapping of the right abutment post-failure rock surface* (preliminary maps and tabulations completed in August 1976).

This first-phase mapping was accomplished on a three-sheet contoured orthophotomap of the canyon wall at a scale of 1″=20′. At the time this work was done, the remnant of the key trench and much of the rock surface directly downslope were covered by zone 1 material that was in place or had sloughed from the escarpment formed during the failure of the dam. The map shows traces of major joints, contacts between units of the welded tuff, and locations of post-failure waterflows from the abutment. Included with the map are an explanation of the geologic units and tabulated descriptions of the individual joints and waterflows.

(2) *Geologic mapping of the bottom of the right abutment key trench and downslope strip along centerline of grout cap* (completed in November 1976; station numbering revised in December 1976).

Following excavation of the zone 1 material from the key trench and downslope area, a 20- to 30-foot-wide strip of the newly exposed rock surface was mapped in great detail between grout cap stations 11+40 and 16+20. The graphic results of this survey consist of (a) geologic overlays for strip topographic maps at a scale of 1″=5′, and (b) geologic sections along the upstream and downstream drill holes of the grout curtain at a scale of 1″=5′, as well as (c) geologic sections 100 feet downstream (at a scale of 1″=10′) and 150 feet upstream (at a scale of 1″=5′) of the centerline of the dam. These maps and sections, which mainly show details of the jointing, are accompanied by an expanded explanation of the geologic units and tabulations of descriptions of the individual joints.

(3) *Detailed geologic mapping of the right abutment post-failure rock surface* (completed in January 1977).

The results of this survey consist of 24 geologic overlays for very large scale (approximately 1″=5′ to 1″=11′) oblique helicopter photographs of the right abutment and tabulations of descriptions of the individual joints. The explanation of the geologic units and symbols is essentially the same as that for the key trench maps. The photographs, all but three of which were taken October 12 or October 29, 1976, after most of the loose debris had been removed, consist of an overlapping series directed into the abutment. They show most of the exposed rock surface on the right canyon wall from about 1,250 feet downstream to about 1,000 feet upstream of the centerline of the dam with the exception of the upper part of the key trench.

(4) *Geologic logging and pressure-testing of post-failure (600 series) core drill holes located along the right abutment grout curtain* (37 holes were drilled, logged, and pressure-tested in 1976).

Although the main purpose of this drilling program was to check the perviousness of the grout curtain, information on joints, lithology, and presence of grout was obtained by geologic logging of the cores. The locations and logs of the post-failure drill holes are provided in the report of the Grouting Task Group, Appendix C of the IRG report. Thirty-four relatively shallow holes were drilled to check the grout curtain under the spillway and upstation to station 14+27.8 to appraise the grout curtain in the key trench; these holes ranged from 21 to 145 feet deep. Ten of these holes were vertical, 14 were angled away from the river, 8 were angled toward the river, and 2 were angled through the grout curtain, one each upstream and downstream. Three deeper holes were drilled in the vicinity of stations 3+00 to 5+00; this stationing was selected for proximity to the two large vertical fissures intersecting the key trench at approximately stations 3+50 and 4+35. The first of these holes (DH-650), located at station 3+00, was directed upstation at an angle of 59° from the horizontal and reached a depth of 351.5 feet. The second deep hole (DH-651) was a vertical hole at station 4+34 which was intended to pass through the welded tuff into underlying sediments to a total depth of 1,000 feet. Due to technical difficulties encountered in coring through the sediments, this hole was drilled three times (DH-651, 651A, and 651B) to depths of 622, 495, and 890 feet with sediments being sampled at depths below approximately 500 feet. The third deep hole (DH-652), which was located at station 5+11, was directed downstation at an angle of 56° from the horizontal to a total depth of 450 feet.

In addition to the coring, logging, and pressure-testing of the 37 holes along the grout curtain, 6 of the holes were photographed with a Corps of Engineers borehole camera. The U.S. Army Engineer Waterways Experiment Station is analyzing and interpreting the borehole photography.

(5) *Laboratory testing of deep core samples from 500-series drill holes.*

Physical and mechanical tests were conducted in 1976 on five samples from cores which had been obtained when the 500 series of holes was drilled in 1974 and 1975. Size gradation analyses, Atterberg limit tests, and triaxial and unconfined compression tests were run, as appropriate, on these samples from drill holes 501, 503, 506, and 507. One sample was from the basal part of the welded tuff and the other four samples were from underlying volcanic ash, tuff, and tuffaceous sediments.

(6) *Finite element study of dam foundation.*

Stress analyses were made by the Bureau of Reclamation of the natural foundation at Teton Dam to estimate the magnitudes of stresses and deformations induced in the foundation by the imposed loadings of the dam, reservoir, and bank storage. The analyses were performed by the finite element method using two-dimensional plane strain concepts. Preliminary analyses based on estimated deformation properties of the foundation materials were completed in September 1976.

(7) *Releveling to investigate possible foundation elevation changes.*

In order to detect foundation deformations which could have occurred due to the imposed loadings of the dam, reservoir, and bank storage, all recoverable foundation reference points under the embankment for which pre-embankment construction elevations were available were resurveyed to determine elevation changes. These reference points are on the right abutment grout cap. Post-failure elevations were obtained in early November 1976. In addition, all existing bench marks within the site area were releveled and tied to monuments several miles from the dam.

GENERAL GEOLOGY OF THE DAMSITE

Teton Dam is located in a steep-walled canyon that was eroded by Teton River into a gently rolling, loess-covered volcanic upland adjoining the northeastern part of the Rexburg Bench. Prior to construction of the dam, the river flowed through the damsite area in broad meanders on a 750-foot-wide, flat-bottomed flood plain and the rocky canyon walls rose about 280 feet above the canyon floor (fig. B5).

The rock that forms the canyon walls and which constitutes the major bedrock formation throughout the damsite and reservoir area is a rhyolite welded ash-flow tuff. It overlies poorly to moderately indurated sediments and is overlain by thin surficial deposits of eolian silt, alluvium, and slope debris.

Remnants of intracanyon basalt are locally present at and downstream from the damsite.

WELDED TUFF

The rhyolite welded ash-flow tuff is the middle member of the Huckleberry Ridge Tuff (Christiansen and Blank, 1972; and Christiansen, unpublished manuscript). This welded tuff is extensively exposed in the canyon walls, partly as rock ledges, but over large areas of these slopes it is obscured by a thin cover of slopewash. Some of the outcrops have a crude but distinct pinnacle and pillar appearance, reflecting the locally more or less columnar pattern of high-angle joints.

The thickness of the welded tuff and the elevation of its base vary greatly within the damsite, as shown by prefailure exploratory borings and deep grout holes. Near the axis of the dam (fig. B4), the thickness of the welded tuff ranges from a minimum of about 50 feet in the left side of the channel section, where the ancestral Teton River had eroded a canyon in the ash-flow sheet, to a maximum of at least 573 feet in drill hole 506 about 500 feet beyond the right end of the key trench, where a very complete section is preserved. Near the axis of the dam, the base of the welded tuff ranges in elevation from less than 4,710 feet, at least 328 feet below the preconstruction canyon floor of Teton River, to about 5,150 feet (or about 110 feet above the level of the canyon floor) in two widely separated drill holes beyond the end of the left wing and key trench. About 1,200 feet upstream from the dam centerline, in the horseshoe-shaped topographic recess adjacent to the intake tower of the river outlet works, the base of the welded tuff is at an elevation of at least 5,117 feet and possibly about 5,150 feet, and would daylight were it not covered with about 10 feet of slopewash (U.S. Bureau of Reclamation, 1971, drawing 549-125-263). About 420 feet downstream from the dam centerline the apparent base of the welded tuff was exposed at an elevation of about 5,015 feet in the floor of the river outlet works tunnel during construction; and about 450 feet farther downstream from the dam centerline the base of the welded tuff is at an elevation of 4,697 feet in a vertical drill hole near the end of the tunnel.

The welded tuff is densely to partially welded and generally is fairly hard; hand-size fragments break with a moderate hammer blow. It is relatively lightweight, by comparison with the local basalt, for example. The texture generally is porphyritic with moderate to abundant phenocrysts mainly of light-colored alkali feldspar up to about 1/4 inch in size within a matrix of devitrified, flattened and welded glass shards. The rock contains scattered lapilli of pumice and volcanic rock fragments and zones of flattened vesicles and vugs. The welded tuff is devitrified except for a thickness of a few feet at the base of the sheet. The color of the devitrified tuff ranges from light gray to medium gray, with shades of red, brown, and purple. Locally the rock has been stained reddish brown to brick red by ground water and possibly hydrothermal alteration.

The top and bottom parts of the welded tuff sheet differ appreciably from the rest of the formation. The uppermost 20 feet or so is generally more pink than gray, softer, less densely welded and therefore of lighter weight, more vesicular, and the pumice fragments are less flattened. Except where emphasized by platy joints, foliation is less distinct or absent. Locally the ash-flow tuff contains thin partings of air-fall tuff. The basal part of the welded tuff, as revealed by some of the deeper drill holes, is commonly a black, densely welded, glassy porphyritic rock known as vitrophyre. The vitrophyre consists of small light-colored phenocrysts of feldspar and quartz in an obsidian-like matrix. Its observed range in thickness in the various drill holes is about 2 feet to more than 25 feet.

Most of the welded tuff exhibits faint to distinct foliation, which is the nearly parallel alinement of planar textural and structural elements in the rock, especially flattened vesicles and vugs and, to some extent, collapsed pumice fragments. The foliation is a result of processes of accumulation, compaction, and welding of the ash-flow deposits. The foliation is generally flat lying or gently dipping, but locally it is steeply inclined, as on the flanks of the topographic recess adjacent to the intake tower, where the steep dips probably resulted from ash-flow deposition on a sloping surface or differential compaction during welding. Steep dips in the upper middle part of the right canyon side upstream from the key trench may have resulted from secondary flowage during cooling. Lineation commonly is associated with foliation, but is much less prominent. It is the nearly parallel alinement of linear features in the rock, especially elongate phenocrysts, pumice and rock fragments, and vesicles, and probably resulted from processes of accumulation or secondary flowage.

The welded tuff at the damsite as well as elsewhere along the canyon is characterized by the presence of prominent and abundant joints, which are more or less planar fractures that occur individually and in sets that are roughly parallel. Most of the joints probably resulted from tensional stresses caused by cooling and thermal contraction of the rock after it solidified. The joint system consists mainly of intersecting high-angle (steeply dipping to vertical) joints and low-angle (gently dipping or relatively flat lying) joints. The attitude, or orientation, of individual joints and of joint sets may vary somewhat both laterally and vertically (see fig. B9).

Most of the high-angle joints are nearly vertical and trend northwest. The dominant set strikes about N. 25° W. to N. 30° W. and is well developed in the canyon-side portions of both abutments and in the rocks intersected by both outlet tunnels. A second major set that strikes about N. 60° W. to N. 70° W. is well developed in the lower upstream part of the right abutment, throughout the river outlet works tunnel, and in the downstream part of the auxiliary outlet works tunnel. Major joints and fissures encountered in the far-right segment of the key trench strike about N. 75° W. to N. 80° W. In most of the spillway excavation the principal joints strike about N. 40° W. Northeast-trending high-angle joints are also present in the welded tuff; at the damsite they are generally subordinate to the northwest-trending sets, but in canyon-wall exposures 1 to 2 miles downstream they are the most prominent joints. The patterns of high-angle joints are particularly complex in the channel section (cutoff trench) and very irregular in the upper part of the spillway excavation.

Individual high-angle joints have been traced on construction exposures and on the post-failure rock surface of the right abutment for as much as 280 feet, but most mapped joints are between 20 and 100 feet in length. Many joints are offset several inches or more on intersecting joints. Spacing between high-angle joints generally ranges from a few feet to about 10 feet, but locally is less than 1 foot and may be as much as 60 feet. The width or openness of most high-angle joints is less than one-half inch, but many joints or fissures are as much as several inches wide and some are several feet wide. Preconstruction mapping of rock outcrops in the canyon walls recorded joint widths of as much as 5 feet in the right abutment, and 3 feet in the left abutment. During construction, major high-angle fissures were discovered in the distal parts of the right key trench (max. width about 5 feet) and left key trench (max. width 3 feet) and along the upper haul road on the right side of the canyon upstream from the dam (max. width 3 feet). The additional widening of joints since their original formation during cooling of the welded tuff probably was caused mainly by horizontal tectonic extension and gravitational creep, perhaps aided by lateral stress relief (see Appendix B-1).

Low-angle joints are arbitrarily defined here as joints that dip less than about 15°. They tend to parallel the generally flat lying or gently dipping foliation and locally may superficially resemble the bedding of sedimentary rocks. The direction of dip of the mapped joints is mainly in the northern quadrants; on the right canyon side the predominant direction is west to northwest, into the abutment. The lengths of most of the mapped joint traces are relatively short, less than 50 feet. On the post-failure rock surface of the right abutment several low-angle joints in the upper part of the middle unit of the welded tuff have been traced for about 200 feet, and the jointlike discontinuity (contact) between the middle and lower units has been traced for at least 400 feet upstream of the dam centerline. On the left abutment, the locations of some grout seeps during the pilot curtain grouting suggested the presence of a low-angle joint extending from elevation about 5130 feet 100 feet upstream of the grout curtain to elevation about 5060 feet 300 feet downstream, but its presence was not confirmed by geologic mapping during construction.

Spacing between low-angle joints generally ranges from about 5 to 15 feet except in the upper 70 feet or so of the welded tuff, where a much closer spacing results in a platy structure. The joints in the platy unit are mostly discontinuous and spaced from a few inches to about 1 foot apart; joint widths generally range from about one-fourth inch to 2 inches. In the more massive rock beneath the platy unit, the spacing of low-angle joints locally is more than 30 feet, whereas in zones of closely spaced joints associated with prominent foliation, it may be less than 1 foot. Most of the joints in the more massive rock either are tight or are less than one-half inch in width, but some are as much as several inches in width.

In their natural condition, some joints are open and unfilled and others are partially or wholly filled with clay, silt, silty ash, soil, or rubble, especially near the natural ground surface. Many joint surfaces are stained or coated with iron oxides and manganese oxide, and deposits of calcium carbonate line the surfaces or completely fill some joints. As a result of grouting, many joints were partially or completely filled with cement or cement and sand mixtures, and some especially large voids in the far-right part of the right key trench were largely filled with concrete.

The high permeability of the welded tuff, which has been by far the most important geologic problem in the development of the damsite, apparently is due almost entirely to the presence of open joints or fissures. Information on permeability and its relation to jointing comes mainly from results of water-pressure and pump-in tests on the core drill holes, geologic logs of those holes, joint surveys using a borehole television camera, records of water losses during drilling, and records of grout takes. These data show that joints are more abundant and open, and rock-mass permeability accordingly much higher, above elevation about 5100 than below that elevation. Furthermore, the pilot curtain grouting carried out on the upper part of the sloping left abutment showed that conventional grouting procedures were inadequate to close out the intensely jointed upper 70 feet or so of the welded tuff.

Laboratory tests of rock properties were performed in 1970 on a total of 20 representative core specimens of welded tuff from five drill holes. All of the specimens were from below elevation 5040. Specimens from two of the holes were from the approximate alinement of the river outlet works tunnel, two of the holes were located at the power plant site, and specimens from a hole on the right abutment were from the approximate alinement of the auxiliary outlet works tunnel. Average values from the tests were as follows: modulus of elasticity, 1.8 million lb/in^2; Poisson's ratio, 0.16; unconfined compressive strength, 6,168 lb/in^2; absorption, 4.13 percent by weight; and specific gravity, 2.32 (range 2.24 to 2.40). No in situ values were obtained.

SEDIMENTARY DEPOSITS UNDERLYING THE WELDED TUFF

Unconformably underlying the welded tuff at the damsite are various sedimentary deposits of lacustrine, alluvial, and pyroclastic origin and of probable Pliocene age. In reports and drawings by the Bureau of Reclamation these deposits generally are referred to collectively as "sediments" (as shown in fig. B4) or "lakebed sediments." For convenience, in this report the deposits also will be referred to as sediments, although some of them are moderately indurated. Fragmentary information about these poorly known deposits has come mainly from drill holes, commonly with poor or no core recovery, and to some extent from deep grout holes. The sediments are not exposed at the damsite except very locally in the topographic recess just upstream from the river outlet works intake tower. In all of the construction excavations of the project they were found only at one place, in the invert of the river outlet works tunnel between stations 23+89 and 23+93 (i.e., about 420 feet downstream of the dam centerline), where claystone was exposed beneath welded tuff. Geologic logs of drill holes record a considerable range of lithology and degree of induration of the sedimentary materials. Little is known about the distribution and interrelations of the various lithologies, but sand and gravel and variably cemented sandstone and conglomerate are commonly present, and thick sequences of lacustrine claystone and siltstone are present beneath welded tuff under at least part of the left abutment and channel section. Thin beds of air-fall tuff and other pyroclastic materials were found below the glassy base of the welded tuff in some drill holes.

The contact between the sediments and the overlying welded tuff is an irregular erosion surface which has a local relief of at least 440 feet and slopes locally steeper than 30° (fig. B4). The highest elevations of the contact were encountered in the left abutment and the lowest elevations in what may have been a paleo-canyon well below the bottom of the present canyon. The thickness of the sediments and the depth and nature of their lower contact at the damsite are not known. The maximum recorded thickness is about 390 feet in post-failure drill hole 651-B, which bottomed in gravel. Drill hole 102, in the canyon bottom, penetrated 230 feet of sediments and bottomed in thick claystone. Drill hole G, in the recess adjacent to the intake tower, penetrated 242 feet of claystone.

The permeability of most of the sedimentary materials is less than that of the highly fractured welded tuff above elevation 5100, but it varies considerably with grain size and texture, apparently being very high in some of the gravels and very low in thick claystone bodies. Data on pressure tests and grout takes are quite limited both in number of determinations and depth of coverage, and generally are difficult to correlate with specific lithologic units. There was no return of drilling water during the drilling of much of these materials. High grout takes occurred in sediments in the downstream row of grout holes at depths somewhat greater than 200 feet between stations 31+00 and 34+00. This was thought to be due to hydrofracturing or displacement of material around the holes at the contact of the sediments with the overlying welded tuff (see summaries of interviews with Bock and Aberle in Appendix C, the Grouting Task Group report). This led to a decision to grout only the upper 10 feet of the sedimentary materials, at least in that area, even though it meant a reduction in the depth criteria for the grout curtain. Grout takes were very low in the left side of the channel section, where the top of the sedimentary materials is topographically high and the grout holes penetrated 10 to 85 feet of sediments.

INTRACANYON BASALT

Basalt is discontinuously present in the bottom of the canyon of Teton River as erosional remnants of a lava flow that filled the canyon to about elevation 5005. It is mostly buried at shallow depths under recent alluvium of the river flood plain. In the dam foundation the basalt is restricted to the left side of the channel section (fig. B4), where it has a maximum thickness of about 124 feet. There it is separated from the underlying welded tuff by a thin deposit, about 4 to 22 feet thick, of older alluvial silt, sand, and gravel. The intracanyon basalt also is present in the power and pumping plant foundation on the left side of the canyon and farther downstream in the spillway stilling basin on the right side. The erosion surface on the basalt exposed in the cutoff trench during construction was remarkably smooth, without local channels, slots, or potholes.

The basalt is dark gray to black, hard, very fine grained, and dense to moderately vesicular. It contains closely spaced, randomly oriented, hackly joints and other fractures which were not mapped because of their ubiquitous nature, complexity, and lack of apparent pattern. Many of the fractures are stained with iron oxides and manganese oxide and coated or filled with clayey material. Although highly fractured, the basalt is a competent foundation rock. Laboratory tests of rock properties of the basalt were performed in 1970 on a total of seven representative core specimens from two drill holes at the power and pumping plant site and one drill hole at the spillway stilling basin site. These specimens were comparable to the basalt in the channel section of the dam foundation. Average test values were as follows: modulus of elasticity, 7.7 million lb/in^2; unconfined compressive strength, 13,270 lb/in^2; and specific gravity, 2.72 (range 2.50 to 2.86). No in situ values were obtained.

Water-pressure tests in the exploratory core drill holes in the channel section showed the basalt there to be tight and the thin alluvial fill between the basalt and the welded tuff to be permeable.

Surficial Deposits

Surficial deposits in the damsite area consist of wind-deposited silt on the uplands, alluvium in the canyon bottom, and various slope deposits on the canyon sides.

Wind-deposited silt

On the uplands bordering the canyon of Teton River the welded tuff is overlain by windblown silt, or loess, which ranges in thickness from 1 foot or less near the canyon edge to more than 50 feet. The deposits consist predominantly of silt of low plasticity (ML in the Unified Soil Classification system) with small amounts of fine sand and clay. The silt is loose to moderately cemented by calcareous material, which also is present as discrete particles. Borrow area A, on the upland northwest of the canyon, served as the source of the zone 1 material of the embankment.

Alluvium

The flood plain of the Teton River is underlain by young alluvial deposits which have a maximum thickness of about 100 feet at the damsite. During construction of the cutoff trench, these deposits were excavated down to the surface of the underlying welded tuff and basalt bedrock (fig. B4). The alluvium that was exposed in the excavation is composed of two distinct units. The upper unit, about 80 feet in maximum thickness, consists mainly of sand and gravel with come cobbles and boulders, and contains some lenses of silt and clay as much as 2 feet thick. The lower unit, about 20 feet thick, consists of impervious layers of silt and clay with some lenses of sand and gravel as much as 1 foot thick. It is about 330 feet wide as measured parallel to the axis of the dam.

Borrow area C, located in the flood plain upstream of the dam, and the excavation for the cutoff trench served as the sources of the zone 2 material of the embankment. This material is a mixture of sand and gravel with few fines (GW or GP).

Slope Deposits

Large areas of welded tuff in the canyon walls are obscured by a thin blanket of surficial materials that were deposited on the slopes. In the preconstruction geologic mapping (U.S. Bureau of Reclamation, 1971) three categories of slope deposits were differentiated, but it was recognized that they grade complexly into one another and that boundaries between them are approximate or indefinite.

Slopewash is by far the most extensive unit, especially on the upper part of the canyon walls. It consists of a mixture of silty soil and gravel- to boulder-size fragments of welded tuff that has been transported slowly downslope by the action of gravity and running water. The slopewash deposits generally range in thickness from a few inches to possibly 10 feet, but they are more than 30 feet thick along the alinement of the pump discharge line south of the power and pumping plant. Thin silt locally covers slopewash on the upper part of the left abutment.

Several patches of talus were mapped in the middle and lower parts of the right abutment area. These deposits, part of which were removed during construction, probably were no more than 5 or 10 feet thick. The talus consists of loose, platy, and angular fragments of welded tuff as much as 2 feet in maximum dimension.

An area of landslide debris and associated slopewash was mapped on the left side of the canyon in the lower part of the topographic recess just upstream of the river outlet works intake tower. The deposit, apparently more than 40 feet thick, consisted of a mixture of silty soil and fragments of welded tuff as much as about 6 feet in maximum dimension. These surficial deposits and the underlying claystone were thought to be unstable. During rapid drawdown of the reservoir at the time of dam failure, a small part of the material in the recess area slid into the canyon. On the right abutment, geologic mapping and exploration prior to and during construction did not indicate any evidence of landslides or instability there.

GROUND WATER

Regional

The report "Ground-water geology of the Rexburg Bench," by Gordon Haskett (1972) of the Bureau of Reclamation, shows contours on the regional water table in the general area lying between the Henrys Fork and the Snake River and including the Teton damsite. In general, the regional water table lies 200-500 feet beneath the ground surface, within complexly interlayered welded tuff, detrital sediments, and basalt flows. The regional slope of the ground-water gradient is westward with many irregularities. In the vicinity of the Teton Dam and Reservoir there is also a perched water table, which prior to reservoir filling was 100 feet or more above the regional water table and also had a general westward gradient. Haskett (1972, p. 12) reported that the Teton River "has no loss in the 8- or 10-mile reach above the damsite, but loses up to 50 cfs in the 5-mile reach downstream." According to Haskett in a memorandum from the Regional Planning Officer to the Regional Engineer, April 1, 1976, flow data from gages on the Teton River showed that during the period November 1974 through September 1975 (just prior to filling) the river possibly had a small net average gain in the 17-mile stretch above the damsite.

Damsite

Pre-reservoir Filling

The ground-water conditions in the vicinity of the damsite prior to filling of the reservoir were determined by the Bureau of Reclamation through a several-year-long program of monitoring selected wells that included periodic water-level measurements, water-quality sampling, and temperature logging. Prior to reservoir filling the perched water table in the vicinity of the damsite was somewhat above river level in the area immediately southeast of the river and as much as 50 feet below river level northwest of the river, as shown in figure 6, reproduced from a memorandum of April 7, 1976, from the Regional Director, Boise, to the Director of Design and Construction, on the subject "Monitoring Ground Water Conditions—Teton Project, Idaho." No seeps or springs were observed along the canyon walls in the vicinity of the proposed dam.

Possible variations in the regional water table, which lies about 100 feet beneath the perched water table, as observed in wells 2 miles west of the damsite, are of interest in connection with the hydraulic uplift hypothesis of failure advanced by Marshall Corbett. He suggested the possibility that the abnormally high runoff in the spring of 1976 could have led to a large recharge in the regional aquifer and created "overpressure" in the water in the sediments beneath the welded tuff in the vicinity of the reservoir and dam, and thus caused disturbance of the dam. The possibility of abnormally high pore pressures being created in the period January-June 1976, was examined by E. G. Crosthwaite, Water Resources Division, U.S. Geological Survey (written communication, September 1976). He has found that by using annual records, including those for 1971, another high runoff year, the regional water table near the dam shows an annual low about May-June and an annual high in September-November. In his opinion, ground-water recharge by an annual snowmelt-runoff event does not cause a rise in ground-water levels at the damsite during the period January-June.

General Ground-water Conditions During Filling of the Reservoir

Pertinent data concerning the general rise of ground-water levels in wells in the region of the damsite are shown in figure B6. Additional information is contained in a memorandum of April 1, 1976, from the Regional Planning Officer to the Regional Engineer, on the subject "Water Table Behavior During Early Stage of Filling Teton Reservoir" (prepared by Gordon Haskett), some of which is quoted below.

1. "Aquifer testing of Teton Dam was confined to an area near the right abutment, plus numerous conventional packer tests in scattered exploratory drill holes. Transmissivity, permeability, and storage were calculated to be about 0.5 ft^2/min., 1,500 feet/year, and 0.02. The low yield of irrigation wells

north of the dam is conformable with these calculated values."

2. "On the north side of the reservoir where there is most control, the rise of the water table and arrival times form a pattern which indicates a uniform advance. This favors a belief that at least to the present pool elevation, we are dealing with a fairly uniform rock mass, not cut by continuous open channels."

3. "On the south side of the reservoir, the geology is more complicated. The underlying sediments show considerable relief as apparent on the cross section [figure B6].... The sediments appear to have retarded the reservoir effect.... In DH 503, the rate of rise may change when the water table tops the sedimentary ridge and is in welded tuff."

A memorandum of June 2, 1976, on the subject "Water Table Response to Initial Filling of Teton Reservoir" to Chief, Drainage and Groundwater Branch, Bureau of Reclamation, from R. W. Ribbens, contains an analysis of changing water levels in observation wells between October 6, 1975, when reservoir filling began, and March 15, 1976. The analysis was made to determine the mathematical models appropriate for estimating ground-water table rises at various distances from the rising reservoir and losses into the region northwest of the reservoir by ground-water flow. Contours on the water table as of June 1, 1976, in the vicinity of the Teton Dam are shown in figure B7, which is reproduced from Bureau of Reclamation drawings 549-100-116B.

Some analyses of behavior of ground water in the area of the right abutment during the period subsequent to that of Mr. Ribben's study, from March 15, 1976, through failure to June 20, 1976, are contained in Appendix B-2 of this report of the Geology Task Group. These analyses are derived from all available basic data. Although the data are inadequate to determine changes in ground-water levels downstream from the grout curtain near the area of failure in the final stages of filling, they do indicate: (a) abrupt changes in the rates of rise of water levels in observation wells located upstream from, just beyond the end of, and just downstream from the end of the grout curtain; and (b) abrupt changes in the differences in water levels among pairs of these wells during the periods April 5-15 and, more importantly, May 13-19, 1976. These departures from a steadily changing state can be interpreted as responses to (a) increased filling rates of the reservoir, which occurred during these periods; (b) influx from the rising reservoir reaching more permeable zones in the fractured welded tuff; or (c) changes in hydraulic characteristics of the welded tuff as the ground-water level rose. Each of these interpretations is subject to largely unanalyzed effects of possible variation in hydraulic characteristics of the rock, such as transmissivity, saturated thickness, and storage coefficient.

GEOLOGY OF THE RIGHT ABUTMENT AND ASSOCIATED APPURTENANT WORKS

The following description of the geology of the right abutment and associated appurtenant works is limited mainly to pertinent local details that supplement the description of general conditions covered in General Geology of the Damsite. It deals almost exclusively with one geologic formation, the welded tuff. Appended to this report is a brief report (Appendix B-1) by Prostka (1977) that describes some aspects of the joints, fissures, and irregularly shaped voids in the welded tuff of the right abutment and discusses the origin of these features.

The areas described here are the abutment area of the right canyon side, the spillway crest area, the right wing of the dam that extends for about 900 feet on the nearly flat upland beyond the spillway, and the auxiliary outlet works tunnel and access shaft.

Right Canyon Side

The canyon side, prior to construction, extended from the flat, alluviated canyon floor at about elevation 5035 to the edge of the upland at about elevation 5300, and had a slope of about 30° to 35° (fig. B5). Welded tuff was exposed in scattered, bold outcrops from near river level up to about elevation 5200, but over most of the canyon side it was covered by deposits of slopewash a few feet thick and a few local thin patches of talus (fig. B3). The 100-foot-deep cutoff trench in the alluvium disclosed that the bedrock profile on the right canyon side below elevation 5035 was very irregular with several benches and intervening steep slopes (fig. B4).

The principal source of information on the geology of the welded tuff in the right canyon side is the detailed geologic mapping by the Bureau of Reclamation of the post-failure rock surface of the right abutment (see Geologic Investigations by the Bureau of Reclamation). The following description is based mainly on the results of that investigation.

Subdivisions of the Welded Tuff

The welded tuff in the right abutment has been subdivided into three informal units, primarily on the basis of differences in jointing and associated foliation (fig. B9). These units are (1) an upper platy unit with mostly low-angle joints, (2) a middle unit with mostly moderately to steeply dipping joints, and (3) a lower blocky, massive unit with near-vertical and low-angle joints. Unit 2 appears to terminate abruptly downstream of the key trench.

Unit 1 is about 55 feet thick. It is characterized by abundant subhorizontal platy joints, which are approximately parallel to the foliation, and much less common near-vertical joints. The platy joints are spaced mostly 2 to 6 inches apart, but some are as much as 2 feet apart. The more closely spaced joints occur in several zones that are each about 5 feet thick in the key trench. The platy joints generally range in width from about 1/4 inch to 2 inches. Some are coated with calcite, and caliche and silt fill many of the joints in the upper 6 feet or so of the unit. Near-vertical joints in unit 1 in the abutment are best documented by the mapping of the bottom of the key trench. The dominant joint set strikes about N. to N. 20° W. Other near-vertical joints strike about N. 60°-90° E., including a throughgoing joint as much as 2-1/2 inches wide that crosses the key trench at grout cap centerline station 11+85. Most of the near-vertical joints are spaced about 2 to 5 feet apart and are tight or less than 1/2 inch wide, but some are as much as 4 inches wide. Most of these joints are planar and smooth, and about half are stained with iron oxides and manganese oxide and coated with calcite. The rock in unit 1 is light to medium pinkish or brownish gray in the upper part and grades to light to medium gray in the lower part. The basal contact with unit 2 is gradational over a few inches; it is at about elevation 5242 in the key trench and at elevation 5245 about 100 feet upstream of the grout cap.

Unit 2, which occurs mainly upstream of the key trench, is about 66 feet thick at the key trench. It is characterized by a complex, blocky pattern of closely to widely spaced joints resulting from a combination of dominant, moderately dipping joints with low-angle joints in the upper part of the unit and with high-angle joints in the lower part. The moderately dipping joints define a broad archlike structure (fig. B1-9 and Appendix B-1); near the key trench they have an average strike of about N. 15° W. and range in dip from about 16° to 38° SW., whereas farther upstream they strike about N. 30° W. and dip about 45° NE. The spacing of these joints is mostly about 2 to 5 feet, but it ranges from less than 1 foot to as much as 8 feet. Unit 2 is medium to dark gray.

Unit 2 can be further subdivided, on the basis of jointing, into two parts, or subunits. The upper subunit is about 17 feet thick just upstream of the key trench. It contains several near-horizontal joints that are parallel to the foliation, are spaced 3 to 8 feet apart, and can be traced for several hundred feet. The lowermost and most prominent of these joints forms the contact with the lower subunit at about elevation 5225, and for about 100 feet upstream of the key trench it is between 1/2 inch and 2 inches in width. In the lower subunit, which is about 49 feet thick, the foliation and joints parallel to it strike about N. 38° to 50° W. and range in dip from about 60° to 85° NE. The spacing of these joints is mostly from 2 to 5 feet, but it ranges from about 1 foot to 10 feet. At least two northeast-trending vertical joints make up part of the face of the post-failure cliff formed by the lower subunit. Most of the joints in unit 2 are tight or less than 1/2 inch wide, but some are as much as 4 inches wide. Calcite coats many of the joints and fills some joints which are as much as 1/2 inch wide. Iron oxides and manganese oxide stain many of the joints.

Unit 2 apparently does not extend more than about 50 feet downstream of the grout cap. At that location, the lower 15 feet or so of unit 2 seems to be abruptly terminated against unit 3 rocks along a prominent near-vertical joint that strikes about N. 20°-30° W. and which has been traced riverward at least 40 feet in unit 3. The extent of unit 2 upstream is not known beyond approximately 400 feet from the grout cap.

The prominent, nearly horizontal contact between units 2 and 3 is an abrupt discontinuity between high-angle foliation above and near-horizontal foliation below as well as between strongly contrasting joint patterns (fig. B9). Even more significant, upstream of the key trench the rock along the contact is brecciated and forms a single irregular zone that varies in thickness from about 2 inches to about 3 feet, or several zones in a vertical interval of as much as 10 feet. The upper and lower boundaries of the contact zone are locally planar, but in general they are wavy and irregular (Appendix B-1, fig. B1-10). In the thicker segments of the zone the most highly brecciated rock is in the lower part. The highly brecciated rock consists of angular fragments that range in maximum dimension from less than 1/2 inch to 2 feet. Numerous openings are present in the breccia zone. They are mostly irregularly shaped and vary in width from 1/4 inch to as much as 8 inches. The rock fragments are cemented together by coatings of calcite as much as 1-1/2 inches thick. The contact between units 2 and 3 has been traced for at least 400 feet upstream of the dam centerline, well into the reservoir area. It is at elevation 5176 in the key trench and at elevation 5184 about 100 feet upstream of the grout cap. Through the bottom of the key trench the contact is a nearly horizontal joint filled with calcite as much as 1-1/2 inches thick.

Unit 3 is characterized by blocky, massive outcrops and widely spaced high-angle and low-angle joints. The dominant set of high-angle joints strikes about N. 20°-25° W. and dips mostly about 80°-90° SW. These joints are very prominent in the downstream part of the abutment (fig. B9), where some of them have been traced more than 200 feet. Several have been traced through the bottom of the key trench between grout cap centerline stations 13+30 and 14+23, including one or more joints that cut the lower part of unit 2. A second major set of high-angle joints strikes about N. 60°-70° W. and dips mostly about 75°-90° NE. It is well developed in the upstream part of the abutment. Spacing between the northwest-trending high-angle joints generally ranges from a few feet to about 10 feet, but locally is less than 1 foot. These joints are mostly tight or less than 1/2 inch in width, but many are between 1/2 inch and 2 inches in width, and several are as much as 4 inches wide. A subordinate, widely spaced set of near-vertical joints strikes about N. 40° E., approximately parallel to the canyon. These joints apparently control the location of some of the low cliffs which are conspicuous in the upstream part of the abutment (fig. B9) and probably also control the nearly linear cliff in which the breccia zone along the contact between units 2 and 3 is exposed. Many of these northeast-trending joints are tight or less than 1/2 inch wide, but a few short, grout-filled joints near the outer edge of the broad bench in the upstream part of the abutment are as much as 8 inches wide. Most of the high-angle joints are smooth and stained with iron oxides and manganese oxide and some are coated with calcite. The color of the rock is mostly medium gray.

The low-angle joints in unit 3 are subparallel to the well-developed foliation. They dip mostly 10° or less and generally are inclined in a westerly direction more or less into the abutment. In the upstream part of the abutment these joints form prominent benches (fig. B9). The spacing between the joints generally ranges from about 5 to 15 feet, but locally is more than 30 feet, and in several zones of closely spaced joints associated with prominent foliation it is less than 1 foot. Most of the low-angle joints are tight or are less than 1/2 inch in width.

Coincidence of Geologic Features Between Grout Cap Centerline Stations 13+00 and 14+00

A coincidence of major throughgoing joints and other rock defects occurs along the right key trench between grout cap centerline stations approximately 13+00 and 14+00, an interval that includes the whirlpool that developed about 80 feet upstream in the reservoir shortly before the breaching of the dam. These geologic features, which may have contributed to the failure, are as follows:

(1) A group of prominent, nearly vertical joints that intersects the bottom of the key trench between grout cap centerline stations 13+30 (el. 5198±) and 14+23 (el. 5142±) and trends about S. 20°-25° E., diagonally through the downstream part of the abutment and toward the canyon. Individual joints have been traced continuously as much as 280 feet. Several of these joints are 1/2 inch to 2 inches in width and one is as much as 3 inches in width. This group of joints may include the joint that apparently terminates the lower part of unit 2 of the welded tuff just downstream of the key trench.

(2) The prominent, nearly horizontal contact between units 2 and 3, which is brecciated and contains numerous openings upstream of the key trench for a distance of at least several hundred feet. The contact intersects the bottom of the key trench at about station 13+65 (el. 5176).

(3) A thick, throughgoing zone of low-angle closely spaced joints and prominent foliation that intersects the bottom of the key trench between about stations 12+84 (el. 5220±) and 13+30 (el. 5195±).

(4) A vertical joint that strikes N. 45° E. and intersects the bottom of the key trench at station 13+48 (el. 5191±), in line with the cliff in which the breccia zone along the contact between units 2 and 3 is exposed upstream of the key trench. The reported location of the whirlpool is on this same line.

Slope Stability

Geologic mapping and explorations of the right abutment prior to and during construction did not indicate any evidence of landslides or instability. After the dam failure, an area of slightly slumped rock was mapped between 400 and 550 feet upstream of the dam centerline and extending up the slope about 250 feet from elevation 5060 to about elevation 5210. Many of the high-angle joints in the slumped area widened, some to an opening of as much as 2 feet. This slope movement probably was caused by seepage forces from bank storage return flow during the dam failure and possibly by erosive action by the escaping reservoir water.

Major Fissures Along Haul Road Upstream of Abutment

During construction, a group of 16 large debris-filled fissures was exposed along a haul road on the right canyon side between 650 and 1,450 feet upstream of the dam centerline and within a vertical interval between about elevations 5217 and 5284. These fissures are vertical to near vertical and strike between about N. 40° W. and N. 90° W., with an average strike of about N. 70° W. They range in width from about 1 inch to 3 feet. The debris is composed of loosely to moderately compact silt and angular fragments and blocks of welded tuff. Voids as much as about 3 inches in width are present locally.

Spillway Crest and Auxiliary Outlet Works Access Shaft

In the spillway crest area the upper 15 feet or so of the welded tuff that was exposed during construction is pink to gray, slightly to moderately welded, and contains scattered, angular fragments of pumice as much as 6 inches in size. Near the bottom of the shallow excavation this material grades downward to about elevation 5282 into a moderately to densely welded tuff, part of which is platy. The high-angle jointing as mapped in the excavation appears to be generally random in orientation, but some fairly long, continuous joints trend about N. 20° W. and a minor set trends about N. 80°-85° E. Silty volcanic ash filling is common in the high-angle joints, several of which are more than 2 inches wide and one of which is as much as about 10 inches wide.

In the auxiliary outlet works access shaft, a northwest-trending nearly vertical fissure as much as 1 foot wide was exposed at elevations about 5270 to 5280. Calcite deposits as much as 2-1/2 inches thick line the walls of the fissure and some ash filling is also present. In addition, several other open or partially to completely filled joints as much as 3 inches wide were encountered at lower elevations in the shaft.

Right Wing of Dam

The right wing and key trench extend for about 900 feet beyond the spillway. During preconstruction investigations in this area, borehole television camera observations disclosed many cracks and joints of apparent random orientation in the upper several hundred feet of the welded tuff. Most of the cracks that were recorded range from 1/10 inch to 1/2 inch in width and the widest is less than 2 inches in width. The construction geologic map of the key trench shows that the high-angle joints in the bottom of the trench mainly trend nearly north-south or east-west. Most of them are less than 1 inch wide, but some are as much as 1 foot wide and are coated with calcite deposits and filled or partially filled with silt and rubble.

During excavation of the right end of the key trench, unusually large open fissures were found crossing the trench near station 4+00. The two main fissures are essentially vertical and have an average strike of about N. 80° W. They are shown on figure B4 of this report, on the construction geologic map of the key trench, and in greater detail on Bureau of Reclamation drawings 549-147-133 and 134 dated April 2, 1974. The following description of these and some nearby fissures is taken from an attachment to a memorandum dated March 14, 1974, from the Project Construction Engineer to the Director of Design and Construction:

> "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 N. 80° 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 small fissure strikes about N. 75° 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 2-1/2 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 1-1/2 feet wide and 1-1/2 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-foot-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."

Holes drilled to explore the extent of the large vertical fissures encountered a possibly continuous subhorizontal zone of voids, soft material, and broken rock about 1 to 5 feet thick approximately 15 to 20 feet beneath the floor of the key trench between stations 3+45 and 4+55.

Auxiliary Outlet Works Tunnel

The auxiliary outlet works tunnel was driven in densely welded tuff of unit 3 for its entire length. The two prominent, well-developed sets of high-angle joints have average strikes of about N. 30° W. and N. 60° W. The N. 30° W. set is dominant except between tunnel stations 21+40 and 27+80 (i.e., about 300 to 950 feet downstream of the dam centerline), where joints of the N. 60° W. set are very strongly developed. generally closely spaced, and are virtually the only joints present. A minor joint set that strikes about N. 30° W. and dips about 10° to 35° SW. was observed in local areas of the tunnel. Spacing between joints generally ranges from about 3 to 10 feet, but in some sections it ranges from 1/2 foot to 5 feet, and locally it is as much as 40 feet. Most joints are tight or less than 1/2 inch in width, but locally some joints are as much as 3 inches wide. Many joints are stained with iron oxides and manganese oxide and filled with clay.

PERMEABILITY OF THE WELDED TUFF IN THE RIGHT ABUTMENT

The permeability of the welded tuff is controlled almost wholly by the spacing, openness, and continuity of fractures rather than by connections between small pores within the rock itself. The welded tuff is thus very inhomogeneous with regard to permeability and also anisotropic, depending on the orientation of the fractures.

During preconstruction studies of the damsite, permeability was estimated by injecting water under pressure, at least two pressures, into sealed-off segments of the exploratory drill holes. The location of drill holes in the right abutment is shown in figure 88. Permeability was further indicated by records of depths at which drilling water was lost during deepening of the drill holes, and by measurements of water take during pump-in tests on completed holes. The spacing, width, and filling, if any, in fractures as recorded in the geologic logs provided additional evidence of potential permeability; also, TV camera survey logs of some of the drill holes gave more precise information on the location, openness, orientation, and filling of the fractures. Grout takes during the pilot grouting program and later during construction of the grout curtain provided supplementary information on the original permeability of the welded tuff before grouting.

Permeabilities in the welded tuff as calculated from pressure tests and reported quantitatively in feet per minute should be regarded only as comparative rather than absolute values. Equipment calibration, except for flow and pressure meters, was not done. Where large rates of flow are measured, lack of calibration can introduce considerable error. The formula used for calculation of permeability is valid only for a homogeneous medium in which injected water spreads uniformly around the drill hole rather than through fractures. Thus, the figures reported grossly misrepresent the actual velocity of flow through fractures; nevertheless, they offer comparative measures and are corrected for expected greater flow for tests made under higher static heads at greater depths.

As might be expected, permeabilities as determined from exploratory drill holes, and also as indicated during grouting, varied widely both from place to place and with depth at any one place. As indicated in the description of the general geology of the damsite, joints and fractures are more prevalent in the upper parts of the welded tuff. This is reflected by a general and significant increase in measured permeabilities at shallower depths in drill holes in the right abutment.

In figure B10 some of the various measures and indications of permeability are brought together and presented for two representative sections of welded tuff along the key trench in the right abutment. Figure B10A is a section between stations 12+00 and 15+00 and figure B10B is a section at the right end of the key trench between stations 0+00 and 5+00.

Pre-grouting permeabilities measured in drill holes 203 and 204 in the vicinity of section A were generally low, although there was total loss of drilling water at shallow depths in both drill holes. Grout takes in the area of section A were generally significantly greater nearer the surface of the key trench. Measured permeabilities in drill holes shown in section B were much greater than in section A and quite variable. They were significantly greater above elevation 5100 than below. Grout takes similarly increased upward from about that elevation, although with exceptions and reversals. The very high grout and concrete consumption close to the base of the key trench resulted mostly from treatment of large open fissures in the general vicinity of stations 3+50 and 4+35.

Post-grouting permeabilities in the area of section A, as determined by pressure-testing in the short post-failure 600-series drill holes, were in general greater than recorded before grouting in drill holes 203 and 204. This could be attributed to loosening of the jointed rock and damage to the grout cap during the erosion of the right abutment at the time of failure. Deeper testing of the grout curtain in this vicinity has not yet been completed. Drill hole 633, whose collar is 13.5 feet upstream from the grout curtain centerline, is inclined 70° from the horizontal in a downstream direction and extends to a depth of 90 feet (to about elevation 5106). High permeability was measured in depth intervals 50-55 feet and 55-60 feet, where the hole would have been only 4 to 7 feet downstream from the grout cap centerline. There was total loss of drilling water below a depth of 77 feet and leakage of water carrying silt from a vertical joint a few feet downstream from the grout cap.

Post-grouting permeable zones in short segments and total loss of drilling water were recorded at drill holes 501 and 504 located upstream from the grout curtain. Some grout-filled joints as much as 3/8 inch in width were encountered in drill hole 504; grout from grouting operations in the key trench entered drill hole 501 while it was being drilled. In drill hole 505, oriented N. 20° W. and inclined 30° from the horizontal, all drilling water was lost below a depth of 70 feet; an open seam at 171.3 feet allowed rods to drop 0.6 foot after loosening the chuck, and no percolation tests were taken because of caving and raveling conditions in the hole. A pump-in test at depth 199.8 consumed 2,300 gallons in 40 minutes without the water level rising to the surface. Similar tests at hole depth 399.8, when the water level stood at 395.0, consumed 5,705 gallons in a 90-minute test and 7,970 gallons in a 105-minute test. The water level was still at depth 395.0 feet after the tests. Drill hole 505 penetrated rock that lies wholly beyond the end of the grout curtain and no grout was encountered during drilling.

Post-grouting post-failure drill hole 650 showed high permeability in a zone beneath the bottom of the key trench near the fissure exposed at about station 3+50 during excavation of the key trench. If this fissure were vertical it would line up approximately with a zone of permeability and total water loss in drill hole 652 at about elevation 5160-80 and with permeable zones noted in drill hole 501 somewhat lower.

In summary, the rock in the right abutment is very permeable, especially above elevation 5100, except where grouting was effective. Post-grouting tests, while showing the curtain to be generally effective, reveal also local areas, as shown in both figures B10A and B, where leakage through the curtain might have occurred. Ample opportunity for large flows around the end of the curtain is apparent.

DISCUSSION OF ADEQUACY OF PRE-FAILURE GEOLOGIC INVESTIGATIONS

The geologic investigations for Teton Dam and Reservoir were conducted over a period of at least 20 years in several phases from early reconnaissance studies through detailed mapping of construction excavations (see Geologic investigations by the Bureau of Reclamation). The investigations quickly and properly focused on the major engineering geologic problem associated with the site—the high natural permeability of the foundation and reservoir rock resulting from the presence of abundant open joints or fissures. In general, the surface and subsurface investigations appear to have been well conceived and executed and the data obtained to have been adequate to establish the basic geologic features that should be considered in the design. However, in our opinion, the development of a more complete and better integrated three-dimensional picture of the geology of the abutments would have been desirable to better define and presumably focus even greater engineering attention on the critical problem of open joints that would be in contact with the embankment.

Preconstruction Investigations

An examination of the preconstruction geologic map of the damsite (fig. B3) shows that along the dam axis on the left abutment a continuous section of welded tuff was exposed from about river level to about elevation 5285, and numerous core holes were drilled in the canyon side in the vicinity, mostly in connection with the pilot grouting program. On the right abutment, by contrast, exposures of welded tuff along the dam axis were extremely limited because of a thin cover of slopewash and talus, and only three exploratory core holes were drilled in the canyon side riverward from the spillway.

Additional exploratory core holes and a cleanup operation in the right abutment well before construction would have been desirable to develop more information on the spatial distribution, continuity, and interrelations of rock defects. This information could have been used to construct detailed geologic sections similar to those prepared for the power plant area and river outlet works intake area but including data on rock properties, geologic structures, and drilling and testing records that would aid in subsurface correlation of defects within the welded tuff. For example, by the use of such methods it is likely that the most impressive and possibly most important low-angle discontinuity in the abutment, the geologic contact (with breccia zone) between the middle and lower units of the welded tuff, would have been detected.

Construction Investigations

The principal indication of the adequacy of geologic investigations during construction is the documentary record of construction geologic drawings and reports, annotated panoramic photographs of the steep slopes of surface excavations, and travel report memoranda. As judged from this record, the monitoring of geologic conditions during construction was generally adequate to verify basic geologic conditions determined by the preconstruction investigations. Geologists' field notes and planetable sheets have not been examined in this review.

During the construction investigations the final excavated surfaces were mapped in detail except for the intracanyon basalt exposed in the cutoff trench, several areas of welded tuff along the key trenches, and a small area of welded tuff in the lower upstream part of the right abutment which was covered by a haul road. The joints in the intracanyon basalt were intentionally not mapped because of their ubiquitous nature, complexity, and lack of apparent pattern. According to the construction geologic maps of the key trenches, almost three-fourths of the bottom of the left key trench was not mapped, including the riverward-sloping lower portion between grout cap centerline stations 25+30 and 27+30 (i.e., between elevations approximately 5100 and 5190). The right key trench was not mapped between the major fissure at about station 3+50 and the right end of the trench, but the downstream wall of the trench in this interval is shown on annotated panoramic photographs. While construction mapping in itself does not assure a safe structure, it normally provides the last opportunity to verify geologic conditions and discover geologic defects. Post-failure mapping of the reexcavated remnant of the bottom of the right key trench upstation from the spillway recorded the presence of several throughgoing high-angle joints that were not mapped during construction; one of these joints is as much as 2-1/2 inches in width.

No low-angle joints are shown on the construction geologic map of the right key trench, presumably because of difficulty in distinguishing such joints from the foliation. However, a Bureau of Reclamation compilation of panoramic photographs taken during construction with geologic overlays of exposures of the right abutment and key trench upstation from the spillway shows numerous prominent though very short traces of mostly closely spaced near-horizontal to low-dipping joints in the trench walls and adjacent abutment. Furthermore, post-failure mapping of the reexcavated remnant of the bottom of this segment of the key trench recorded (on geologic sections) the presence of many low-angle joints, including several zones of closely spaced joints associated with prominent foliation. In our opinion, a special effort should have been made during construction to map and portray low-angle joints on plan views of the sloping surfaces.

More complete information on the openness of individual joints exposed at the ground surface is desirable than appears on the construction geologic maps of the damsite. For example, on most of the construction geologic map of the abutments, notations about the openness of specific joints were not provided unless the widths of the joints were at least 0.1 foot; and although many joints were described as open, it is not clear in each case whether they actually had voids along them or were partially or wholly filled. Much of the uncertainty probably stems from ambiguities associated with the geological profession's nomenclature of joints. To the layman, an open joint is literally open, with a void space between the rock walls of the fracture. To many geologists, however, openness refers to the width of a joint, which may actually be filled, rather than to the presence of void space along the joint. Data on joint openness would be clearer, as well as more complete and explicit, if a given joint or segment of a joint were described as to its total width, presence and thickness of filling or coating within that width, and width of void.

The foliation of the welded tuff was duly noted in drill-hole logs and various reports and maps of the damsite and reservoir area. However, in the detailed investigations of the abutments, especially in the vicinity of the key trenches, a greater effort should have been made to map and record the attitude of the foliation where it departs appreciably from the general near-horizontal orientation. Such anomalous orientations may be important clues to the presence of other geologic structures of engineering significance. An example is the association of an abrupt change in attitude of foliation with an equally abrupt change in character of jointing at the horizontal, brecciated contact between the middle and lower units of the welded tuff in the right abutment, mainly upstream from the grout curtain. Neither the high-angle foliation nor this significant low-angle discontinuity was mapped prior to failure of the dam.

Investigations of Geologic Features Not Related to Dam Failure

Many aspects of the local and regional geology were reviewed by the task group as was necessary in the process of identifying those geologic features of significance in the failure of the dam. We feel that some of the features not directly related to the actual failure nevertheless pertain to the safety and efficiency of the dam and reservoir. Hence, they deserved greater attention in the investigations, and it is appropriate to note them here as a matter of record and for their possible value to the planning and conduct of investigations for other dam projects. In saying this, we also recognize that the state-of-the-art of geologic investigations for engineering projects is constantly changing and that the Bureau of Reclamation is one of the leaders in this field.

The configuration of the irregular buried surface of the pre-welded-tuff sedimentary materials (including pyroclastic deposits) and the physical properties of those materials are important because of their possible influence on reservoir seepage loss and ground-water behavior during reservoir filling and operation, especially in the vicinity of the left abutment. In addition, the topography of the buried surface is at least locally reflected in geologic structures within the welded tuff as in the apparent arching of foliation over the claystone "high" in the topographic recess just upstream from the river outlet works intake tower. It is likely that some local and possibly significant variations in the character of jointing may also be related to the configuration of the ground surface on which the ash flow was deposited. A Bureau of Reclamation memorandum dated April 1, 1976, prepared by Gordon Haskett noted that the configuration of the buried surface is poorly defined away from the dam-axis exploration. Even along the dam axis, however, there are sizable gaps in knowledge about the position of this geologic contact; for example, beneath the spillway crest and most of the right key trench and beneath the sloping lower part of the left key trench. In addition, the physical properties and distribution of the various sedimentary materials below the contact also are not well known. More information on both the configuration of the contact and the character of the sedimentary materials would have permitted a more comprehensive evaluation of the dam foundation and of ground-water changes in the vicinity of the dam.

A set of northwest-trending high-angle faults was mapped in the upper part of the reservoir area, to within 9 miles east of the damsite, during preliminary geologic investigations by the Bureau of Reclamation (Magleby and Sweeney, 1963). Beyond the exposures in the canyon walls, these faults appear as low linear escarpments on the silt-covered upland. Other lineaments, reported but not mapped, are faint escarpments and linear drainageways on the upland near the reservoir area which show up prominently on aerial photographs. These northeast- and northwest-trending lineaments also were thought mainly to reflect underlying rock structures. Apparently no studies were made of these lineaments to determine if they are of fault origin except for a limited investigation of the northeast-trending lineament adjacent to the right end of the dam; and no investigation was made of a suggested northeast-trending fault only 2 miles downstream from the dam (see Regional Geologic Setting). We are also unaware of specific investigations of faults to determine their recency of movement and potential for future activity other than the cooperative Bureau of Reclamation—Geological Survey seismic monitoring program. However, geologic data on recency of faulting may be much more significant than the historic record of seismicity in evaluating fault activity and probable earthquake risk. Some investigations would have been desirable to determine if the eolian silt and surface soil are displaced along faults and lineaments in the vicinity of the reservoir.

The slope stability of the canyon walls of the reservoir area is a potentially important factor in the operation and safety of the project, but we have found no reference to a study or evaluation of slope stability having been made prior to reservoir filling except for the investigation of the landslide in the topographic recess adjacent to the river outlet works intake tower.

COMMENTS ON ADEQUACY OF INVESTIGATION OF PERMEABILITY AND GROUND-WATER LEVELS

Pressure and pump-in tests in the cored drill holes appear generally to have been conducted in a thorough and conscientious manner and to have yielded information on permeability of the rocks in the vicinity of the damsite adequate to indicate that (a) the rocks are highly permeable, (b) the permeability is controlled by joints and fissures, and (c) permeability varies greatly from place to place, and with depth in any one drill hole.

Pre-failure analyses of ground-water movements during reservoir filling were directed toward estimation of water losses by ground-water flow from the reservoir, particularly into the region northwest of the reservoir. These investigations, which included the readings of water levels in some exploratory drill holes near the dam, did not address the problem of dam performance.

For the purpose of evaluating both performance and safety, the design of the dam should have incorporated more observation wells, and they should have been positioned so that an adequate number could register ground-water levels downstream from the grout curtain. Effective monitoring would have required wells to be located, constructed, and observed especially for this purpose; in our opinion, the use of existing exploration drill holes to monitor water levels is rarely satisfactory.

At Teton Dam, no observation wells were emplaced downstream from the grout curtain after completion of grouting in the right abutment. The only exploratory drill hole close to the right abutment downstream from the grout curtain (DH-204) was, in fact, covered by the embankment and made unavailable for water-level monitoring. During the later stages of reservoir filling, when water levels were rising rapidly in the wells that were observed, readings were taken on May 10, 13, 18, 20, 25, and June 1. In our opinion, daily readings should have been made and the data analyzed promptly as acquired. Had this been done, possibly it would have been apparent that changes in the hydrologic conditions of the right abutment during the period May 13-19 warranted special and prompt attention.

CONCLUSIONS

1. The high natural permeability of the rock in the right abutment resulting from the presence of abundant and interconnected open joints or fissures, especially in the upper part of the abutment, was a major factor in the dam failure. Of particular importance was the contact of open-jointed rock with the highly erodible zone 1 materials both upstream and downstream of the grout curtain.

2. A coincidence of major, throughgoing joints and other rock defects occurs along the right key trench between grout cap centerline stations approximately 13+00 and 14+00, an interval that includes the whirlpool that developed about 80 feet upstream in the

reservoir shortly before the breaching of the dam. These geologic features may have contributed to the failure.

3. The closest positively identified faults are about 7 to 8 miles east of the damsite, and they are not known to be active.

4. The records of the seismic monitoring network centered on Teton Dam and Reservoir demonstrate that the failure of the dam was not the result of seismic activity.

5. The surface and subsurface preconstruction geologic investigations generally appear to have been well conceived and executed and the data obtained to have been adequate to establish the basic geologic features that should be considered in the design of the dam.

6. Comprehensive and detailed analyses and interpretations apparently were not made of the spatial distribution, continuity, and interrelations of rock defects within the right abutment. In our opinion, the development of a more complete and better integrated three-dimensional picture of the geology of the right abutment would have helped to better define and presumably focus even greater engineering attention on the critical problem of open joints that would be in contact with the embankment materials.

7. Construction geologic investigations in the right abutment and associated appurtenant works, as evidenced by the available documentary record, were generally adequate to verify the basic geologic conditions determined by the preconstruction investigations.

8. An adequate system was not provided for monitoring ground-water levels to evaluate the performance of the key trenches and grout curtain during filling of the reservoir.

REFERENCES CITED

Algermissen, S. T., 1969, Seismic risk studies in the United States: World Conf. Earthquake Engineering, 4th, Santiago, Chile, Proc., v. 1, p. 14-27.

Christiansen, R. L., 1977 (in process), Quaternary and Pliocene volcanism of the Yellowstone rhyolite plateau region of Wyoming, Idaho, and Montana: U.S. Geol. Survey unpub. manuscript, 344 p., pls.

Christiansen, R. L., and Blank, H. R., Jr., 1972, Volcanic stratigraphy of the Quaternary rhyolite plateau in Yellowstone National Park: U.S. Geol. Survey Prof. Paper 729-B, 18 p.

Haskett, G. I., 1972, Ground-water geology of Rexburg Bench: U.S. Bur. Reclamation, Region 1, Boise, Idaho, 21 p., maps.

King, P. B., and Beikman, H. M., compilers, 1974, Geologic map of the United States (exclusive of Alaska and Hawaii): U.S. Geol. Survey, 3 sheets [1975].

Magleby, D. N., and Sweeney, G. L., 1963, Preliminary geologic report, Fremont dam and reservoir site, Lower Teton Division, Teton Basin Project, Idaho: U.S. Bur. Reclamation, Snake River Development Office, Feb. 1963, v. I, 26 p. plus attachments (photos, geol. maps of dam and reservoir sites); v. II, geologic logs and core photos.

Navarro, R., Wuollet, G., West, J., King, K., and Perkins, D., 1976, Seismicity of the Teton Dam area, June 16, 1974-June 9, 1976: U.S. Geol. Survey Open-File Report 76-555, 23 p.

Oriel, S. S., Prostka, H. J., Schleicher, D., and Hackman, R. J., 1973, Draft preliminary report on geologic investigations, eastern Snake River Plain and adjoining mountains: U.S. Geol. Survey unpub. report transmitted to U.S. Bur. Reclamation July 1973, 6 p., enclosures.

Prostka, H. J., 1977, Joints, fissures, and voids in rhyolite welded ash-flow tuff at Teton damsite, Idaho: U.S. Geol. Survey Open-File Report 77-211.

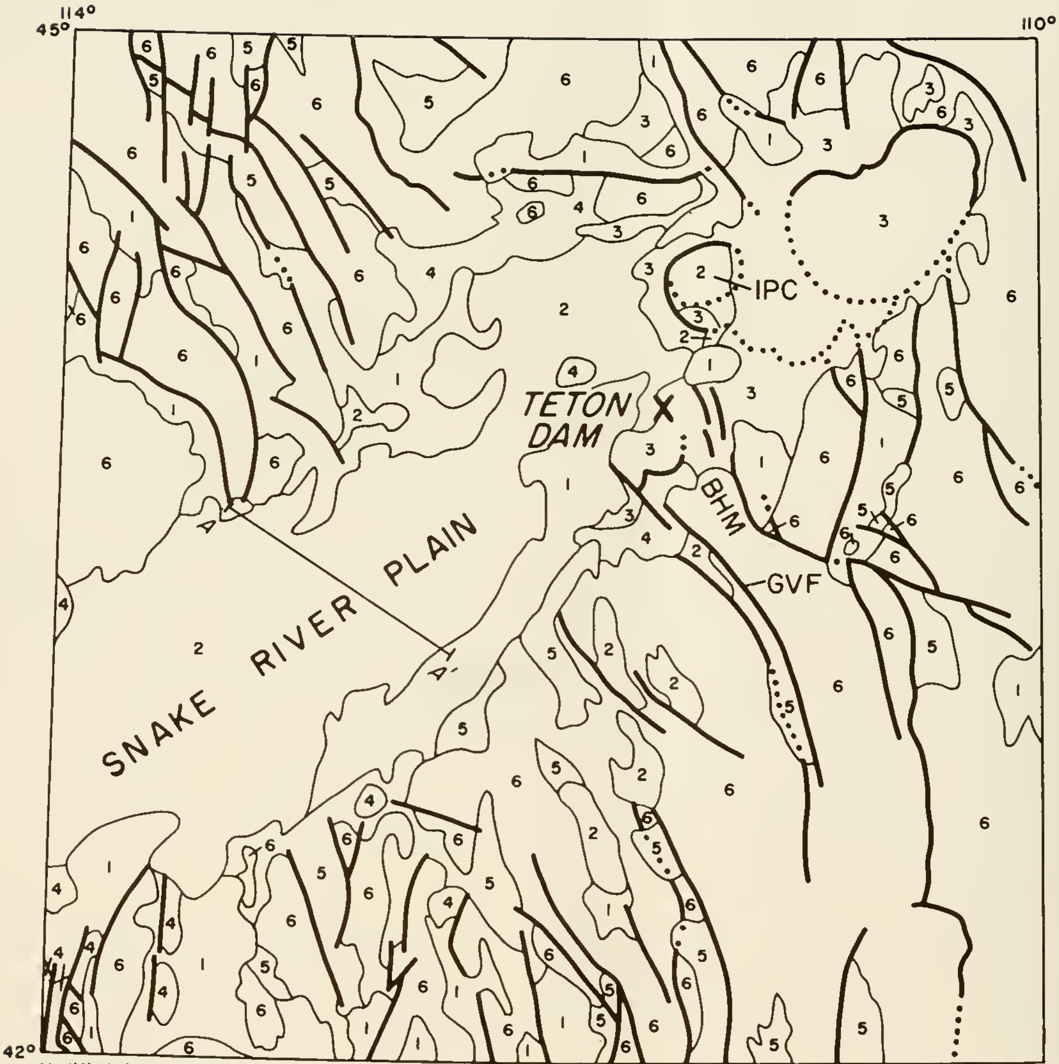
Prostka, H. J., and Hackman, R. J., 1974, Preliminary geologic map of the NW 1/4 Driggs 1^{o} by 2^{o} quadrangle, southeastern Idaho: U.S. Geol. Survey Open-File Report 74-105.

U.S. Bureau of Reclamation, 1971, Preconstruction geologic report, Teton damsite, Lower Teton Division, Teton Basin Project, Idaho: U.S. Bur. Reclamation, Snake River Planning Office, Boise, Idaho, April 1971, 13 + 232 p., plus additional tables, drawings.

——1976, Teton Dam geology: U.S. Bur. Reclamation, Geology and Geotechnology Br., unpub. rept., 44 p., extensive lists and index of documents.

Zohdy, A. A. R., and Stanley, W. D., 1973, Preliminary interpretation of electrical sounding curves obtained across the Snake River Plain from Blackfoot to Arco, Idaho: U.S. Geol. Survey Open-File report, 5 p., 16 figs., 2 pls.

FIGURES

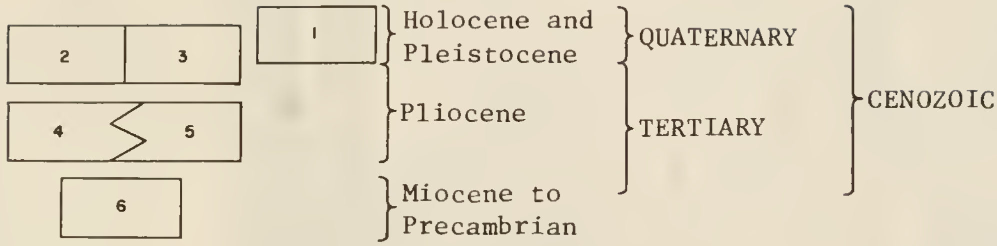


Modified from Geologic map of the United States, compiled by King and Beikman (1974)

0 50 100 MILES

Figure B1.—Generalized geologic map of southeastern Idaho and vicinity, and generalized cross section of eastern Snake River Plain. (Sheet 1 of 2)

CORRELATION OF MAP UNITS



DESCRIPTION OF MAP UNITS

1. Quaternary surficial deposits
2. Quaternary basalt lava flows and minor pyroclastics; may also include some basalt of Pliocene age
3. Quaternary and upper Pliocene rhyolite ash-flow tuffs, lava flows, and tuffaceous sediments of the Yellowstone-Island Park area; includes Huckleberry Ridge Tuff in Teton Dam area
4. Middle and lower Pliocene rhyolite ash-flow and air-fall tuffs and tuffaceous sediments
5. Middle and lower Pliocene tuffaceous sediments; locally contains interlayered rhyolite ash-flow tuffs and lava flows of rhyolite and basalt
6. Miocene to Precambrian metamorphic, plutonic, sedimentary, and volcanic rocks, undivided

———	Contact	BHM	Big Hole Mountains
——····	Normal fault; dotted where concealed	GVF	Grand Valley fault
		IPC	Island Park caldera

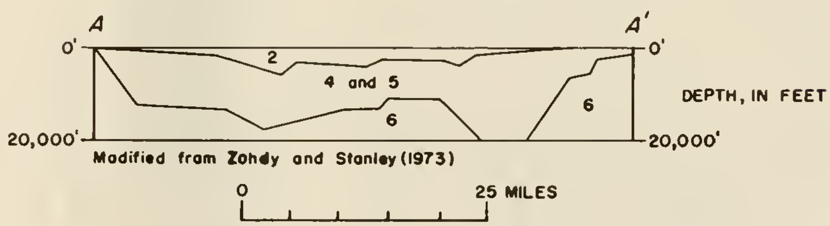
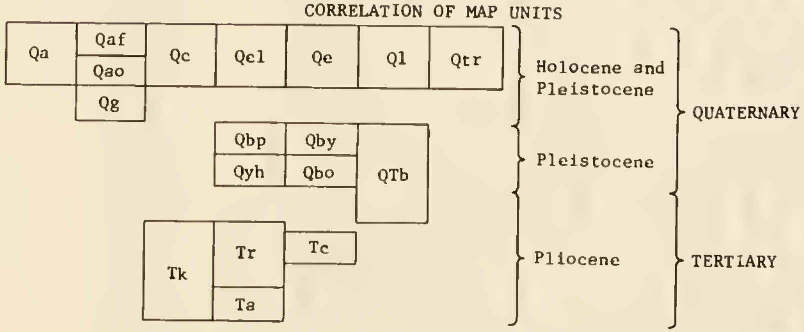


Figure B1.—Generalized geologic map of southeastern Idaho and vicinity, and generalized cross section of eastern Snake River Plain. (Sheet 2 of 2)

CORRELATION OF MAP UNITS



CONTACT--Dashed where approximately located; dotted where It separates areas of detailed mapping from reconnaissance mapping of Paleozoic rocks

NORMAL FAULT--Dashed where approximately located; dotted where concealed; queried where questionable. Bar and ball on downthrown side

STRIKE AND DIP OF BEDS

BASALTIC ERUPTIVE VENT

CLOSED DEPRESSION

DESCRIPTION OF MAP UNITS

SURFICIAL DEPOSITS (HOLOCENE AND PLEISTOCENE)--Age ranges of individual units overlap

Qa Alluvial deposits undivided--Unconsolidated gravel, sand, and silt of flood plains, stream terraces, and alluvial fans

Qaf Alluvial fan deposits--Unconsolidated gravel, sand, and silt

Qao Older alluvial deposits--Unconsolidated gravel, sand, and silt of older terraces and fans

Qc Colluvium--Unconsolidated poorly sorted debris deposited on slopes by mass movements and rockfalls

Qcl Landslide deposits--Unconsolidated poorly sorted slide deposits generally with hummocky bouldery surfaces

Qe Dune-sand deposits--Unconsolidated well-sorted sand

Ql Loess--Unconsolidated well-sorted windblown silt

Qtr Travertine--Light-gray calcareous hot-spring deposits

Qg Glacial till--Unconsolidated poorly sorted glacial till generally lacking distinct morainal form; mostly covered by loess

VOLCANIC ROCKS AND ASSOCIATED SEDIMENTS

Qbp Basaltic pyroclastic deposits (Pleistocene)--Unconsolidated to poorly consolidated deposits of basaltic cinders, scoria, ash, and bombs, in part palagonitic, near basalt vents. Includes some welded spatter

Qby Snake River Group (Pleistocene)--Lava flows of basalt, mostly pahoehoe

Qyh Huckleberry Ridge Tuff (Pleistocene)--Rhyolite welded ash-flow tuff. Extensively covered by loess in Rexburg Bench area

Qbo Older basalt (Pleistocene)--Lava flows of basalt; minor amounts of interlayered coarse- to fine-grained sediments and basaltic pyroclastic deposits. Extensively covered by loess in Rexburg Bench area

QTb Basalt undivided (Pleistocene and Pliocene)--Lava flows of Snake River Group, older basalt, and basaltic pyroclastic deposits in southeast quarter of map area

Tk Kirkham Hollow Volcanics (Pliocene)--Mainly rhyolitic welded tuffs, lava flows, and nonwelded tuffs of several ages in southeast quarter of map area

Tr Rhyolite (Pliocene)--Mainly rhyolitic welded tuffs, lava flows, and nonwelded tuffs of several ages

Tc Conglomerate (Pliocene)--Moderately well consolidated conglomerate

Ta Andesite (Pliocene)--Lava flows of platy-jointed andesite

Figure B2.--Geologic map of the region of Rexburg Bench and the canyon of Teton River. (From Prostka, H. J., and Hackman, R. J., 1974, Preliminary geologic map of the NW 1/4 Driggs 1° by 2° quadrangle, southeastern Idaho: U.S. Geol. Survey Open-File Report 74-105.) (Sheet 1 of 2)

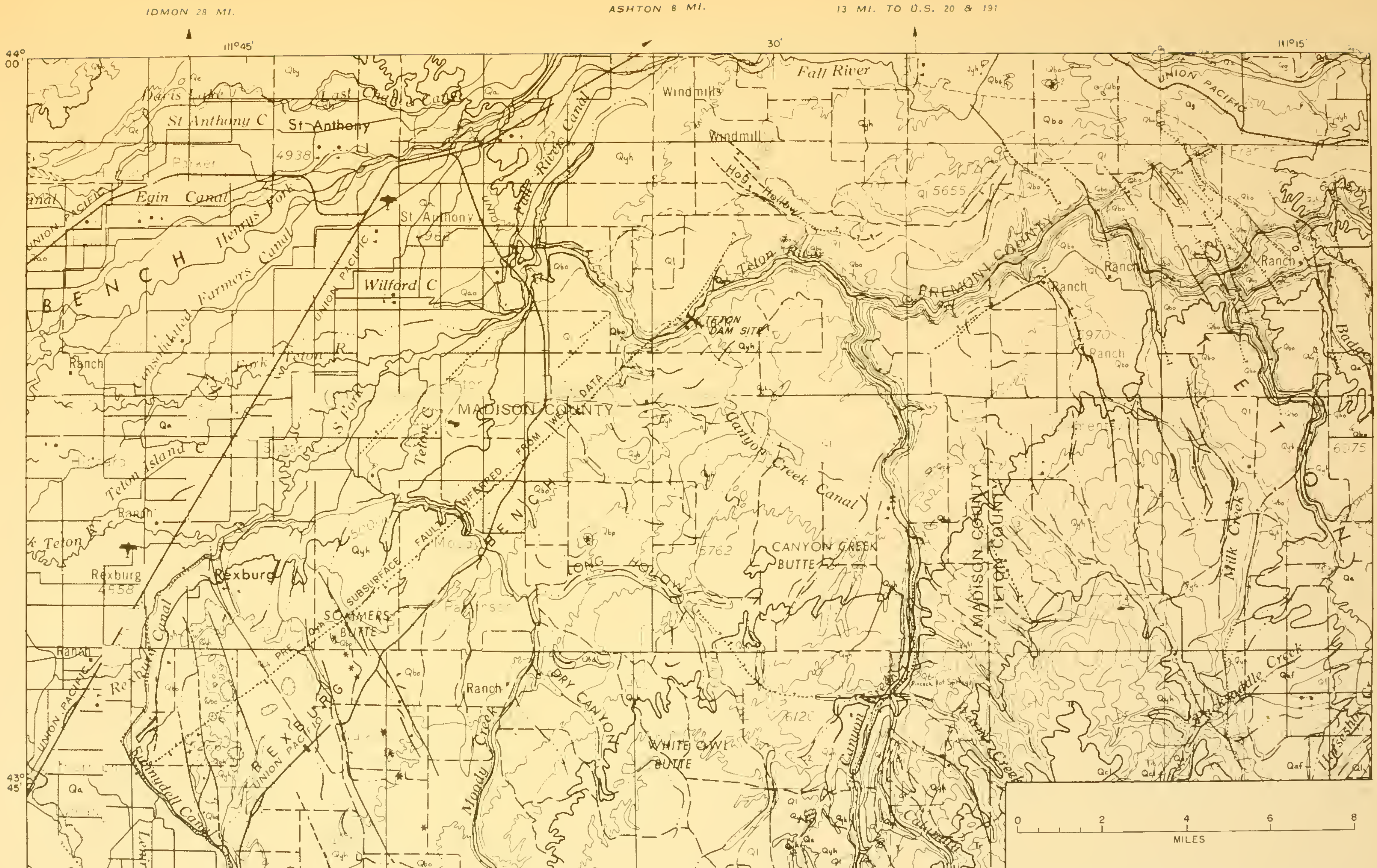


Figure B2.—Geologic map of the region of Rexburg Bench and the canyon of Teton River. (From Prostka, H. J., and Hackman, R. J., 1974, Preliminary geologic map of the NW 1/4 Driggs 1° by 2° quadrangle, southeastern Idaho: U.S. Geol. Survey Open-File Report 74-105.) (Sheet 2 of 2)

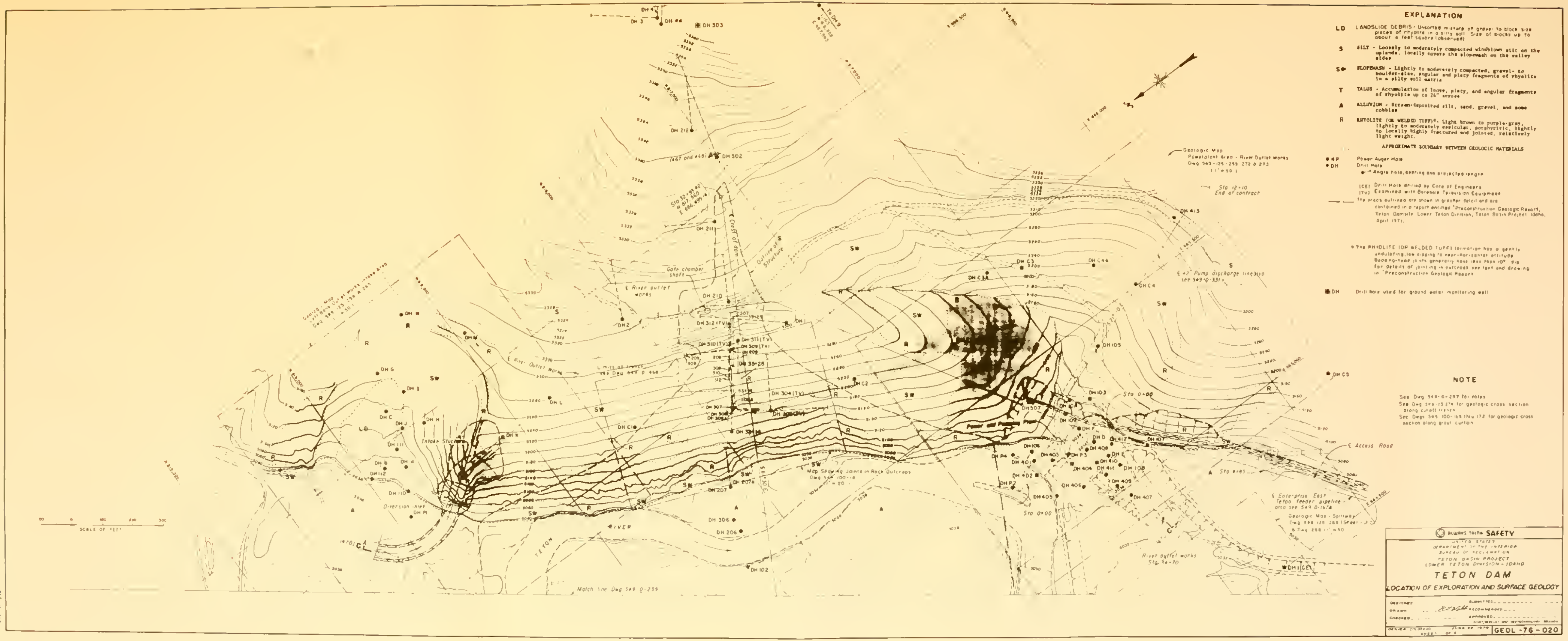


Figure B3.—Surface geologic map of Teton damsite showing locations of exploration. (Sheet 1 of 2)

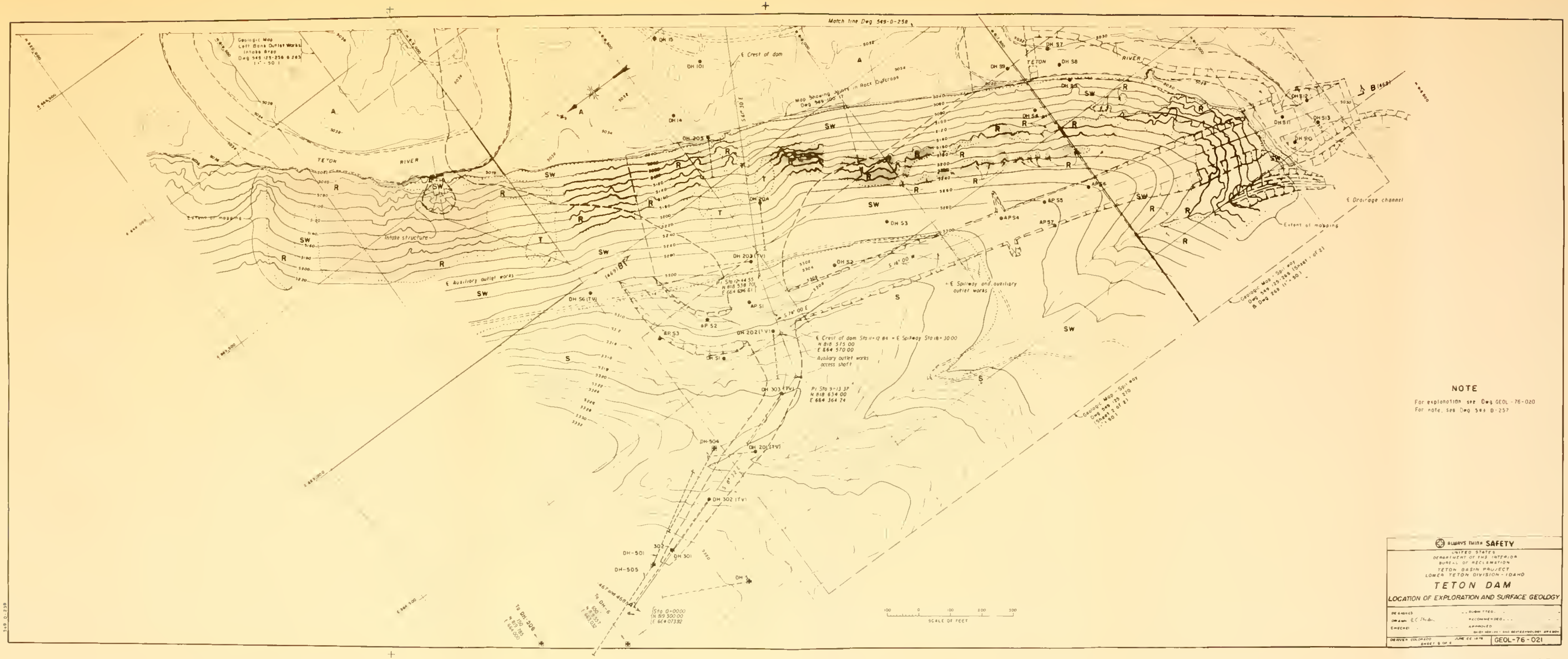


Figure B3 —Surface geologic map of Teton damsite showing locations of exploration (Sheet 2 of 2)

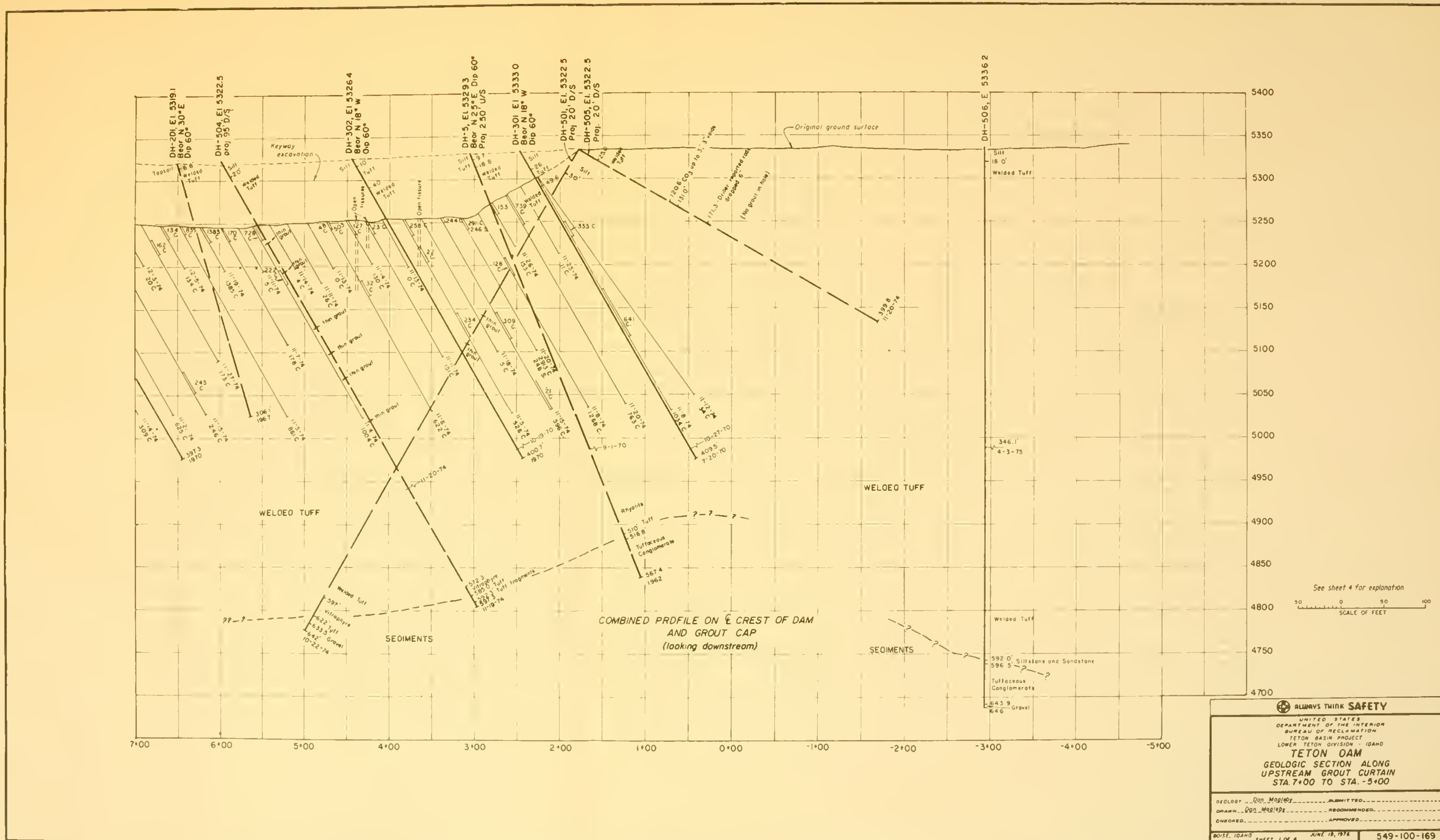


Figure B4.—Geologic section along upstream row of holes of grout curtain, station –5+00 to station 49+00. (Sheet 1 of 4)

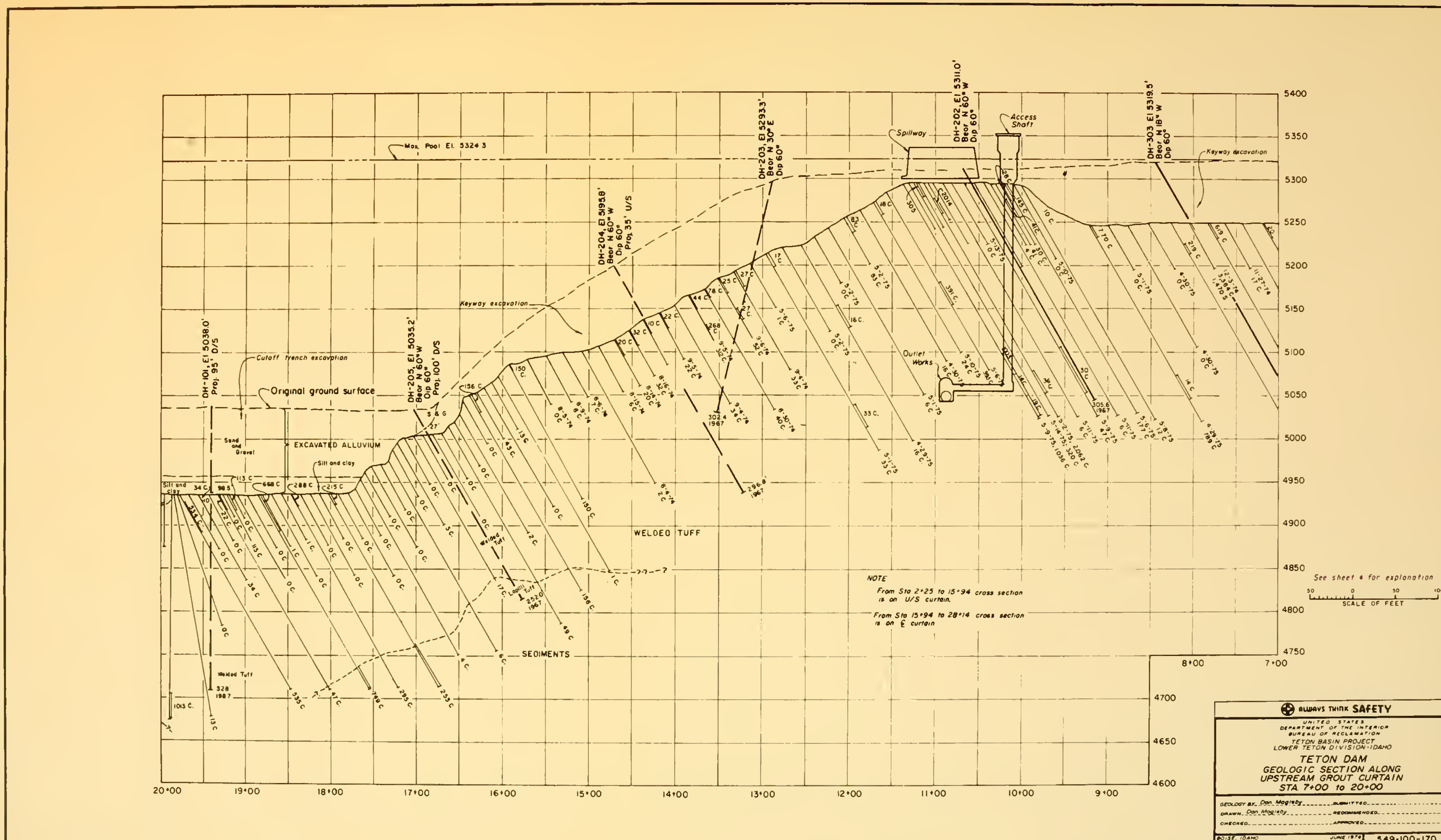


Figure B4.—Geologic section along upstream row of holes of grout curtain, station −5+00 to station 49+00. (Sheet 2 of 4)

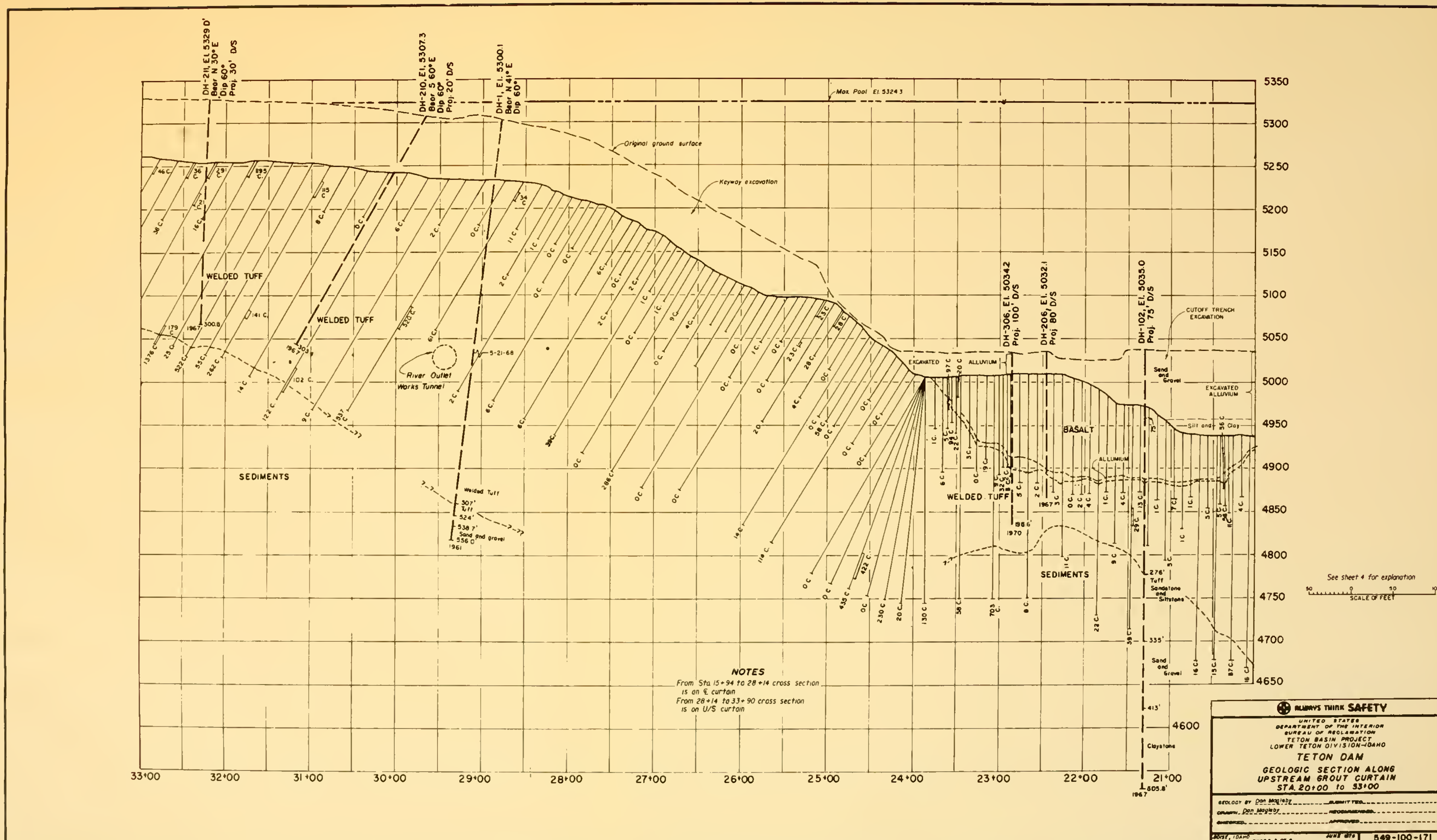


Figure B4.–Geologic section along upstream row of holes of grout curtain, station –5+00 to station 49+00. (Sheet 3 of 4)

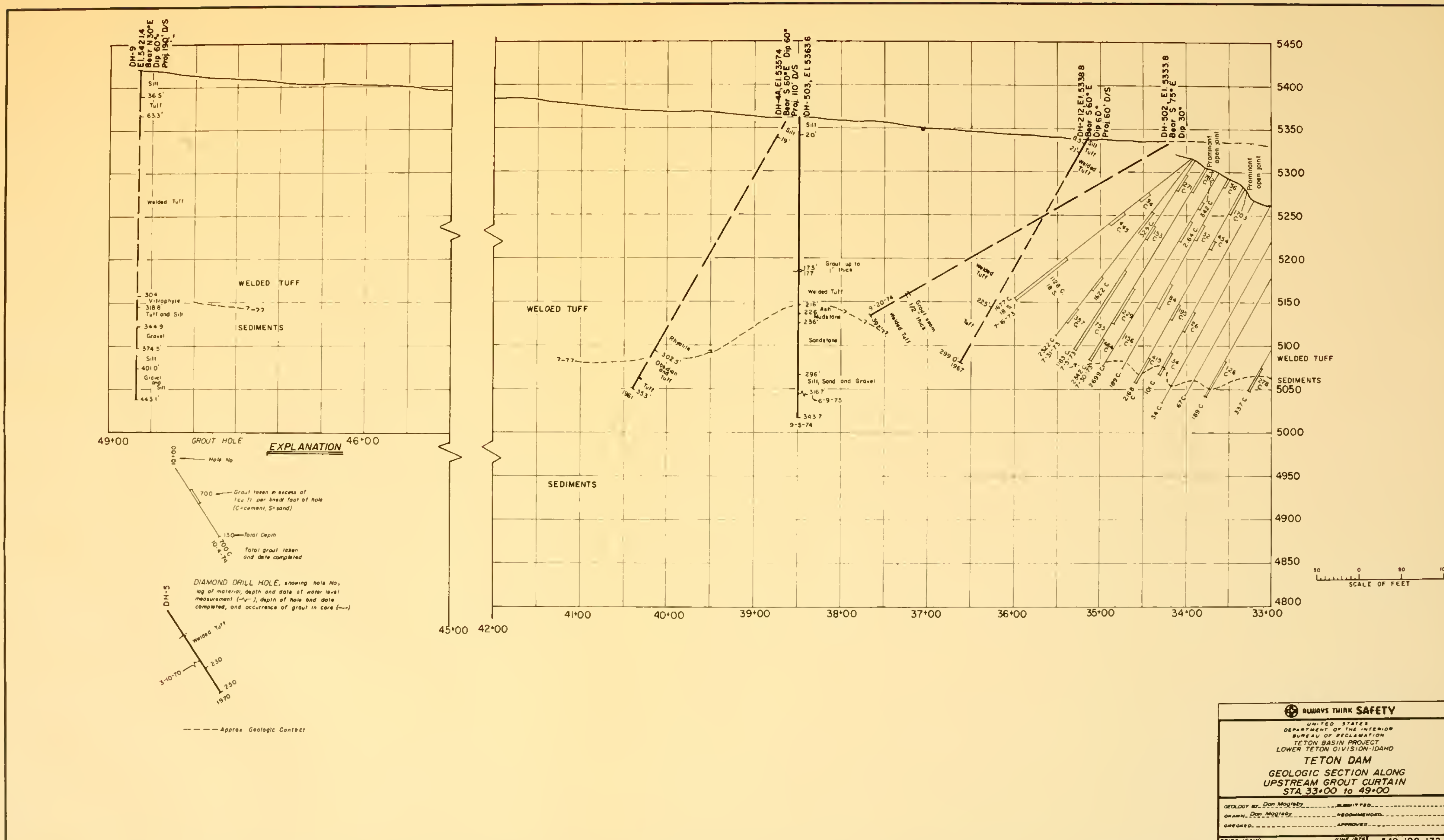
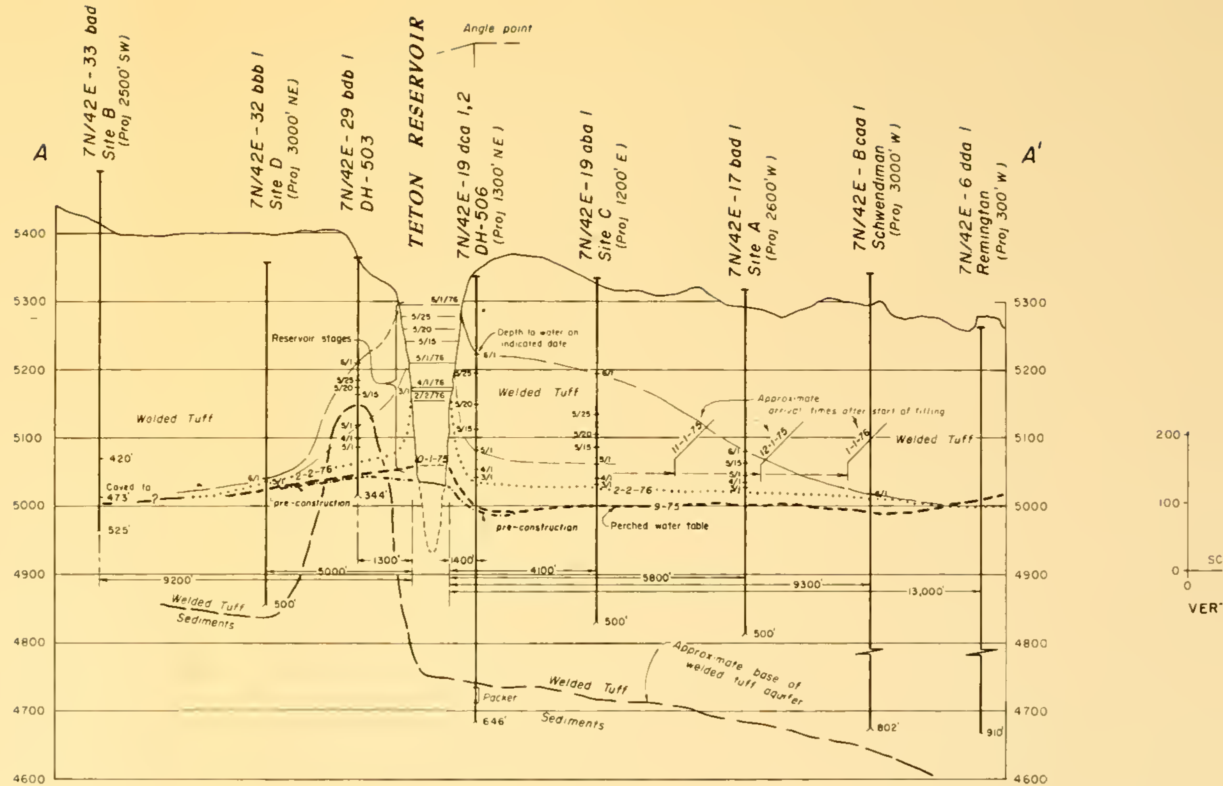


Figure B4.—Geologic section along upstream row of holes of grout curtain, station –5+00 to station 49+00. (Sheet 4 of 4)

Figure B5.—Aerial view of Teton damsite prior to construction. View is looking downstream, southwesterly, at the lower end of the canyon of Teton River. Newdale, Idaho, is about 3 miles from the damsite. The approximate outlines of the dam and appurtenant works are shown. Bureau of Reclamation photograph P549-125-423 by D. Roderick, October 17, 1968.



A
A'
7N/42E - 33 bad
Site B
(Proj 2500' SW)
7N/42E - 32 bbb I
Site D
(Proj 3000' NE)
7N/42E - 29 bdb I
DH - 503
TETON RESERVOIR
Angle point
7N/42E - 19 dca I,2
DH-506
(Proj 1300' NE)
7N/42E - 19 aba I
Site C
(Proj 1200' E)
7N/42E - 17 bad I
Site A
(Proj 2600' W)
7N/42E - 8 caa I
Schwendiman
(Proj 3000' W)
7N/42E - 6 dda I
Remington
(Proj 300' W)
5400
5300
5200
5100
5000
4900
4800
4700
4600
Reservoir stages
Depth to water on indicated date
Welded Tuff
Approximate arrival times after start of filling
Caved to 473'
pre-construction
Perched water table
Sediments
Packer
Approximate base of welded tuff aquifer
2-2-76
9-75
11-1-75
12-1-75
1-1-76
420'
525'
500'
344'
646'
802'
910'
9200'
5000'
1300'
1400'
4100'
5800'
9300'
13,000'
200
100
0
VERT

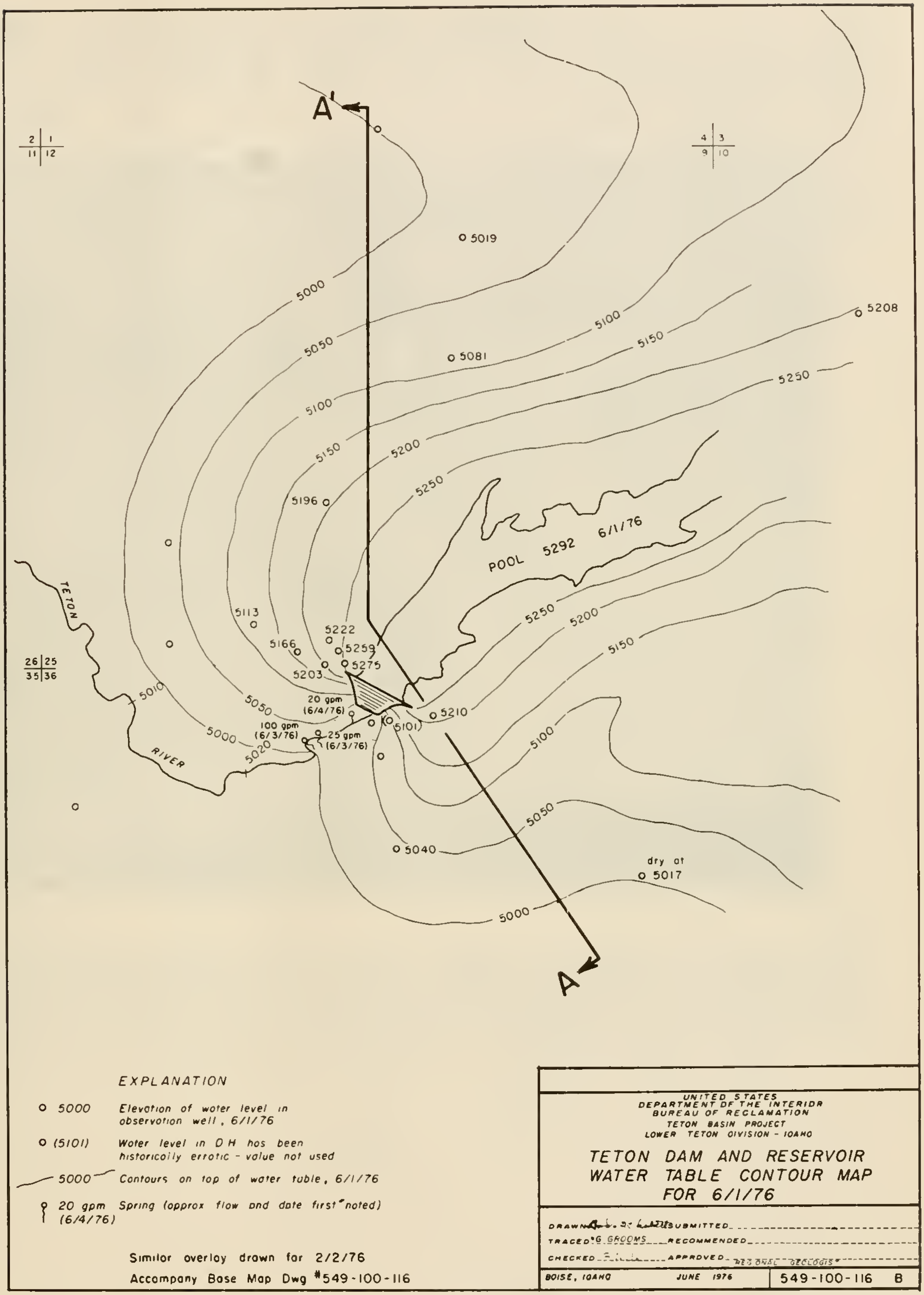


Figure B7.—Teton Dam and Reservoir water table contour map for 6/1/76.

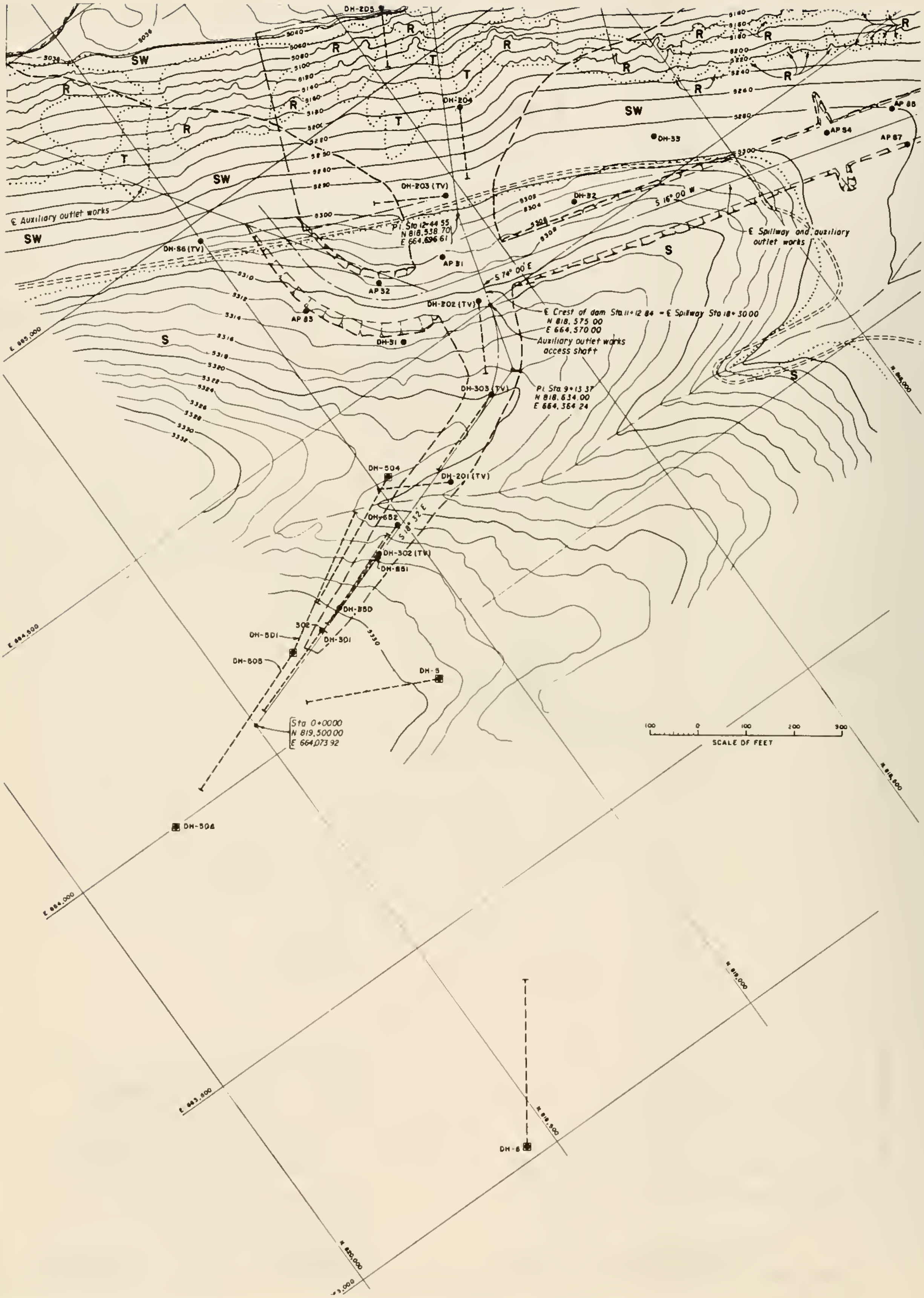


Figure B8.—Right abutment of Teton Dam showing location of structures and selected exploratory drill holes. (See figure B3 for explanation of symbols.)

Figure B9.—Aerial view of right abutment in November 1976, after embankment material removed from key trench. Small area of loose fill on bench in right center is temporary stockpile of excavated embankment material. Grout cap extends diagonally across center of photograph. Gap in grout cap between two throughgoing near-vertical joints is at grout cap centerline stations 13+98 to 14+22. The exposed rock is welded tuff. Upper arrow indicates the nearly horizontal contact between unit 1 (above, with platy joints) and unit 2. Lower arrow indicates the unit 2-unit 3 contact, which extends downstream (to the left) about 50 feet beyond the grout cap, where it apparently is abruptly terminated by a high-angle joint. Dark area in lower center is water from abutment cleanup operation. View to north-northwest. Photograph by Bureau of Reclamation.

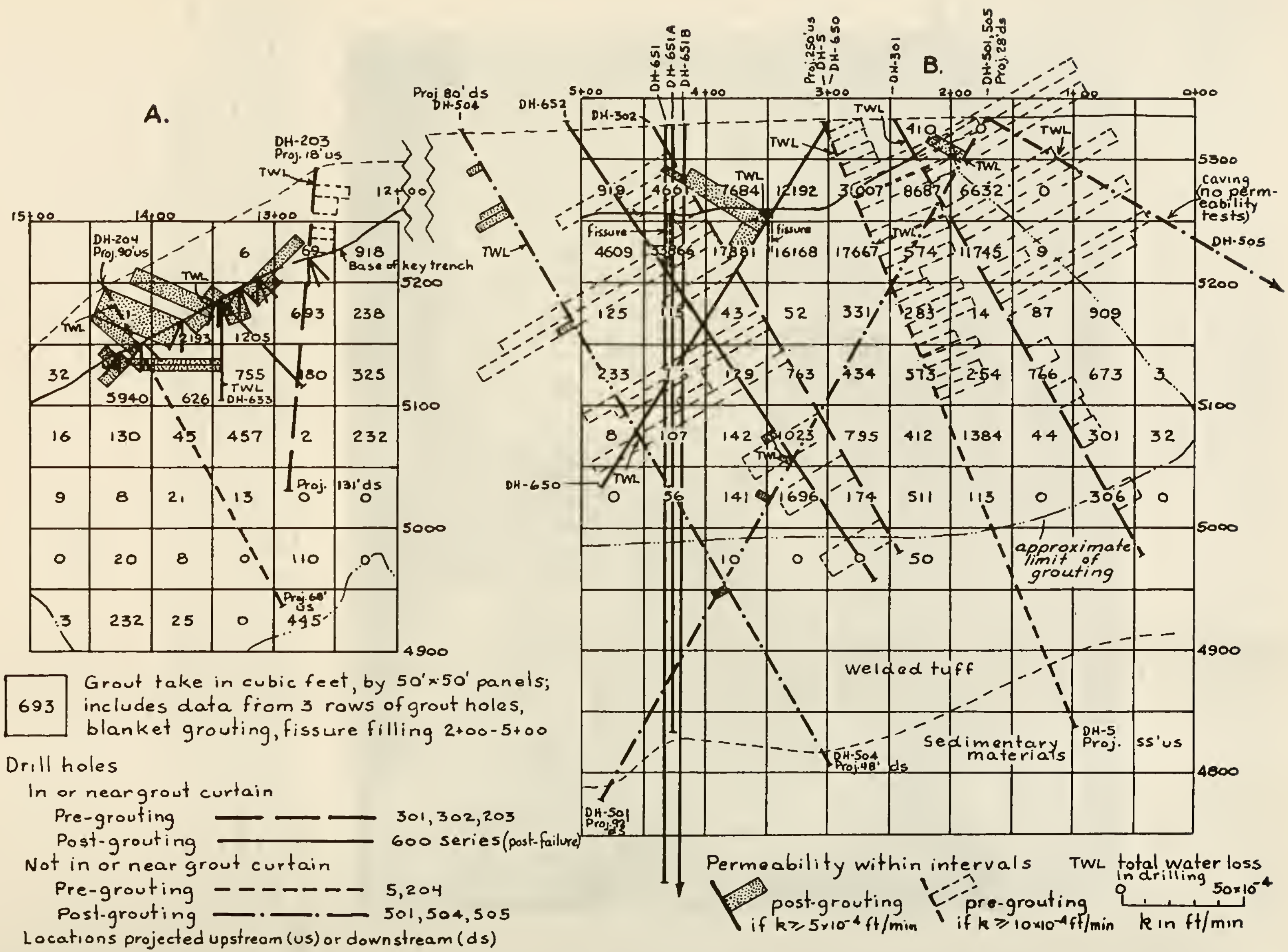


Figure B10.—Grout take and permeability in two parts of section along key trench, right abutment, Teton Dam.

APPENDIXES

APPENDIX B-1

Joints, Fissures, and Voids
in Rhyolite Welded Ash-flow Tuff
at Teton Damsite, Idaho
by
Harold J. Prostka

UNITED STATES DEPARTMENT OF THE INTERIOR

GEOLOGICAL SURVEY

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Open-File Report 77-211

1977

This report is preliminary and has not been edited or reviewed for conformity with U.S. Geological Survey standards or nomenclature.

JOINTS, FISSURES, AND VOIDS IN RHYOLITE WELDED ASH-FLOW TUFF AT TETON DAMSITE, IDAHO

By HAROLD J. PROSTKA

Abstract

Several kinds of joints, fissures, and voids are present in densely welded rhyolite ash-flow tuff at Teton damsite. Older fissures and voids probably were formed in the ash-flow sheet during secondary flowage, which probably was caused by differential compaction or settling over irregular topography. The younger, more abundant fissures are mostly steep cooling joints that probably have been opened farther by horizontal tectonic extension and gravitational creep, perhaps aided by lateral stress relief.

Introduction

This report briefly describes and examines the origin of some aspects of the joints, fissures, and irregularly shaped voids that are present in the rhyolite welded ash-flow tuff of the right (northwest) abutment of Teton Dam, Idaho. Visits to the canyon and damsite were made over a period of several years during the course of regional mapping (Prostka and Hackman, 1974). After failure of the dam on June 5, 1976, the damsite was revisited at the request of the Department of the Interior Teton Dam Failure Review Group. The right abutment was carefully examined alone and with members of the Review Group and, on one occasion, with Donald A. Swanson, U.S. Geological Survey, who provided additional insights and discussion on the origin and significance of the volcanic features exposed there. The terminology and concepts pertaining to ash-flow tuffs that are used here are those developed and summarized in several definitive papers by Smith (1960a, b) and Ross and Smith (1961), in which the processes of deposition, compaction, welding, and cooling of ash-flow tuffs are described.

The rhyolite that forms the walls of the canyon of Teton River and the abutments of Teton Dam has been correlated with the Huckleberry Ridge Tuff, a densely welded ash-flow tuff that has been radiometrically dated at 1.9 million years (Christiansen and Blank, 1972) in Yellowstone National Park.

On Bureau of Reclamation post-failure geologic maps of the right abutment of Teton Dam, the welded tuff has been subdivided into three informal units that are distinguished primarily by variations in prominence or degree of development of foliation and related platy joints, variations in dip of foliation, and variations in dip and spacing of joints (figs. B1-1, B1-2, B1-3). The contact between unit 1 (the upper unit) and unit 2 (the middle unit) is gradational over a few inches; the contact between unit 2 and unit 3 is a breccia zone that is mostly about 2 inches thick, but which laterally grades into an interval about 10 feet thick containing several thin discontinuous breccia zones.

Low-angle Foliation and Platy Joints

Low-angle foliation and related platy joints appear to have resulted primarily from depositional layering and from flattening and collapse of the ash-flow sheet. Zones of prominently developed platy joints are layers that probably had, in addition, more abundant entrapped gases either because of an initial higher concentration of pumice in these layers or because they represent gas-rich pulses of the eruption that formed vesicle-rich or lithophysal layers which subsequently collapsed during cooling and degassing of the sheet. These platy collapsed zones apparently were mechanically weaker than the more massive layers of the ash-flow sheet. During secondary flowage, they formed horizontal zones a few inches to about 1 foot thick of short (less than 6 inches), closely spaced, gently dipping imbricate joints (fig. B1-4). These zones occur between thicker, more blocky layers of the rhyolite and are best developed in unit 1. The layered appearance of the sheet, as seen from a distance (figs. B1-1, B1-2), is due to the presence of these platy zones as well as to the foliation. Elongate pumice fragments in the tuff at the damsite do not everywhere display a strongly preferred orientation; instead, many are randomly oriented, especially in unit 2. This may reflect exceptionally high turbulence within the ash flow during emplacement.

Steeply Dipping Foliation and Joints in Unit 2

Foliation that dips 70° north-northeast to vertical is shown throughout the lower three-fourths of unit 2 on Bureau of Reclamation post-failure geologic maps. Locally, closely spaced joints have developed parallel to the foliation; many of them are slightly open and lined with coatings of silica and alkali feldspar. The steep foliation may be due to a steep primary depositional fabric that has been called ramp structure (Schmincke and Swanson, 1967), although the apparently random orientation of elongate pumice fragments does not lend support to this interpretation.

In addition to the steep joints there are many low to moderately dipping joints, many of which are curved and which define a broad archlike structure in unit 2 (fig..B1-5). The origin of these curved joints is not clear and no explanation for them is offered here.

Figure B1-1.—Oblique aerial view of right abutment, Teton Dam, showing overall appearance of rhyolite welded ash-flow sheet. Note threefold subdivision of sheet upstream (to the right) of the embankment remnant in the key trench. View to west taken June 1976.

Steep Irregular-shaped Fissures

Steeply dipping fissures of lenticular to highly irregular shape, a few inches to several feet in length (fig. B1-6), are rare but are present in a few places in unit 1. These fissures generally dip northeastward like the small imbricate joints in the platy zones of unit 1 (fig. B1-4) and many of the steep foliation joints in unit 2 (fig. B1-5). Voids along all three kinds of features are lined with coatings and replacements of silica and alkali feldspar that were deposited by gases escaping upward through the ash-flow sheet during compaction, cooling, and welding. The similar direction of dip of all three kinds of features and the presence of vapor-deposited coatings in voids along them suggest that they all probably formed at about the same time in reponse to tensional and shear stresses that developed during differential compaction and secondary flowage of the ash-flow sheet. These fissures and joints generally strike northwest, as do the axes of folds in the lower part of the sheet (fig. B1-7) in the vicinity of the damsite, and the predominantly northeast dips suggest that the upper part of the sheet may have slid northeastward over the lower part as a result of secondary flowage that may have been caused by differential compaction of the ash-flow sheet over irregular topography.

Very Steep to Vertical Columnar Joints and Fissures

The most abundant steep to vertical joints have smooth planar surfaces. Most of them are columnar-type joints that formed by thermal contraction during final cooling and consolidation of the ash-flow sheet. At the damsite the northwest-trending joint set is more prominantly developed than the northeast-trending set. Because the amount of separation along joints in volcanic rock that is due to thermal contraction alone is typically much less than 1 inch, the separations of as much as several feet that are found along some joints at Teton damsite must be due to later additional widening of some kind. The amount of separation along these steep joints varies vertically, commonly quite abruptly at intersections with low-angle joints (fig. B1-8); this abrupt variation requires some slippage along the low-angle joints, as indicated in figure B1-9. This

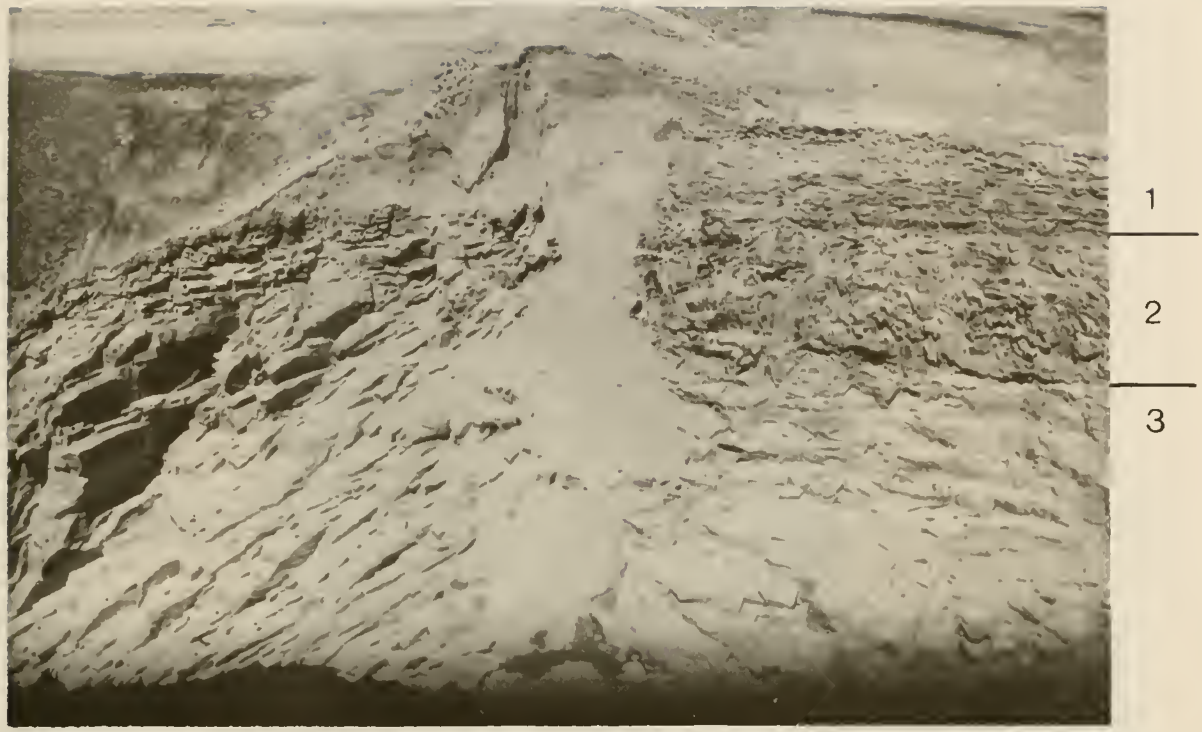


Figure B1-2.—Aerial view of right abutment, Teton Dam, nearly perpendicular to the canyon wall. Note abrupt termination of unit 2 just downstream from the embankment remnant. View to northwest taken June 1976.

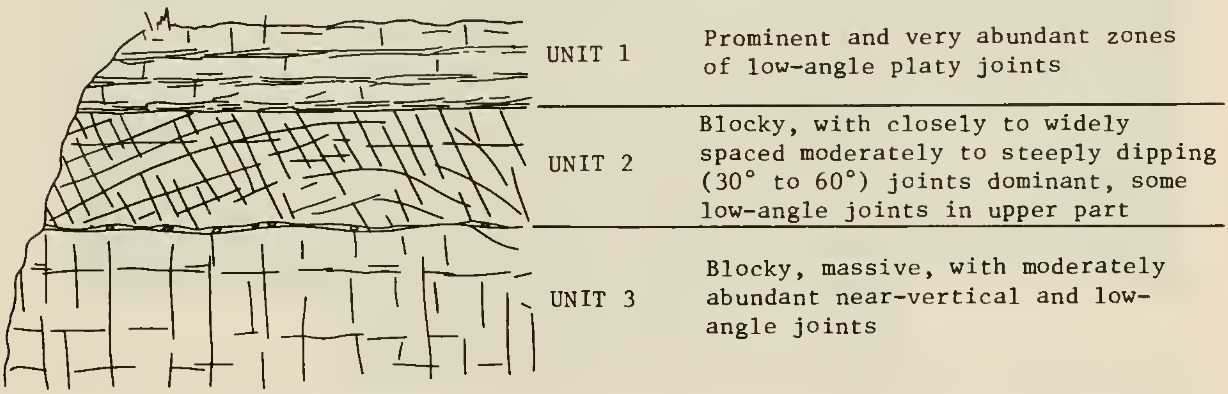


Figure B1-3.—Schematic section of welded rhyolite ash-flow tuff in right abutment, Teton Dam, as exposed after failure just upstream from the embankment remnant, showing principal joint features of the three informal units distinguished on Bureau of Reclamation post-failure geologic maps.

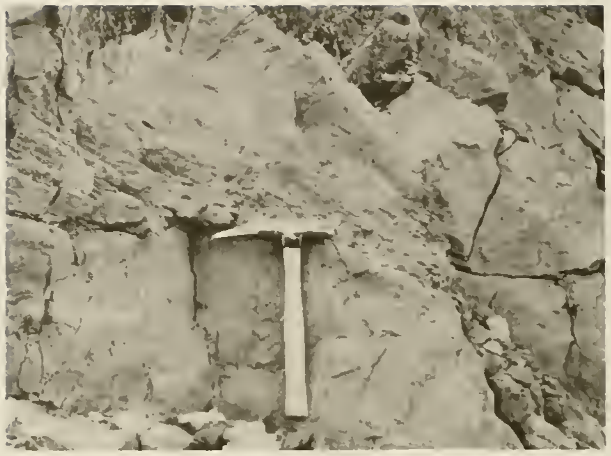


Figure B1-4.—Zone of closely spaced, short imbricate joints in unit 1 exposed along a haul road a short distance upstream from the right abutment. Hammerhead is at the contact between the imbricate zone and underlying blocky welded tuff.

Figure B1-5.—Right abutment just upstream from key trench, showing subhorizontal platy joints of unit 1, archlike form of curved joints superimposed on steep foliation and related joints dipping to the right (north-northeast) in unit 2 (see also fig. B1-3), and widely spaced joints in unit 3. Prominent white lines are engineering reference lines painted on the rocks. The parallel lines are at 50-foot intervals upstream from the dam centerline.

Figure B1-6.—Steeply dipping irregularly shaped fissure about 2 feet wide that probably formed as a viscous pull-apart structure during secondary flowage of the rhyolite. Note small steeply dipping tension cracks along the left side of the large void. This fissure is exposed in unit 1 along a haul road a short distance upstream from the right abutment.

Figure B1-7.—Fold in lower part of ash-flow sheet exposed in right canyon wall less than 1/2 mile upstream from the dam. The axis of this fold is about in line with the arching of foliation over a pre-rhyolite topographic high exposed in the left side of the canyon just upstream from the intake tower.

Figure B1-8.—Large vertical fissure apparently offset along several low-angle joints in unit 1.

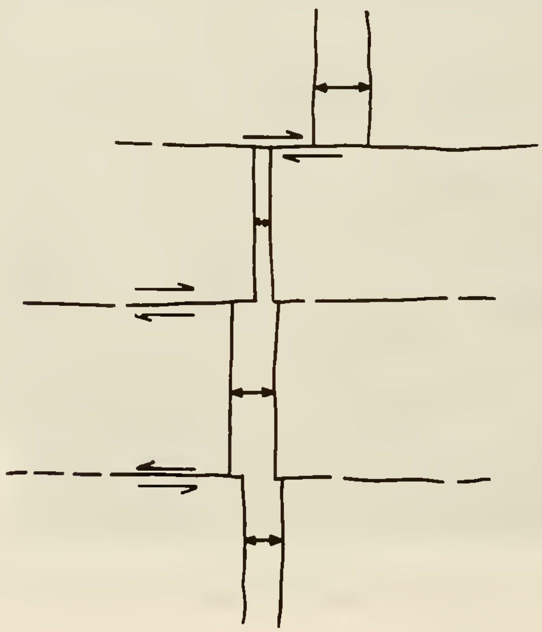


Figure B1-9.—Diagram of vertical fissure; horizontal offsets show how different separation (↔) along a vertical joint is accommodated by slippage (⇌) along low-angle joints in rhyolite at damsite.

relationship between high-angle and low-angle joints is best seen in unit 1 along the upper haul road upstream from the right abutment.

Brecciated Contact Between Unit 2 and Unit 3

Small amounts of lateral slippage have occurred along many minor low-angle joints. Locally, however, some major low-angle joints appear to have individually accommodated the aggregate lateral slippage due to widening of several steep joints, a process which may have formed the brecciated contact between unit 2 and unit 3. This contact is a prominent discontinuity (figs. B1-1, B1-2) that can be traced from the dam centerline, to the left of the key trench, upstream along the right abutment for at least 400 feet. Sharp angular fragments of brecciated rhyolite occur discontinuously in voids along this zone, which is undulatory or wavy in detail (figs. B1-5, B1-10). Matching of details of the fit between the upper and lower surfaces suggests that lateral displacement of not more than a few inches could have occurred along the zone. Upstream, the zone splits into several gently curved branches which die out laterally. Downstream, the zone ends abruptly about 50 feet downstream from the grout cap, which is in the middle of the key trench.

Because the breccia fragments are all sharply angular and devoid of vapor-deposited coatings, the breccia must have formed when the rhyolite was relatively cold and brittle, sometime later than the episode of secondary flowage. Deposits of calcite on and between breccia fragments are not brecciated or fractured, indicating that no additional displacement has occurred along this zone since it was formed.

Causes of Enlargement or Widening of Steep Joints at the Damsite

There are several possible causes of additional widening or enlargement of steep joints since their original formation by contraction during cooling. They fall into two general classes: those related to horizontal extension, or pulling apart, and those related to erosion, or removal of rock material.

Processes of widening related to horizontal extension include (1) tectonic crustal extension; (2) gravitational creep, aided by weathering, especially frost action; (3)

Figure B1-10.—View of prominent contact between unit 2 and unit 3 just upstream from embankment remnant. Arrows indicate location of contact.

movement of rock toward the canyon because of lateral stress relief, or unloading; and (4) movement caused by differential settlement due to subsurface erosional sapping of fine-grained sediments beneath the rhyolite.

Evidence for (1) tectonic crustal extension includes the following: (a) the Snake River Plain and surrounding Basin-Range provinces of eastern Idaho have been undergoing regional tectonic extension from at least late Miocene time to the present; (b) tectonic extension in the Teton-Rexburg Bench area, in particular, has been most active since late Pliocene time, or since emplacement of the Huckleberry Ridge Tuff less than 2 million years ago; and (c) the predominant northwest trend of fissures at the damsite is consistent with the predominant northwest trend of Quaternary fissure zones, such as the Great Rift on the Snake River Plain, and with the dominant trend of late Cenozoic, possibly active, normal faults adjacent to the plain.

(2) Gravitational creep has been the demonstrable cause of block movement with attendant joint widening at a number of places along the canyon, but because most of the prominent wide fissures at the damsite trend at high angles to the canyon walls it does not seem likely that creep could have been the principal cause of joint widening. Intersecting joints conceivably might have blocked out large rock masses that migrated canyonward, perhaps influenced by the character of the underlying sediments, especially claystone; however, the relative sparsity of wide fissures oriented parallel to the canyon walls argues against this process being the most important one.

(3) Stress relief due to erosion of the present canyon may account for some widening of joints, particularly in the lower part of the canyon; it probably is not an especially important cause, however, and its effects would be difficult to distinguish from widening due to gravitational creep.

Little evidence supports (4) differential settlement caused by subsurface erosional sapping. However, because the rhyolite is underlain by variably indurated fine-grained sediments interlayered with gravels, some of which may be ground-water aquifers, this process cannot be entirely discounted.

Erosional processes include spalling or slabbing of fractured rock along joints and enlargement by hydrothermal leaching. Spalling has contributed to the further enlargement of already opened joints, but it cannot be the primary process of widening because there is no effective way of removing the coarse rubble from the fissure. Hydrothermal leaching probably is not important except very locally, because nearly all of the exposed fissures have surfaces that are smooth and planar, not irregular and corroded.

On the basis of the available evidence, most of the high-angle fissures at Teton damsite seem to be cooling joints that subsequently have been further opened mainly by horizontal tectonic extension and locally by gravitational creep, perhaps aided to some extent by lateral stress relief. Other fissures, formed earlier during differential compaction and secondary flowage of the ash-flow sheet, are not as numerous or as extensive as the later ones. The rate of extension and its recency have not adequately been established.

References Cited

Christiansen, R. L., and Blank, H. R., Jr., 1972, Volcanic stratigraphy of the Quaternary rhyolite plateau in Yellowstone National Park: U.S. Geol. Survey Prof. Paper 729-B, 18 p.

Prostka, H. J., and Hackman, R. J., 1974, Preliminary geologic map of the NW 1/4 Driggs 1^{o} by 2^{o} quadrangle, southeastern Idaho: U.S. Geol. Survey Open-File Report 74-105.

Ross, C. S., and Smith, R. L., 1961, Ash-flow tuffs; their origin, geologic relations, and identification: U.S. Geol. Survey Prof. Paper 366, 81 p.

Schmincke, Hans-Ulrich, and Swanson, D. A., 1967, Laminar viscous flowage structures in ash-flow tuffs from Gran Canaria, Canary Islands: Jour. Geology, v. 75, no. 6, p. 641-664.

Smith, R. L. 1960a, Ash flows: Geol. Soc. America Bull., v. 71, no. 6, p. 795-841.

——— 1960b, Zones and zonal variations in welded ash flows, *in* Shorter contributions to general geology, 1959: U.S. Geol. Survey Prof. Paper 354-F, p. 149-159.

APPENDIX B-2

ANALYSES OF GROUND-WATER LEVELS IN THE RIGHT ABUTMENT

FIGURES

ANALYSES OF GROUND-WATER LEVELS IN THE RIGHT ABUTMENT

The following analyses of ground-water levels in observation wells in and near the right abutment were made to determine, from the available data, the changes that occurred during reservoir filling and after failure and their possible significance.

Hydrologic Conditions in the Right Abutment During Filling of the Reservoir

The data concerning hydrologic conditions in the right abutment during filling of the reservoir were derived from observations of water levels in previously drilled exploration holes that were monitored for this purpose during reservoir filling. Of the drill holes that were monitored only those close to the grout curtain in the right abutment were selected for analysis by the Geology Task Group. The analyses, we believe, provide pertinent information concerning ground-water levels in the region downstation (northwest) of the spillway. However, they also bear indirectly on hydrologic conditions in the area of greater interest in the vicinity of the grout curtain upstation (toward the river) from the spillway.

The analyses are derived from observations reported in the memorandum of June 14, 1976, from Regional Director, Bureau of Reclamation, Boise, to Director of Design and Construction on the subject "Reservoir Seepage Loss Study—Teton Dam and Reservoir," with attachments, which pertains to water levels in the reservoir and in drill holes 5, 6, 501, 504, and 506(2) (2 refers to the upper piezometer in that hole), and from Records of Subsurface Investigations for Teton Dam and Power and Pumping Plant, Bureau of Reclamation, June, 1971, reprinted June 1976. The location of these drill holes is shown in figure B8 of Appendix B. The analyses consist of three bodies of data: I, the variation of water levels with time shown in figure B2-1A; II, variation of rates of water-level rise with time, shown in figure B2-1B; and III, variation of differences in water levels between selected locations with time, shown in figure B2-2. The interpretation of these data is in section IV.

I. Variation of water levels with time—Figure B2-1A.

1. Filling of the reservoir continued at a fairly steady rate through the months of January, February, March, and early April 1976.
2. In early April the filling rate increased markedly.
3. The behaviors of water levels in drill holes 501 and 504, both upstream from the grout curtain, are similar. Drill hole 504, being nearer to the reservoir, generally showed water-level elevations about 3 days earlier than drill hole 501 during the later stages of filling from April 10 until the middle of May; later the delay was greater, because these two inclined drill holes are separated farther at higher elevations.
4. The behaviors of water levels in drill hole 5, downstream from the right end of the grout curtain, and drill hole 506(2), beyond the end of the grout curtain, are similar. The extreme northwestern limit of grouting is probably at about station 0+00 at elevation 5100, but the effective grout curtain probably did not extend downstation beyond station 1+00 (see fig. B10, Appendix B). The behavior of water levels in drill holes 5 and 506(2) appears somewhat different from that of the 501-504 pair. Water levels in both drill holes 5 and 506(2) were always at considerably lower elevations than in drill holes 501 and 504, as was to be expected.

II. Variation of rates of water-level rise with time—Figure B2-1B.

1. The filling rate curve for the reservoir (2-day averages) shows a fairly constant rate until about April 3, when a marked increase in rate occurred, peaking at about April 13 at about 3 ft/day (peak 1) and falling back, with an intervening minor rise, to about 1 ft/day on April 28.
2. After April 28 the rate of reservoir filling increased steadily to a peak of about 4 ft/day on May 19 (peak 2), then dropped off to a rate of about 2.3 ft/day at the time approaching dam failure.
3. Comparison of rate curves:

   a. *Comparison of drill hole 501 (just upstream from the curtain, a slant hole 60° from horizontal directed S. 30° E.) with the reservoir.*—The rate of rise in drill hole 501 generally followed that of the reservoir but exceeded it slightly during much of March, in mid-April, and in mid-May.

It appears to have lagged about 1 day behind the April 13 peak of reservoir filling rate and 3-1/2 days behind the May 19 peak of reservoir filling rate. The time spacing of readings in the drill holes was not daily, as at the reservoir, which makes all such correlations subject to some error. Based on a water-level reading on June 1, the rate of rise of water in drill hole 501 in late May became less than the rate of rise in the reservoir.

b. *Comparison of drill hole 506(2) with the reservoir.*—Drill hole 506 is vertical, about 300 feet beyond the end of and on line with the grout curtain. The upper piezometer, 506(2), was erratic until early April, its rate of rise being sometimes above and sometimes below that of the rate of the reservoir. Between April 3 and 4, essentially at the same time as the beginning of the rise of reservoir rate to peak 1, drill hole 506(2) also began a distinct increase in rate. The peak rate of the reservoir on about April 13 and minimum rate on about April 28 were not reflected in the record of drill hole 506(2) because of lack of readings. The rate increased after May 5 and exceeded that of the reservoir rise rate on about May 12, finally peaking at a rate of about 10.55 ft/day on May 19, at the same time that the reservoir rate peaked. This rate is 2.7 times the peak maximum rate of rise of the reservoir. As the rate of reservoir filling declined late in May, so also did the rate of rise of water in drill hole 506(2), although it remained at a faster rate than that of the reservoir.

c. *Comparison of drill hole 5 with the reservoir.*—Drill hole 5 is located just barely behind and downstream from the grout curtain (see fig. B10, Appendix B). It is angled 60° from horizontal and directed N. 25° E. Its record is perhaps the most important of all, as it is the only drill hole in the right abutment behind the grout curtain (a short distance) that was monitored during filling of the reservior. Figure B2-1B shows that its rate of rise approximately followed that of the reservoir until early April. Between April 3 and 4, essentially at the same time as the reservoir filling rate accelerated toward peak 1, drill hole 5 showed an increase in rate of water-level rise. The rate of rise in drill hole 5 reflected either (a) the April 13 peak (peak 1) of the reservoir rate on about April 18, a delay of 5 days, (b) increased permeability at about elevation 5180, which was the reservoir level on about April 10, or (c) a change in other hydraulic properties of the welded tuff. The minimum rate of rise in drill hole 5 between April 27 and 28 apparently reflected the April 28 minimum rate of rise of the reservoir, but the spacings of readings on drill hole 5 and averaging rates over long times probably accounts for this minor "anticipation." After that, the rate of rise of water in drill hole 5 accelerated, surpassing the rate of rise of the reservoir around May 17 or 18 and peaking between May 22 and 23 at a rate of 11.64 ft/day, or 2.95 times the peak rate of rise of the reservoir that was attained on May 19. The rate of rise of water in drill hole 5 then declined abruptly, although remaining higher than the rate of rise of the reservoir. The rate of rise of the reservoir also declined, although the decline in the rate of rise of the reservoir was much less abrupt.

III. Variation of differences in water levels between the reservoir and drill holes and among drill holes with time.

Differences in water levels between the reservoir and selected drill holes, and between selected drill holes, were examined, as they varied with time, because pressure differential is one of the principal factors determining the velocity of water moving from place to place and for removing debris that might be clogging fissures.

In figure B2-2, curve A shows the variation of the pressure differential between the reservoir and drill hole 506(2). A rise in early April apparently reflects the increase in reservoir filling rate, and the slight decline in mid-April apparently reflects the decline in filling rate during that period. The rise in early May appears also to be a response to the increase in filling rate of the reservoir. The pressure differential on May 13, distributed in some unknown manner over the approximately 1,300 feet to the reservoir edge, was about 133 feet of water. On or about May 13 a very rapid decline in pressure differential began that continued until the last observation preceding the dam failure.

Curve B, showing the variation of the difference in water level between the reservoir and drill hole 5, shows a sharper increase in level differences than curve A in late April and early May, reflecting perhaps the effect of the grout curtain in retarding the rise of water in drill hole 5 compared with drill hole 506(2). The sharp drop occurs on about May 18, about 5 days later than at drill hole 506(2).

Curves C and D show, respectively, the variations in pressure differential between drill hole 501 and drill hole 506(2) and between drill hole 501 and drill hole 5, the latter drill hole being on the other side (downstream) of the grout curtain from drill hole 501. These curves are similar in that both show a steady state in the latter part of March, in equilibrium with the then steady rate of reservoir filling, and both record a rise reflecting the increased filling rate in early April. However, curve C, the pressure differential between drill hole 501 and drill hole 506(2), shows a rapid decline beginning on May 13, or between May 13 and 14, whereas the similar rapid decline in pressure differential between drill hole 501 and drill hole 5 was delayed. The delay period was 6 days, if only the data points are used to compare the peaks, and about 5 1/2 days if water levels are interpolated between times of observation (dotted line).

Curves E and F show, respectively, the variations in pressure differential between the reservoir and drill holes 501 and 504. Their traces are similar to each other and very different from curves A and B. Curves E and F show a general decline indicating a tendency for water levels in drill holes 501 and 504 to come to equilibrium with the reservoir. This trend is disturbed briefly in early April, apparently by the accelerated filling rate of the reservoir. Curve E shows a minor loss of pressure differential beginning about May 13, which was recovered by June 1, back to the declining trend of late April and early May.

Water-level observations in drill holes 5, 6, and 506(2) permit drawing contours on an assumed plane surface representing the water table passing through the three drill holes (fig. B2-3). This surface was gently inclined toward N. 70° W. during early stages of filling, dipped steeply directly westward about May 13, swung to about S. 70° W. on June 1, and to S. 65° W. on June 5 (using projected values for water levels). These changes appear to be those that might be expected as the water table, formerly adjusted to the northeast-trending Teton River, was affected by the rise of water in the reservoir, and by the grout curtain. During filling of the reservoir the direction of slope of the water table between drill hole 5 and the river is not known from observation, but it was probably toward the south, or even southeast as indicated in figure B7 of Appendix B.

IV. Interpretation of data

The curves presented in figures B2-1, B2-2, and B2-3 are derived from the available data and arithmetic operations on these data. It is probable that more information could be extracted from a computer-supported analysis of transient conditions with consideration of the moving reservoir boundary and the variation in space and time of the hydraulic characteristics of the rock, such as saturated thickness, transmissivity, and storage coefficient. However, the lack of data on water levels at critical times and places would make the value of more elaborate analysis questionable. The interpretation of the simple curves in the figures is attended with considerable uncertainty; the curves do, however, bring out some relations that bear on ground-water conditions preceding failure:

1. There were two periods of departure from steady rates of rise of water levels in observation wells in the vicinity of the right end of the grout curtain; one in

the period April 5-10 and a much more important one in the period May 13-19.

2. These periods of change may reflect (a) change in filling rate of the reservoir, (b) water levels reaching zones of different permeability, or (c) variation with water-level elevation of other hydraulic characteristics of the welded tuff.
3. The difference in water levels between the reservoir and the drill holes, and among some of the drill holes, varied abruptly in mid-May. As the hydraulic head is one of the principal factors determining the velocity of water moving from place to place and for removing the fillings of joints and open fissures, it should be considered in analysis.
4. The data in figures B2-1, B2-2, and B2-3 indicate an expectable flow to the end of and around the grout curtain and the possibility of a very strong pressure pulse that reached drill hole 5 on or before May 19. It would take a longer time for this pulse to progress from drill hole 5 to the canyon wall of the Teton River and the dam embankment itself than it took to go from drill hole 506(2) to drill hole 5. The distances of travel are uncertain, owing to the unknown path from drill hole 506 to drill hole 5, the 60° inclination and northeast orientation of drill hole 5, the unknown pattern of joints between drill hole 5 and the dam, and unknown direction and magnitude of gradients of the water table between drill hole 5 and the river. Using crude estimates of distance, a delay time between drill hole 506(2) and drill hole 5 of 6 days, and a uniform rate of travel, the water-level rise recorded at drill hole 5 would have reached the vicinity of the canyon wall-dam embankment interface downstream from the grout curtain about 16 days later, on June 4.

   On June 3 two small springs developed in the right abutment 600 and 900 feet downstream of the spillway stilling basin, flowing about 40 and 60 gallons per minute, respectively. On June 4 a spring flowing about 20 gallons per minute developed on the right abutment about 150 feet downstream from the toe of the dam. Early in the morning of June 5, the day of failure, the first major seepage, 20 to 30 cubic feet per second, developed in the right abutment at about elevation 5045 near to and above the toe of the embankment. These springs of June 3-5 may record the arrival of the rise in water levels discussed above, and indicate progressive wall-rock saturation that may have augmented other possible processes that led to failure higher in the embankment.
5. The drop in pressure differential between drill hole 501 upstream from the grout curtain and drill hole 5, downstream, beginning on about May 19 could have been caused by the establishment of a direct hydraulic connection through the grout curtain, rather than around it. Perhaps both processes operated. Flow over the curtain and grout cap on about May 19 near the right end of the key trench seems unlikely (a) because the key trench was filled with impermeable zone 1 material, and (b) because the key trench bottom in that vicinity is about at altitude 5250 and the water level in drill hole 501 did not reach 5250 until around May 25. However, post-failure permeability tests in drill hole 650 indicate that flow under the trench at stations 3+50 to 3+70 could have begun before water levels reached 5250 (see fig. B10 in Appendix B).
6. The observed changes in (a) water levels in the drill holes, (b) their rates, and (c) the differences in levels among them, and between them and the reservoir, indicate hydrologic events in the period May 13-19 that are worthy of consideration. The significance of these events is not clear because the available data are inadequate. What is clear is that more observation wells to measure water levels during reservoir filling were needed in the right abutment closer to the reservoir on both sides of the grout curtain near the dam embankment. Although failure occurred in the right abutment, the same comment applies to the left abutment.

Hydrologic Conditions in the Right Abutment After Failure

The failure of the dam lowered the reservoir level to about elevation 5055. Water stored in the banks of the reservoir returned to the canyon walls, producing large springs and seeps in the right abutment that were observed from the air and ground. Their positions were plotted and their flows were measured. Post-failure water-level readings were recorded at the observation drill holes shown in figure B2-1, as well as at other wells. Water levels in drill holes close to the dam dropped rapidly after the reservoir emptied. The rocks intersected by drill hole 5 have very high permeability above elevation 5100 (see fig. B10 in Appendix B). When the reservoir emptied, the level in drill hole 5 dropped as fast as did the levels in wells on the upstream side of the grout curtain (drill holes 501, 504) (see fig. B2-1). It is of interest to note in figure B2-2 that the rapid declines in curves C and D that begin in mid-May continue on the same trend through the time of dam failure to the next reading, on June 9. They then begin to level out. As shown in figure B2-3, the direction of inclination of the water table, and its gradient, within the triangle defined by water levels in drill holes 5, 6, and 506, rapidly returned to conditions approximating those before reservoir filling, although the level on June 13, 1976, was about 120 feet higher than on September 24, 1975 (fig. B2-3).

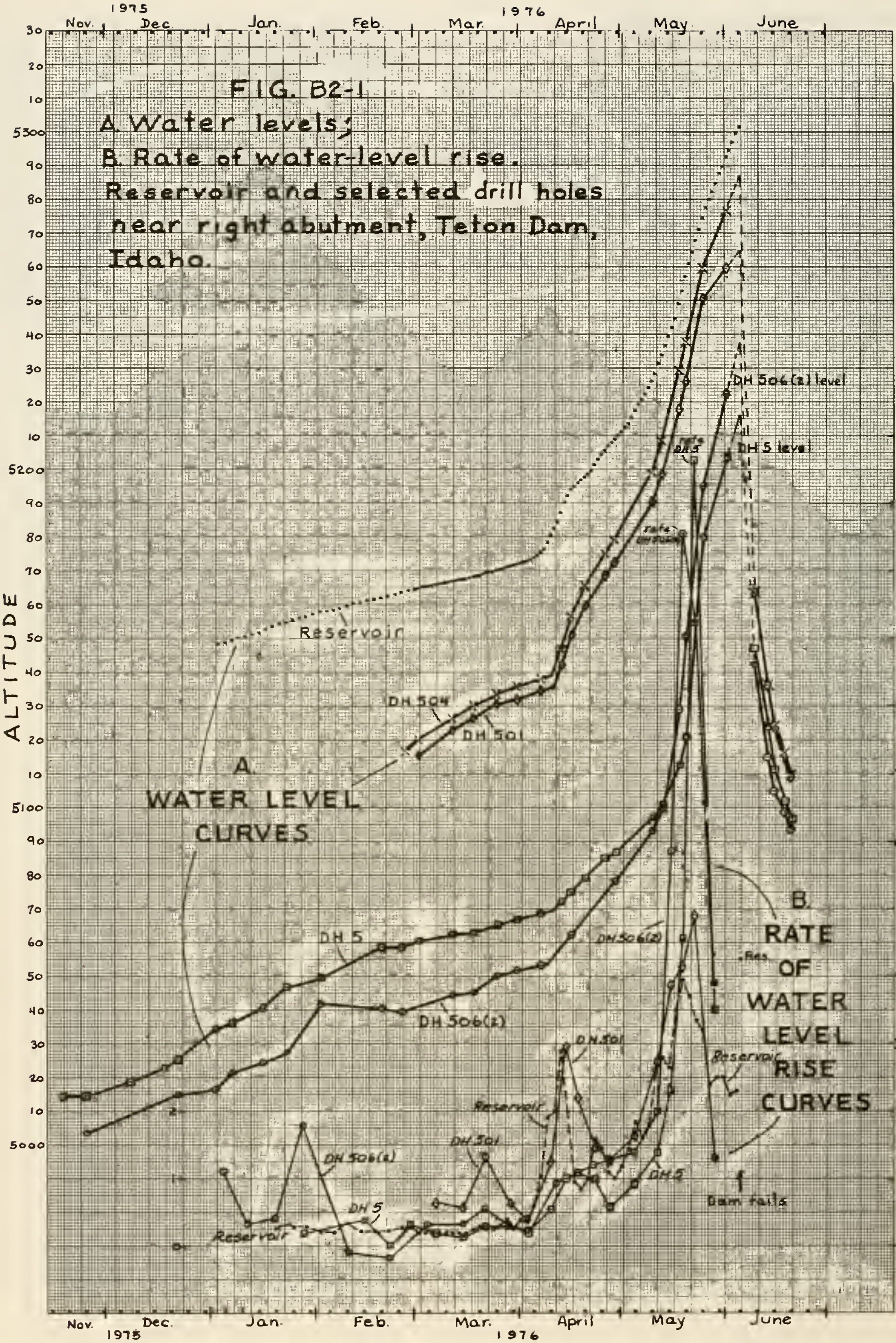


Figure B2-1.—A, Water levels; B, Rate of water-level rise. Reservoir and selected drill holes near right abutment, Teton Dam, Idaho.

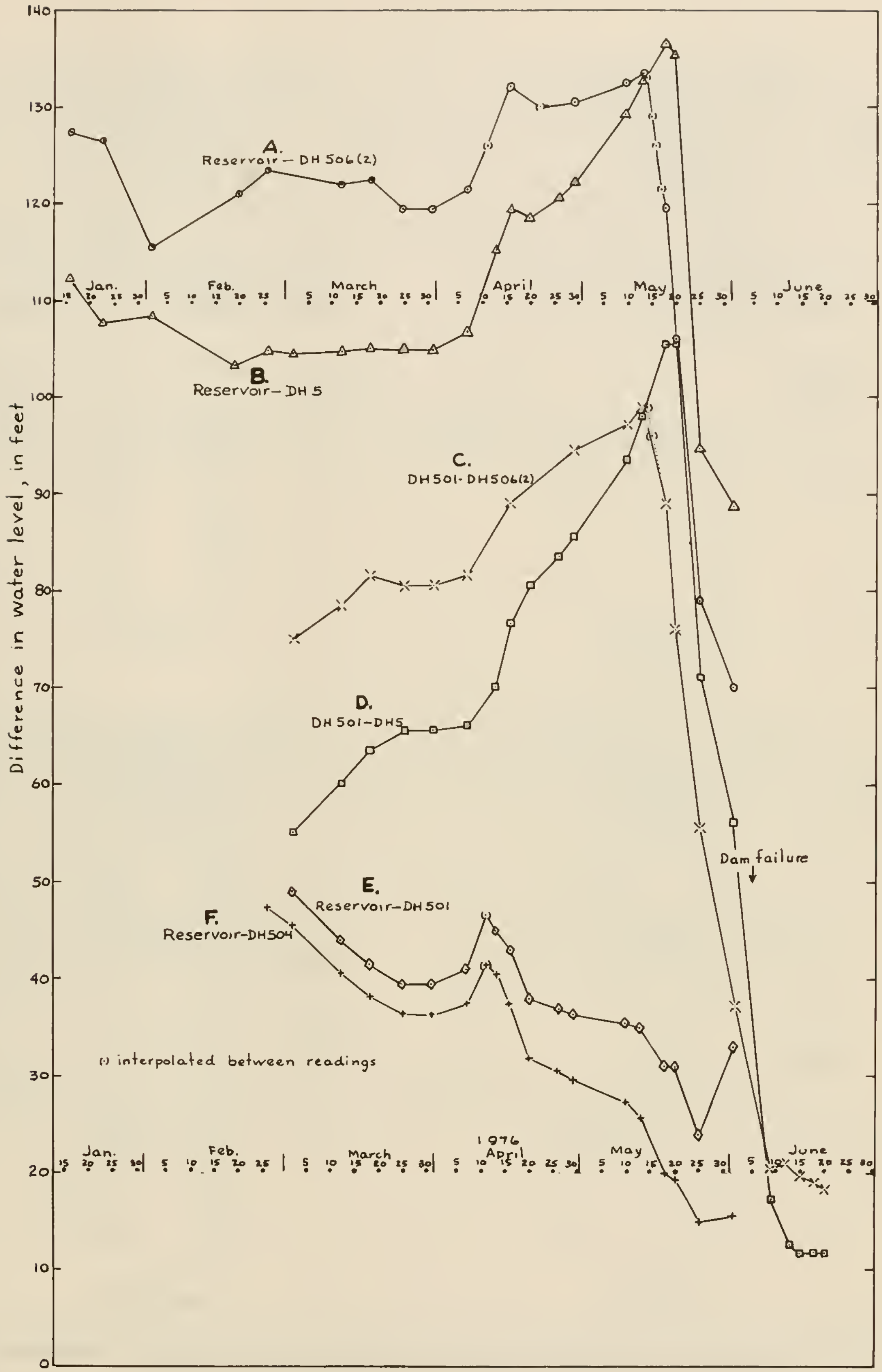


Figure B2-2.—Variation of differences in water levels with time.

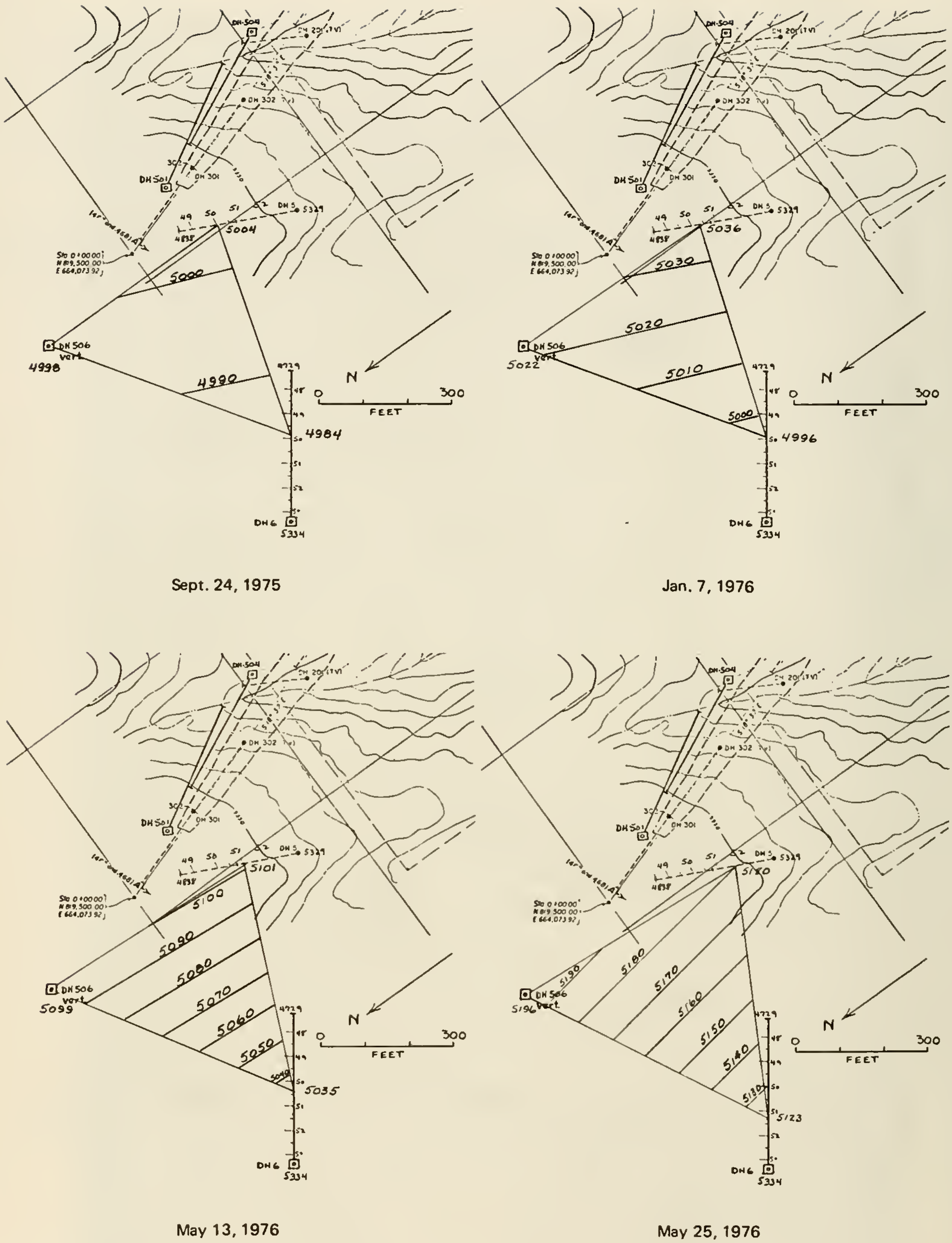
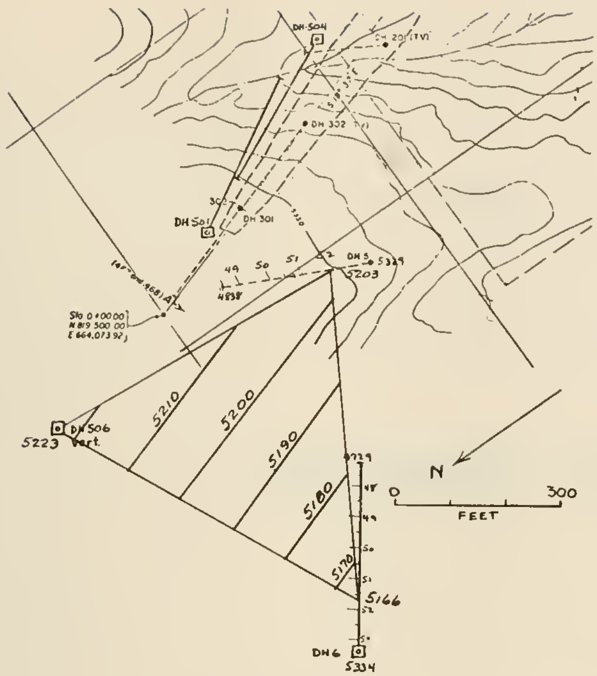
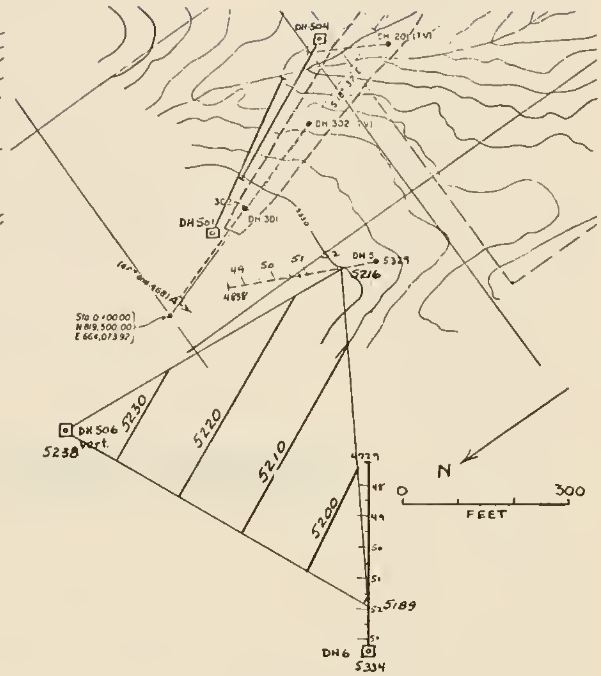


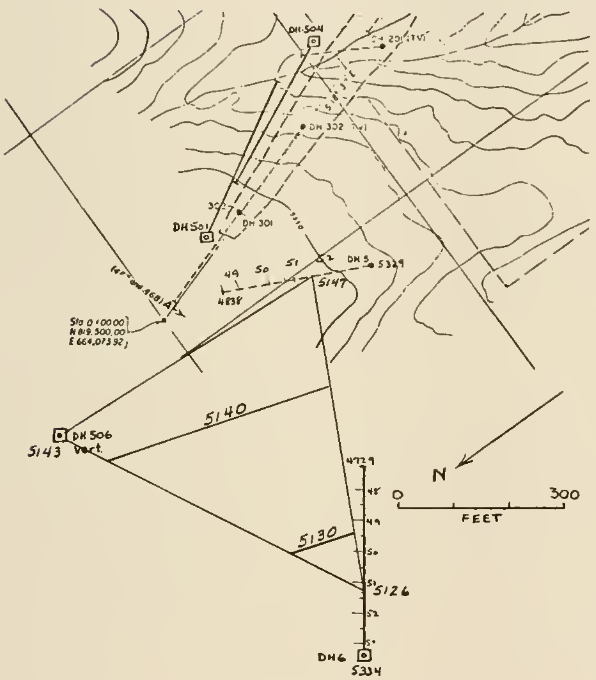
Figure B2-3.—Contours on an assumed plane water table defined by water levels in drill holes 5, 6, and 506; shown at various times before and during reservoir filling and after failure. (Sheet 1 of 2)



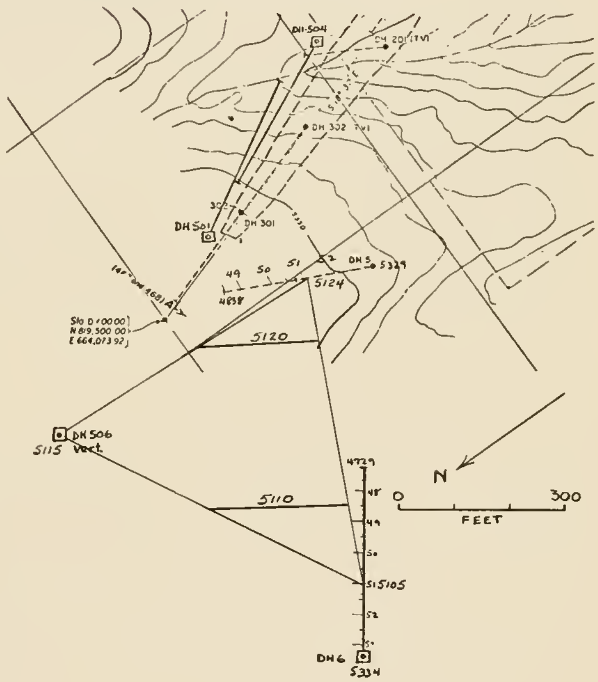
June 1, 1976



June 5, 1976
(before failure, from
projected data)



June 9, 1976



June 13, 1976

Figure B2-3.—Contours on an assumed plane water table defined by water levels in drill holes 5, 6, and 506; shown at various times before and during reservoir filling and after failure. (Sheet 2 of 2)

REPORT OF FINDINGS

Grouting Task Group

U.S. Department of the Interior
Teton Dam Failure Review Group

December 10, 1976

CONTENTS

ABSTRACT

This report is the result of a study to evaluate the adequacy of the Teton Dam grout curtain and surface grouting to perform their design functions and to determine, so far as possible, the contribution, or lack thereof, of the rock treatment to the June 5, 1976, dam failure. The study was conducted through interviews of involved Bureau of Reclamation and contractor personnel, examination of construction records, written design questions to the Bureau of Reclamation, site observations, field explorations, and laboratory tests (not yet completed).

The curtain grouting was performed in general accordance with the contract specifications. Though the grout curtain has been termed a multiline curtain on the abutments, it is, in reality, a single line curtain with grouting in rows up and downstream from the centerline row to inhibit grout travel from the centerline row. Only the centerline row was split-spaced to closure. The potential performance of the grout curtain should have been normal in terms of limiting but not completely blocking seepage. During the postfailure drilling and water pressure testing, a permeable zone was discovered at shallow depths in the grout curtain in the vicinity of station 13+50.

An attempt was made to seal open joints in the rock surface outside the key trench between elevations 5075 and 5205. Although the need for a rock surface treatment program was considered in the early stages of design, such a program was apparently not included in the final design. This treatment was not included in the construction specifications. It was discontinuous where treatment was used; it was not tied into the grout curtain; and it was not applied to a significant abutment area above elevation 5205.

There is a significant coincidence of features in the area between stations 13+00 and 14+00. These features occur along a line between the locations of leaks observed downstream prior to and during failure and the upstream whirlpool.

The only discovered grout curtain permeabilities are not considered high enough to have been significant of themselves to the failure development. Available evidence leads the Grouting Task Group to conclude that design reliance on curtain grouting to eliminate erosive seepage and resultant poor rock surface treatment contributed to a failure between stations 13+00 and 14+00.

TRANSMITTAL DOCUMENT

In compliance with the assignment made by the Chairman, Department of the Interior Teton Dam Failure Review Group, the Grouting Task Group submits this Report of Findings on the aspects of rock treatment involved in the Teton Dam failure.

Paul R. Fisher

Paul R. Fisher, Chairman,
Staff Geologist,
Engineering Division,
Directorate of Civil Works,
Office, Chief of Engineers
Corps of Engineers

Ray Cope

Ray Cope,
Civil Engineer, Grouting Specialist
Midwest Technical Service Center
USDA Soil Conservation Service

James H. Coulson

James H. Coulson,
Principal Civil Engineer,
Geotechnical Engineering
Division of Engineering Design,
Tennessee Valley Authority

INTRODUCTION

Purpose

The purposes of the study reported herein are threefold:

1. To evaluate the adequacy of the grout curtain to perform its design function

2. To evaluate the adequacy of complementary surface treatment operations

3. To determine the probable contribution, or lack thereof, of the rock treatment to the failure

Background

The Secretary of the Interior and the Governor of the State of Idaho appointed an independent panel from private industry and acadamia to determine the cause of the June 5, 1976, Teton Dam failure. In addition, the Secretary established the Department of the Interior Teton Dam Failure Review Group, calling on other Federal agencies for membership. The mission of the Interior Review Group is also to determine the cause of failure and, more importantly, to provide recommendations for methods to prevent recurrence of such failures. The Interior Review Group established three task groups to assist in conducting investigations: (1) an Embankment Construction Task Group, (2) this Grouting Task Group, and (3) a Geology Task Group.

There is close interplay between embankment design, rock treatment design, and the existing geologic conditions at any specific site. It is virtually impossible to divorce consideration of one aspect of rock treatment (i.e., surficial treatment/surface grouting) from another (i.e., subsurface treatment/curtain grouting). It is equally impossible to evaluate any type of rock treatment without understanding both the requirements for protection of the superimposed structure and the geologic conditions which dictate the method of treatment to provide the required protection. As a consequence, close liaison is being maintained between the three task groups. The Grouting and Construction Review Task Groups overlapped their investigations in the area of rock surface treatment.

Office and field investigations were performed by the USBR. Information from these investigations was provided to both investigative bodies. The investigation requests were coordinated to assure that no duplications occurred.

Scope

The scope and chronology of the Grouting Task Group investigations are summarized on Table 1.

The report is organized into five sections:

The first states the purpose, background, and scope of the Grouting Task Group investigations and the organization of the report.

The second section summarizes the Grouting Task Group investigative procedures.

Table 1

SCOPE AND CHRONOLOGY OF GROUTING TASK GROUP INVESTIGATIONS

Formation of Task Group	July 12-16, 1976
Task Group visit to Teton Dam; interviewed grouting contractor and Bureau personnel, examined rock treatment records, inspected right abutment conditions, met with Geology Task Group Chairman	August 17-19
Chairman, Task Group met with Geology Task Group and attended Interior Review Group meeting at Teton Dam, Idaho Falls, and Denver	September 13-17
Task Group visited the Bureau of Reclamation, Denver, to interview rock treatment designer and Teton Dam field engineer	October 15-16
Task Group visited Teton Dam; inspected rock surface conditions on the right abutment, met with Geology Task Group, and met with the Interior Review Group field representative	November 3-5
Chairman, Task Group, met with Waterways Experiment Station Concrete Laboratory personnel concerning Teton Dam grouting	November 18
Task Group met with Bureau Construction Representatives to discuss revised report	February 1, 1977

The third section provides a general discussion of rock treatment considerations and of the Teton Dam rock treatment and how it relates to the physical site characteristics.

The fourth section presents our conclusions concerning the adequacy of the rock treatment at Teton Dam and any probable contribution of the rock treatment to the failure.

The fifth section outlines the remaining investigations required to validate our conclusions.

INVESTIGATIVE PROCEDURES

Interviews

Interviews with USBR and contractor personnel were conducted during the August 17-19 Task Group visit to Teton Dam and the October 15-16 visit to the Bureau in Denver. Individuals interviewed were Richard Bock, Head, Earth Dams Section; William Harber, retired principal Teton Dam designer; Lloyd Gebhart, Construction Liaison; Robert Robison, Project Construction Engineer; Peter Aberle, Project Field Engineer; Jan Ringel, Douglas Jarvie, and Kenneth Hoyt, embankment inspectors; Claude Daniels, grouting inspector; Brent Carter, Chief, Geology Branch, Pacific Northwest Region; William and Ed McCabe, McCabe Brothers Drilling Co.; and V. M. Poxleintner, Morrison Knudsen Co.

Summaries of the Grouting Task Group interviews are contained in Appendix C-1.

The Construction Review Task Group has also conducted an extensive investigation into the "surface grouting." Results of its interviews on that subject are contained in that Task Group report.

Construction Records

Those Bureau construction records which were examined consisted of the field inspectors' drilling and grouting reports, the L-10 monthly grouting summaries, the prefailure and postfailure geologic maps, and construction photographs of the rock conditions on both abutments.

The Grouting Task Group examination of the detailed records was limited to the right abutment between stations 11+00 and 17+00.

Records which the Grouting Task Group did not examine were the Bureau grouting inspectors' log books and records kept by McCabe Brothers Drilling, Inc.

Interior Review Group Design Questions and Bureau Answers

The Interior Review Group presented a series of questions to the Bureau dealing with the design considerations for Teton Dam. In addition, the Independent Panel presented questions to the Bureau. The questions and answers dealing with rock treatment comprise a considerable portion of the basis for the Grouting Task Group's analysis, discussion, and conclusions. Those questions and answers are presented in appendix G of the main report.

Site Observations

During the August 17-19 visit to Teton Dam, the Grouting Task Group examined the surface of the right abutment by viewing it from the visitor overlook, the top of the embankment, the river level at the riverside edge of the breached embankment, and by walking over the upstream surface of the abutment.

During the November 3-5 visit, we examined in detail the cleaned rock surface in the key trench and downstream portion of the right abutment, and reexamined the upstream rock surface.

Field Explorations

Both the Independent Panel and the Grouting Task Group requested subsurface explorations on the right abutment to check the effectiveness of the grouting program. The requested explorations are detailed in Appendix C-2.

The Independent Panel exploration requests were given first priority. The status of exploration drilling, maps showing location of postfailure borings, and boring logs are presented in Appendix C-5.

Laboratory Testing Program

Because of the unprecedented high proportions of calcium chloride used in the curtain grouting, the Grouting Task Group requested that the Bureau perform a testing program to determine grout behavior, durability, and resistance to erosion for the various mixes containing calcium chloride. The Bureau conceived and proposed a program which was approved by the Grouting Task Group. Results of this testing program should be available during March 1977. The grout testing program is outlined in Appendix C-3.

DISCUSSION

Introduction

Certain of the Bureau grouting procedures are different from those practiced by the agencies represented on the Group. In order to set the stage for our evaluation, the following discussion on general rock treatment considerations summarizes the experiences of the members of the Grouting Task Group.

The safety of an engineering structure and the reduction of permeability in the foundation are usually the objectives of a grouting effort.

Though a single or multiline grout curtain is not 100 percent effective, it is quite often the only economical or feasible means of reducing the flow through rock foundations and abutments of dams.

Grouting measures should extend from the rock surface to the maximum design depth. The difficulty inherent in the application of pressure grout near the rock surface requires that methods of grouting vary with depth and objectives.

To prevent water from flowing through the foundation near the rock-earth fill interface, and thus endangering the fill, slush-or-slurry grout is applied to the thoroughly cleaned rock surface to backfill the cleaned-out open voids with cement grout. Blanket grouting reduces high permeability immediately below the rock surface. An acceptable blanket grouting design consists of a grid of shallow holes for the injection of grout under low pressures. Commonly the area treated extends laterally rather than in depth and the holes are closely spaced to facilitate complete grout filling of voids at low pressures. Curtain grouting extends the treatment to design depth. It consists of rows of grout holes which are grouted under pressures which increase with depth.

In curtain grouting, primary holes are drilled and grouted at a preselected spacing in the row. If one of these primary holes takes grout, two new holes are drilled to design depth or past the elevation of take. These two secondary holes are located at each side of the primary hole and thus split the spacing in half between primary holes. This procedure is called split spacing. It is carried out until the distance between grout holes becomes too close, 2-1/2 to 3 feet, to be effective. Additional holes are located outside the row rather than within.

Special features, such as fault zones, wide joints, and known large cavities require special design considerations with respect to hole patterns, grouting materials, and pressures. It is the consensus of the Grouting Task Group that a three-row curtain, in which each row is grouted to closure, is commonly considered when high permeabilities are a major problem. This constitutes a realistic compromise between economy and the obtainable efficiency.

Standard rock treatment specifications usually include detailed instructions for the preparation of rock surfaces prior to treatment, and for the treatment of known special features as well as provisions for grouting in freezing temperatures. Specifications based on actual experience from pilot grouting programs are usually more specific than standard specifications. As at Teton Dam, pilot programs generally result in better estimates of quantities, grout pressures, mix ratios, and grouting sequences.

Possible Rock Treatment Related Failure Modes

The six possible causes of failure listed by the Interior Review Group in its second Interim Report are:

1. Cracking or hydraulic fracturing of zone 1 material

2. Piping along the interface between the zone 1 material and the rock foundation

3. Flow through the grout curtain

4. Flow bypassing the grout curtain

5. Cracking due to foundation settlement

6. Cracking due to hydraulic uplift

Failure modes involving the rock foundation would (1) include water movement into the zone 1 material from the rock and vice versa (e.g., water entering zone 1 upstream from the grout curtain and leaving it downstream from the grout curtain), or (2) involve the movement of high-velocity water through the rock immediately adjacent to the zone 1 material. The former could result in removal of zone 1 material by piping; the latter by scour erosion.

The first possible cause listed above could have involved the rock foundation if the hydraulic fracture or crack occurred in a location such that the reservoir water could enter and leave the embankment through rock fractures. The second through the fourth possible causes are obviously rock foundation related. The fifth and sixth possible causes were not given much credence by the Interior Review Group.

Piping failures of embankments and embankment foundations usually begin in the vicinity of the downstream toe and progress headward (upstream). Subsurface scour erosion requires continuous channels downstream from the location of scour to allow continued movement of material from the point of erosion.

The observed mode of failure indicates that the piping at the downstream toe was not the primary cause of failure. Piping may have occurred from the embankment into the abutment rock mass because there are sufficient voids in the form of open fractures in the upper portions of the abutment to accept embankment zone 1 material either being piped or scoured from the key trench.

The preconstruction explorations do not provide sufficient data to make conclusive quantitative analysis of the effective (or fracture) porosity of the rock mass. The bore hole TV logs of right abutment borings indicate a probable effective porosity of about 1 percent in the upper 100 feet of rock. Analysis of core loss in the same borings indicates from 1 to 3 percent between 30 and 100 feet and around 10 percent above. We can assume that these percentages, which are derived from boring and bore hole TV log information, without regard to hole and joint orientation, are substantially lower than the true rock mass porosity. The low water table, rapid infiltration of rain water, consistent lack of drill water return, high and rapid grout takes during pilot and curtain grouting, and observation of the surface of the rock mass all indicate that the Teton foundation is of moderate to high porosity. Also, both the boring logs and TV logs indicated the presence of individual fracture openings up to 2 feet wide.

Investigation Findings

Design of Curtain Grouting

A detailed description of the pressure grouting design is attached as Appendix C-5. The following is a short abstract.

The principal design feature consisted of a one-row curtain to be grouted to closure (refusal). Primary holes were spaced at 80 feet and ranged from 260 feet to 310 feet in depth. Secondary holes on 40-foot centers were 160 feet deep; tertiary holes on 20-foot centers were 110 feet deep; and quaternary holes on 10-foot centers were 60 feet deep. There were provisions to drill closure holes to 5-foot centers, or closer if necessary.

In the abutments only, two outer rows were designed to control lateral grout travel from the center row. Primary holes were at 80 feet, secondary holes at 40 feet, and tertiary holes at 20 feet. Depths ranged from 60 feet to 260 feet. Closer spacing between holes was not specified for closure.

The grouting specification permitted both packer and stage grouting procedures.

With the exception of the spillway gate structure area, the need and choice of locations for blanket grouting were left to the Project Construction Engineer. Blanket holes were "to be ordered on an individual or small area basis to treat specific defects."

Grout mixes were specified to vary from 10:1 to 0.8:1 (water to cement ratio by volume), allowing for sand bulkfiller, for bentonite as a pumping aid for sanded grouts, and calcium chloride as required in the field, for the control of grout travel.

All grouting stages or packer settings were to be water-pressure tested prior to the injection of grout. The decision to grout and the starting mix ratio (either 8:1 or 6:1) were to be determined by these water pressure tests.

Pressures at the collar of the hole were not to exceed 0.75 lb/in^2 per foot of depth measured perpendicular to the closest surface, but not less than 10 lb/in^2.

Bench marks were to be established prior to grouting for the detection of "movements of the formation" or uplift due to grouting, specifically on steep abutments.

Construction of Grout Curtain

The construction field staff made every effort to comply with the contract specifications. They appear to have been interested in the quality of work performed and to have been determined to achieve, or exceed, the desired results.

There were some variations in the methods of grout injection. For example, under comparable conditions some inspectors would change from a neat cement and water grout mix to a sand, cement, and water mix; others would not. The rate at which grout mixes were thickened on rapidly taking grout holes varied somewhat. These variations occurred as the result of authorized exercise of judgment by individual inspectors and are not inconsistent with practice by other agencies on other projects.

A few exceptions to the specified split-spacing criteria were noted on the grouting records. These involved the lack of drilling of centerline fifth order grout holes where takes in scheduled quaternary holes were in excess of a specified amount. Three of these exceptions occurred in the right abutment grout curtain. A valid reason was provided for each exception.

Functionally, the abutment grouting at Teton Dam should not be considered as a three-row curtain. The outer rows were grouted to 20-foot centers and to depths of from 60 feet to 260 feet. The intent was not to install three rows grouted to closure but to facilitate more effective center row grouting. It is our understanding from our interview with Mr. Gebhart that the Bureau has normally closed all rows in a multiline grout curtain.

As part of the Bureau packer grouting procedures, water-pressure tests were routinely performed in holes, the lower section of which had just been grouted. Although this has been employed as a standard procedure by the Bureau, the Grouting Task Group believes that this procedure can result in washing freshly placed grout from rock fractures.

The starting grout mixes (usually 8 water to 1 cement) were much thinner than those used by the other agencies (5:1 or 3:1) and were used longer before changing to thicker mixes. Again, this constitutes normal Bureau practice.

Calcium chloride was used to accelerate the setting of grout in the outer two rows, with the intent of limiting grout travel. Three percent of the weight of cement is considered to be the maximum amount of $CaCl_2$ without having an adverse effect on the grout properties. Amounts above 4 percent are considered to adversely affect the permeability and permanence of the grout. The Bureau used up to 8 percent $CaCl_2$ in rapidly taking grout holes. In cold weather, $CaCl_2$ was added until the grout mix reached 90°F due to chemical reaction. The Grouting Task Group feels that the practice of heating the mixing water or the cement is a more common practice to obtain warm grout. The lower temperature of the rock at depth cooled the grout mix upon injection and reduced the rate of hydration. This, in turn, probably slowed the setting time in the rock sufficiently to allow the grout to flow freely away from the grout hole. Injection rates as high as 7 ft^3/min were used by the Bureau on taking holes in order to maintain design injection pressures. A more conventional means to restrict grout travel away from a grout hole (where subsurface flowing water is not present) is to use thick grout mixes, reduced pumping rates, and low pressures. The high percentages of $CaCl_2$ were not used during the grouting operations on the right abutment in area of the failure between stations 11+50 and 16+00.

Surface Treatment

No formal written procedures were developed within either the design or field construction organizations which detailed the surface treatment program.

Concerns for the protection of fill material susceptible to piping placed on a highly fractured foundation were expressed by Mr. Harber in a collection of design notes and draft material dated March 1967, November 20, 1969, November 12, 1969, November 13, 1969, May 1970, March 10, 1969, March 1967, October 20, 1970, and January 16, 1970. These concerns were not, however, expressed in the document "Design Considerations for Teton Dam, October 1971." This document, which was prepared by Design to familiarize Construction with important design considerations and to further describe the construction specifications, does not address itself to the details of a surface treatment program. In fact, the statement "Erosive seepage under the embankment will be eliminated by injecting the foundation with a grout mix" seems to infer that the "tight grout curtain" referred to in the next sentence was the major defense relied upon by Design to assure that piping was under control.

No provision was made in the contract specifications for general surface treatment of open joints except for blanket grouting to be used on "an individual or small area basis." It was the feeling of the designers that surface grouting should be treated as a field problem to be negotiated with the contractor. The Grouting Task Group feels that in order for Construction to make adequate field decisions, the theoretical reasons for surface treatment must be fully understood. None of the available design documents provided sufficient background information to make valid field decisions.

In late 1973 and early 1974, design, geology, and liaison personnel visited the site at Construction's request to look at the cut-off and key trench excavations prior to placement of fill. The trip reports do not direct themselves to the full scope of a surface treatment program. Interviews with

Messrs. Bock, Harber, Aberle, and Gebhart indicate that during these site visits surface treatment was discussed. After deciding jointly at this time that surface treatment would be done by "bucket grouting," Construction developed a surface treatment procedure, apparently without further detailed design and geological assistance. According to Mr. Gebhart, at one time a memorandum was prepared by Mr. Harber stating that consideration was being given to a surface treatment program using shotcrete. Mr. Arthur has explained that after telephone conversations with the Construction Engineer, it was felt that the studies necessary for the development of a shotcrete program were premature at that time. These communications are apparently the last time shotcreting was considered for surface treatment.

Mr. Aberle stated in the Denver interview that he felt that Construction knew the designers were aware of the open nature of the abutment surface rock because they specified the key trench excavation. Further, he assumed that Design was not particularly concerned about surface treatment because there was nothing in the specifications for surface treatment. Mr. Ringel of the Teton Construction Office Staff independently stated that no Denver Office people were noted as being around during surface grouting operations.

The field construction personnel developed surface treatment procedures to allay their concerns for compaction of zone 1 embankment material over large voids. The embankment and surface grouting inspectors, without the assistance of the project geologist, would designate which cracks were to be filled. The cracks were then filled by grouting with a 0.7:1 neat cement grout or a sanded cement grout depending upon the size of the opening. The surface treatment was performed in a narrow cleaned-up strip of rock just ahead of the advancing fill. Because of the stiffness of the grout mixes used, and because the concerns of Construction were generally for larger cracks, features less than 1/4 to 1/2 inch in width were ignored. In the right abutment, surface grouting was done under zone 1 material from elevation 5075 to 5205. Mr. Gebhart indicated that it was Design's intent.on for surface grouting also to be done in the side walls of the key trench. The inspectors have indicated that very little surface grouting was done in the side walls of the key trench. However, inspection of records shows that there was some surface grouting in the side walls. Postfailure inspection of the key trench revealed a number of untreated open features.

Surface grouting stopped at elevation 5205. Neither Mr. Bock from Design nor the Liaison Officer, Mr. Gebhart, was aware of the decision to stop surface grouting until after the Teton Dam failure. According to field personnel, the geologists also played no part in the decision to stop surface grouting.

Conflicting reports as to the openness of the rock surface above 5205 have been presented to this Group. According to Mr. Aberle, above 5205 the rock was more slabby and large fissures were replaced by "hundreds" of 1/4- to 1/2-inch-wide fissures which were left "untreated." Mr. Ringel stated that there did not appear to be a change in the fracturing characteristics which indicated no further need for surface grouting. Above elevation 5205, no attempt was made to remove native soil from cracks in order to replace it with design fill. The native crack filling material appeared moist and was probably ML, similar to zone 1 material.

During discussions with Mr. Aberle, he expressed his theory for the origin of the large fissures between elevations 5075 and 5205. He felt that many of the near-vertical fissures were behind large detached blocks that had moved toward the river. He was not sure if the designers knew of the possible displaced condition of these blocks. Removal of these blocks would have involved large excavation quantities.

Postfailure Investigations

Boring logs, joint transmissibility test results, and water pressure test results are presented in Appendix C-5.

The joint transmissibility tests were conducted in the bottom of the key trench in the area of the failure between stations 12+93 and 13+50. Water was ponded over exposed joint lengths of from 1.2 to 8.0 feet with maximum ponded water depths of from 0.4 to 1.0 foot. Water losses ranged from 0 to 1.1 gal/min. Water communication to the surface occurred along fissures outside the ponded areas. The amount of water transmitted along these joints would have been considerably greater under the heads imposed by the reservoir. However, the condition of these joints prior to failure is unknown.

For the purpose of examining the water pressure test data, we adopted evaluation criteria for water loss as follows:

Insignificant: less than 0.1 gal/min per foot of boring
Marginally significant: 0.1 gal/min to 0.5 gal/min per foot of boring
Significant: greater than 0.5 gal/min per foot of boring

Water losses during pressure tests in the spillway borings were generally insignificant. Marginally significant losses occurred in the upper portion of one boring in the right spillway bay. At station 3+00, significant water losses occurred in the upper portion of one of the deep angle borings.

A considerable number of shallow borings have been drilled in clusters along the right abutment between stations 12+75 and 14+26. Water losses were insignificant in the boring clusters at stations 12+75 and 13+15. Beginning with station 13+30 and going riverward, marginally significant and significant water losses occurred. The greatest number of significant takes occurred in the boring cluster at station 13+45. Water from some of the pressure tests leaked to the rock surface and also connected to the boring in the station 13+30 cluster which was inclined toward the river. During pressure testing, this riverward-inclined boring at station 13+30 leaked to the surface both up and downstream from the grout cap. However, the seepage quantities measured during these tests were probably not large enough to be considered abnormal for a typical grout curtain. The only concentration of grout takes during the construction of the right abutment centerline grout curtain occurred in the grout holes between stations 13+38 and 13+72.

A review of all geological and grouting information available to date reveals a coincidence of features between stations 13+00 and 14+00. A nearly straight line can be drawn between leaks observed downstream prior to and during failure and the upstream whirlpool. This line crosses the key trench near station 13+75. The following prefailure and postfailure features fall approximately along this line and in the key trench between stations 13+00 and 14+00:

1. Many significant, continuous joints running along and parallel to the line are shown on the as-built geologic drawings. Postfailure exploration of the area also shows well developed, continuous, open or partially filled joints trending parallel to this line. Because of the violent nature of the failure, the postfailure openness of joints observed at the elevation at the bottom of the key trench may not be entirely representative of that which existed during construction.

(It should be noted that postconstruction mapping refers to joints less than 1/2 inch open as being tight. We feel that those joints less than 1/2 inch wide which are capable of passing water should be considered open.)

2. Several significant grout takes in the auxiliary outlet works tunnel are in the area where this line crosses the tunnel.

3. Grout takes occurred in grout holes between stations 13+28 and 13+62 along the centerline curtain.

4. Scour of abutment rock upstream and downstream of the key trench was quite severe landward to station 13+20 where a set of prominent steeply dipping joints cross the key trench.

5. Water loss in postfailure core holes is concentrated in this general area.

These data from prefailure records, records of the failure itself, and postfailure mapping and testing seem to indicate that the failure could have been associated with more significant jointing in the area of the failure and greater difficulty in achieving adequate surface treatment and grouting for necessary protection of the fill. Unfortunately the key trench has been scoured away by the failure, so the condition of the key trench walls is not visible.

SUMMARY AND CONCLUSIONS

Design Concepts

The basic function of a grout curtain is to reduce seepage under a dam to tolerable limits. This serves to enhance embankment stability and to inhibit the loss of valuable reservoir water through the foundation. A grout curtain should never be considered capable of eliminating seepage through the foundation. Seepage in varying quantities will exist along the entire length of a grout curtain, especially in a highly jointed foundation such as that at Teton Dam. Whether or not a grout curtain is used, the embankment must be protected from this seepage to the degree dictated by the properties of the embankment materials and the foundation.

These basic design concepts were correctly addressed at an early date, but ensuing decisions within the design organization, were successful in shifting the emphasis toward creating a "tight" grout curtain which would then "eliminate" erosive seepage at the rock/embankment contacts. Great pains were taken to describe and specify an elaborate key trench and grout curtain, while embankment protection through the proper use of rock surface treatment and appropriately placed filters at embankment/rock interfaces were all but eliminated.

Surface Treatment

Rock surface treatment for embankment protection was not a specified item. It was handled informally after construction began. The responsibility for developing the scope and details of the program were left to the field construction office with no written instructions from Design concerning basic design concepts. Such written instructions would have been necessary to insure a full understanding that surface treatment was fundamental to the ultimate integrity of the dam. As a result, surface treatment was developed in a nonsystematic fashion. It was discontinuous. Where treatment was used, it did not tie into the grout curtain, and it was not applied to a significant abutment area above elevation 5205.

Curtain Grouting

The grout curtain was constructed by a methodology somewhat inconsistent with the experiences of the members of the Grouting Task Group. The performance of the abutment grout curtain is considered by the Group to be that of a single line rather than a multiline curtain. As is normal for most grout curtains, the effective depth was less than the depth of primary holes.

Grouting between stations 11+50 and 16+00 was done with less than 3 percent $CaCl_2$ and should not be considered abnormal in the location of the failure. Elsewhere the quality of injected grout in the outer rows may have been degraded by the excessive use of $CaCl_2$. The effects of high percentages of $CaCl_2$ are the subject of a laboratory testing program being conducted by the Bureau of Reclamation.

Our opinion is that the postfailure testing indicates that seepage through the grout curtain prior to failure was within tolerable limits. Testing does, however, indicate a possible shallow permeable zone in the grout curtain in the vicinity of station 13+50.

Mechanics of Failure

Evidence points to a significant concentration of features along a line between the failure leaks and the whirlpool. This line crosses the key trench between stations 13+00 and 14+00. Because of the concentration of joints and the ineffectiveness of the rock surface treatment, minor seepage in this area may have been a major factor in the development of the failure. Seepage moving along geologic features had direct access to zone 1 material immediately upstream of the key trench and grout curtain. This water then had egress into the fractured permeable rock immediately downstream of the grout curtain thus establishing a seepage path with a high hydraulic gradient.

Conclusions

1. A basic error was made by the Bureau of Reclamation, Division of Design, in assuming that a grout curtain tight enough to eliminate erosive seepage could be constructed.

2. Because of the error cited in item 1, Design failed to recognize the importance of protecting zone 1 material from the inevitable percolation of water through and along the surface of the foundation. Rock surface treatment was not a designed item, and it was inadequate for the protection of zone 1 material on the highly jointed rock foundation.

3. The grout curtain, as constructed, can be considered normal in terms of its ability to limit seepage. Like any grout curtain, it cannot be considered tight.

4. The well-developed set of open joints, which controlled the linearity of the significant features discussed previously, played an important role in the early development of adverse seepage paths.

NECESSARY CONTINUED INVESTIGATIONS

As of this writing, field investigations are still incomplete and laboratory tests of the various grout mixes are underway. It is anticipated that all information will be available for review by mid-1977. At that time, a supplement to this Report of Findings should be prepared to summarize and evaluate the results of those findings pertinent to the charge of the Grouting Task Group.

APPENDIX C-1

SUMMARY OF INTERVIEWS

by the

GROUTING SUBCOMMITTEE

TASK GROUP

TETON DAM

August 17-19, 1976, at Teton Dam
October 15-16, 1976, USBR, Denver

Revised February 4, 1977

INTRODUCTION

The following statements have been compiled from notes taken by the Grouting Task Group during informal interviews with the indicated persons at the damsite August 17-19, 1976, and at the USBR in Denver on October 15-16, 1976. Notes included in parentheses are provided as explanatory information. Included is a memorandum by Mr. Peter Aberle, Teton Project Field Engineer, commenting on some of the statements made by Messrs. McCabe and the USBR field inspection staff.

McCabe Brothers Drilling Company

Mr. Ed McCabe of McCabe Brothers, Drilling and Grouting Contractors

Mr. Bill McCabe of McCabe Brothers

Mr. Howard McCabe of McCabe Brothers (not able to attend)

Mr. V. M. Poxleintner of Morrison Knudsen, Construction Company

(Nearly all comments are from Ed and Bill McCabe. Mr. Poxleintner was present generally only as an observer.)

All drilling and grouting work was done under specific direction of the contracting officer. If there was any difference of opinion between the contractor and inspector, the contractor would advise the inspector but would abide by the inspector's decision. Grouting was done during all three shifts with one of the three McCabe brothers present nearly all the time.

The grout cap was constructed by excavating a narrow slot in the bottom of the key trench or cutoff trench. The slot was 3 feet wide by 3 feet deep with vertical sidewalls. The sidewalls were line drilled by drilling 3-inch holes on 6-inch centers. The blast holes were a single line on 18-inch centers, drilled to grade. The powder factor was recollected as averaging 1/4 lb/yd^3. It was hard to hold the sidewalls and bottom. However, overbreak and rock fracturing beyond the sidewalls and bottom did not appear to be severe. There were very little grout surface leakage along the edges of the grout cap.

No blasting was recalled as having occurred close to the completed grout curtain lines.

There was no interference with the grouting activities. The grouting of a particular stage in a hole was continued to completion through shift changes or weekends, when necessary.

Grout holes were located by Government inspectors. The inclination of the grout nipples were checked by Government inspectors after the nipples were set. The nipple inclination controlled hole inclination. The nipples were "Schedule 45" 2-inch black pipe and were 2 feet 8 inches long. Occasionally 4 to 5-foot nipples were required.

Drilling was done with 1-7/8-inch-diameter diamond plug bits on 5-foot barrels. Grout hole drilling was rapid in the welded tuff; 200 feet per shift per drill was achieved. Drilling was slower in the basalt. McCabe was capable of drilling angle holes parallel (more or less) to the slope of the abutments.

There were generally three or four holes 80 feet O.C. open ahead of one being grouted. There was very little hole-to-hole communication, maybe in a half dozen holes.

Surface leaks were estimated as occurring on less than 5 percent of the holes grouted. If leaks occurred during grouting of a staged hole, the leaks generally recurred during grouting of successive stages.

More rock fracturing was noted on the right abutment than the left and fractures on the right abutment appeared to be going every which way.

Grout was transferred up to 2,000 feet through insulated pipes from the grout batch plant to the holding tanks and pumps adjacent to the header.

Mixes ranged from 8:1 to 0.8:1 (water to cement ratio). Intermittent grouting was used on high taking holes. Five hundred sacks of progressively thicker mix (usually pumped for one-half hour for each successively thicker mix) would be pumped and then the hole would be flushed with water. After a specified period of time (4 hours if $CaCl_2$ was used and 6 hours otherwise), grouting would be resumed starting with a 3:1 or 2:1 neat cement mix for another 500 sacks or until refusal. Sanded mixes were also used at a ratio of 6 parts sand, 5 parts water, and 5 parts cement. All sanded mixes contained bentonite. $CaCl_2$ was used in amounts up to 8 percent of the cement content (by weight). Government forces sampled the grout for testing.

When using $CaCl_2$, grout temperature was checked to control the amount of $CaCl_2$ added. In the beginning, the USBR requsted $CaCl_2$ percentages as high as 10 percent. Because of severe problems with plugged lines, a maximum of 8 percent was used at McCabe's request.

Concern was expressed for the quality of the grout when high percentages of $CaCl_2$ were used. Flash sets with no further gain of strength were observed. Several instances were recalled where the grout arched in a hole. Redrilling found an open hole behind the arch. With high percentages of $CaCl_2$, the grout was in the hole within 1 minute after leaving the batch plant.

$CaCl_2$ was used in a few large takers on the centerline curtain and for all larger takers on the upstream and downstream curtains.

The final hole spacing of 10 feet on the centerline curtain was considered adequate. It appeared that "closure" was obtained.

Grouting was done with 100 and 300 lb/in^2 gages isolated by diaphragm gage savers.

The grout quantities ran over the estimate but not critically. The grouting operations did not significantly impede other construction activities.

The grouting contractor was not involved in the "surface grouting" of rock fractures intersecting the foundation surface. As a matter of observation, the grouting contractor noticed that there was very little "slush grouting" performed.

Jan Ringel, USBR Embankment and Surface Grouting Inspector

Worked on the "surface grouting" activities. No previous experience in dam construction.

All "surface grouting" was gravity grouting. Often, grout was placed in the same crack on several successive applications. Most of the cracks grouted were vertical. Both through going and localized cracks were present. The majority were localized.

The bottom of the key trench was relatively tight. There was some but very little surface grouting in the key trench. Most of the surface grouting was in the surface of the cutoff trench above the key trench.

Both a 0.7:1 neat cement and sand/cement grout were used. The decision on which to use was based on the size of the crack.

There were only a couple of cases where the grout was poured through pipes after the fill was brought up. These cases were not for fracture filling. They were for filling behind overhangs after the fill was brought up to the top of the overhang.

No blasting was recalled near a grouted area.

The reason for not carrying the surface grouting above elevation 5200 was that the field forces were instructed to stop at that point. There did not appear to be a change in the fracturing characteristics which indicated no further need for surface grouting. Above elevation 5200, no attempt was made to remove native soil from cracks in order to replace it with design fill. The native crack filling material appeared moist and probably ML, similar to zone 1 material.

No Denver office people were noted as being around during surface grouting operations.

Doug Jarvie, USBR Embankment and Surface Grouting Inspector

Mr. Jarvie has had previous dam construction experience.

When the surface grouting was first started, the "office" decided what was wanted and field forces took it from there. Surface grouting began when the fill was at approximately elevation 5075. No surface grouting was done in the bottom or sides of the key trench. When cracks were found, the inspector would call the batch plant for grout and determine the mix. Usually 0.8:1 neat cement grout. Sanded grout was used on the biggest cracks. No $CaCl_2$ was used.

An attempt was made to keep rock surface preparation work 5 feet ahead of embankment placement. Sometimes this distance dropped to 2 feet.

The final rock cleanup was usually done with air jets. Sometimes air/water jets were required. The rock surface was moistened before fill was placed. The natural fracture filling was silt and other debris. There were both long and short cracks. Most cracks seemed to run at angle with the centerline. There was no consultation with the geologists during the surface grouting operation.

There was no recollection of grouting through pipes after the embankment was brought up. Grout was placed under some big overhangs.

More surface grouting was done on the right abutment than the left.

Surface grouting was stopped because of orders "from above." At that time, the operation was getting to where there were fewer fractures. Jarvie would have continued the surface grouting further.

Ken Hoyt, USBR Embankment and Surface Grouting Inspector

The criterion was to grout those cracks in which fill material could not be compacted. The grout mixes and procedures were set by "policy." The first 15 cubic yards were to be neat cement. After that a sanded mix was used.

There were hardly any holes in the key trench that needed grouting. The only one recollected was on the high part of the left abutment; lean mix concrete was put in it.

There were more cracks on the right abutment than on the left.

There did not appear to be any preferred orientation to the cracks.

Sometimes a 4-inch (or so) hole would be found that would take 20 cubic yards of grout. One could hold one's hand over such a hole and feel cold air. Holes or cracks needing grout were marked with spray paint. Generally cracks less than one-fourth inch open were not grouted. All cracks were dry, even after a rain. As many as 10 voids per shift were grouted. Often more than one shift was required to complete a crack.

There was not any grouting through a pipe after fill had been placed.

Some grout leakage from the pressure grouting was observed on the sides of the key trench.

There was a geologist present on the day shift but he provided no input to the grouting operations.

The field personnel were told to stop surface grouting in the vicinity of elevation 5200. At that time, they were starting to run out of cracks but more could have been grouted. A few "tubes" were observed above 5200 which went back several feet and bent out of sight. One reason which appeared to be "floating around" was that the "stresses" would be low enough to allow quitting.

Claude Daniels, USBR Curtain Grouting Inspector

Has previous grouting experience.

The original grouting records consist of inspectors' reports, inspectors' drilling reports, and inspectors' grouting reports.

Grout placement criteria were developed by the supervisor of grouting after consultation with the grout inspectors. There was still plenty of room for exercise of individual judgment.

If a hole took 100 cubic feet per hour or more, grouting was stopped after 500 cubic feet had been placed (hole was stalled) and the hole was flushed with 20 cubic feet of water. After 4 to 6 hours, grouting was resumed. If $CaCl_2$ was used, the waiting time was only 3 hours. Occasionally a big taker would refuse after being "stalled." Reaming the grout hole would generally open things up again. When restarting a hole previously taking a sanded mix, a 5:1 neat cement grout was first used.

Tight grout holes were started with an 8:1 water/cement ratio mix. Wide open holes were generally started with a 5:1 mix and then thickened progressively (4:1, 3:1, 2:1, 1:1, 0.8:1) at about 15- to 30-minute increments. $CaCl_2$ was used when a sanded mix was used. Occasionally $CaCl_2$ was used without sand in thick (0.8:1) neat cement mixes. When the rate of grout take dropped to 50 sacks per hour, $CaCl_2$ would be discontinued. On the centerline, $CaCl_2$ was used only at stations 2+60 and 6+40.

Initial testing to determine acceptable percentages of $CaCl_2$ concentrated on set time. No strength tests were recalled. No test cylinders were taken during grouting with $CaCl_2$ mixes.

The criterion for drilling a centerline "closure" hole was a take over greater than 25 to 30 sacks at any one packer setting in the final scheduled holes (resulting in 10-foot spacing). It usually took two adjacent taking holes before a "closure" hole was drilled between them. Generally, no splitting was done around a single taker unless surface leakage was observed. With surface leakage, splitting was done by "feel." These criteria were developed by the field inspectors.

The grout holes appear to have been drilled rather straight. A number of vertical holes drilled around the auxiliary outlet works access shaft had been surveyed with a gyrocompass. These holes drifted less than 1 degree in 200 feet. No inclined holes were surveyed for straightness. The equipment was capable of operating in an inclined hole.

The rock which was excavated for the grout cap was highly fractured in the river section and "seamy" on both abutments. The difference between the right and left abutments was that the right abutment "seams" were open. These open seams got pipe for blanket grouting. Most of the time the line drill holes were preserved on the grout cap excavation side walls. There was very little surface leakage around the sides of the grout cap while grouting at depth. Some leakage was observed around the grout cap when grouting at the nipple.

There was blasting close to completed grout lines on the left abutment (25 to 50 feet away). There was no recollection of close blasting on the right abutment.

There was no formal grout sampling and testing program. Some samples were taken to determine the best amount of $CaCl_2$ additive.

The basalt in the valley section was responsible for quite a bit of packer leakage. Pneumatic packers were used in the basalt to help alleviate leakage. Artesian flow was encountered at the basalt/lake sediment contact. The alluvial materials penetrated appeared to be fine sand. The grout did not travel well in this material. The grout mix used was 10:1. Spacing between grout holes was 10 feet.

Voids were infrequently encountered in grout holes.

The rock appeared more massive with depth in the valley (one row) section of the grout curtain.

Brent Carter, USBR Geologist

(This interview was in the form of a discussion between Messrs. Carter, Coulson, and Fisher while on the right abutment. The following is a combination of information provided by Mr. Carter and observations by Messrs. Coulson and Fisher.)

There are three visible members of the welded ash flow rhyolite exposed on the right abutment.

The uppermost member is a fine-grained pinkish platy tuff. Subhorizontal flow jointing is very apparent; the average joint spacing is less than 1 foot. Second and third near vertical joint sets divide this rock into cuboids. This member appears to be about 30 feet thick.

An intermediate member is a highly jointed gray porphyritic tuff. This rock is cut by lots of steeply dipping and flat dipping joints. These joints appear to strike into and parallel to the canyon walls. Joint surfaces are coated with oxidation weathering products and, frequently, with calcium carbonates. The flow banding changes orientation rapidly. This member is about 60 feet thick. Two very prominent parallel vertical joints were observed in this member striking across the dam centerline. It appeared that the trace of one of these open joints could be seen both up and downstream of the key trench location. This member has been described as a transition between the upper and lower members. However, a breccia zone occurs at its base at about elevation 5160. This zone is a series of subparallel open planes with abundant carbonate deposition.

The intermediate member contained several prominent horizontal joints open from 1/4 inch to 6 inches. These were generally sinusoidal planes covered with calcite. Several delicate stalactites and stalagmites were observed, indicating that the rock mass, at least in the local area, had not been disturbed by the dam failure. The planes were open over large areas. Small, local pillars were in a state of failure, showing that the planes were either open over very large areas or that there had been superincumbent loads greater than the present overlying rock thickness would indicate.

The lower member is a gray prophyritic tuff which is considerably more massive than the overlying intermediate member. It is cut by widely spaced joints but many of these joints are open. While many joints sets were present, three could be easily identified. A prominent low-angle set dips slightly into the abutment. A near vertical joint set strikes nearly parallel to the canyon wall. A third high-angle joint set strikes (variably) diagonally upstream into the abutment and dips downstream. A number of joints were partially filled with neat cement and sand cement grout. No joint was observed to be completely filled. Both tops and bottoms of grout flow surfaces were observed. Open joints were observed next to grout-filled joints. One case was observed where grout flowed along a joint to a cross joint and flowed one way along the cross joint but not the other. The maximum observed width of a grout-filled joint was about 1 foot.

The intermediate tuff appears to warp down in an upstream direction around the more massive lower tuff.

There is a rock slump area in the upstream portion of the abutment.

Richard Bock, Head, Earth Dams Section, and William G. Harber, Head, Design Unit No. 2, retired January 1975

(Both involved with the design of Teton Dam as far back as 1964 when it was known as Fremont Dam. Nearly all questions answered by Mr. Bock. Mr. Harber generally seemed to agree with Mr. Bock's answers, but had little to contribute on his own.)

A grout curtain was included in the design for design safety to prevent high pressures at the downstream toe, to eliminate piping, and to control water loss through seepage under and around the embankment. A piping problem was anticipated and it was necessary to control seepage. Use of the word "eliminated" in the sentence "Erosion seepage under the embankment will be eliminated by injecting the foundaton with a grout mixture," from "Design Considerations for Teton Dam" was probably a poor choice of words.

Water testing during test grouting was the chief source of data to determine the depth of the grout curtain. From this it was assumed that a 310-foot-deep curtain in the abutments and a curtain extending below the alluvial material in the river bottom would be adequate. It was assumed that the effective depth of the curtain was the depth of the primary holes. Actual required depths were determined by construction.

High takes in lake sediments between stations 31+00 and 34+00 and at depths of 200 + were not thought important because (1) grout loss was thought to be concentrated at rock/sediment contact, and (2) back pressures experienced during grouting indicated that grout travel was limited to a bulb area around the hole. The decision to grout only the upper 10 feet of the lake sediments, even though this meant a shallower than normal curtain, was made jointly by Design and Construction personnel and was made in the interest of economy.

The responsibility for determining the potential for and consequences of reservoir rim leakage was given to the geologists.

The Bureau's experience has shown that seepage could always be controlled by grouting. No piezometers have ever been installed in dam foundations by the Bureau to look at the performance of a grout curtain. Historically, the Bureau has used pregrouting testing and installation experiences to assess the performance of a curtain and no direct observations of curtain performance have been made.

High takes in the upper 70 feet led to the decision to use a key trench. Estimates of takes calculated by geologists using void information from drill logs were thought by the designers to be wrong.

The performance of the curtain is that of a one-line curtain backed up by two outer lines installed for construction expediency. The Bureau seldom uses a multiline curtain with all lines closing. In general, blanket grouting is used rather than multiline grouting.

The role of the geologists during the design stage was to supply logs of holes, document water testing information, and construction profiles and sections to provide a general description and interpretation of the foundation geology. Geology does not become involved with foundation treatment design other than to have a look at it after its completion. There was nobody with formal geological training in the Dam Design Branch.

During construction, the project geologist, Mr. Sweeney (now retired) prepared as-built drawings of foundation geology. The cutoff trench below elevation 5100 was cleaned up and mapped at one time. Above 5100, cleanup and mapping was done progressively just ahead of fill construction. The Construction Engineer was responsible for approving the foundation for placement of fill. When the project geologist found a problem area, he was supposed to bring it to the attention of the Field or Construction Engineer who in turn notified Design. (When asked if the project geologist did this, there was no answer.)

The use of $CaCl_2$ to control the flow of grout is a standard technique as far as Design is concerned. Design knows of no previous experience with the high percentages of $CaCl_2$ used at Teton Dam and was not involved in the discussion to use $CaCl_2$ in unprecedented quantities.

Prior to the issuance of specifications, Design worked closely with Construction so all were in agreement on the details of the specifications. After bids were in, Construction was running the job with designers not involved unless a serious design problem occurred. All communications between Design and Construction were then through Mr. Gebhart, the liaison officer since 1973. The designers made few visits to the project during construction. Federal travel restrictions during the energy crisis had no impact on communications between Design and Construction. During the course of construction, the designer was supplied with the L-29 progress reports. If these reports indicated that a visit by Design was warranted, one was made. Designers generally like the current system and saw no access problem to Construction information during construction.

Messrs. Bock and Robison looked at left abutment key trenches in 1973 when the cut slopes of the key trenches were visible but before abutment areas under zone 1 had been stripped. It was concluded that open cracks should be bucket filled with ready-mix concrete as the fill progressed. No criteria were established for the size of crack to be filled, except that those which would readily accept ready-mix would be treated. Design understood that ready-mix from Rexburg was to be used.

The bottom of the key trench was to be treated through pipes set in significant cracks. The sides of the key trench were to be treated as the fill progressed by building a fill dike around the fissure and bucket grouting within the dike. (This appears to contradict the understanding of Construction; there was apparently no sidewall treatment.) No Design or Construction memorandums were sent to the field to detail the surface treatment program.

Design was not involved in the decision to terminate the surface treatment at elevation 5205. Mr. Bock did not know of the decision until after the failure. To Mr. Bock's knowledge, the geologists had no input into the problem. Mr. Harber thought Mr. Sweeney may have been participating with Mr. Robison in decisionmaking.

Mr. Bock is satisfied with surface treatment. Mr. Harber does not know what he would have done.

Surface treatment was not included in specifications. This is in agreement with standard USBR practice. Design does not think a better job would have been done had the specifications included surface treatment.

Mr. L. Gebhart—(Liaison officer between USBR design and construction organizations. Reports to Chief of Construction Division. Visits Teton routinely every 6 months and when needed. Liaison responsibility for USBR southwest region containing about six dams under construction plus all grouting USBR wide. Operates independently with no immediate staff. Main background in grouting. Came up through USBR grouting section.)

Mr. Pete Aberle, USBR Field Engineer

(Interviewed simultaneously with Gebhart. To the extent possible, the individual is identified in the text.)

Mr. Aberle was not involved in pilot grouting. Pilot grouting handled by Messrs. Bob Pittard and Ralph Mulliner in the field and Mr. Wayne Hermes. Chief values of pilot grouting:

1. Indicated need for 70-foot-deep key trenches to get to easily groutable material

2. Showed that the alluvial material under basalt is groutable

3. Showed that the basalt could remain

Mr. Gebhart reviewed grout curtain design. Tried to arrange for closing outer two rows, but Design wanted to limit drilling for economic reasons. General practice to close all lines where multiline curtains used. Recalled six to eight dams where this was the case. A list was supplied to the Task Group showing 12 USBR dams with two or four closed lines built from 1938 to 1971. Grout curtain depths are generally a function of water tables in abutment exploratory holes. A rule of thumb is H/3 + (25 feet to 75 feet), source unknown.

A key trench was not provided under the spillway because:

1. A trench filled with compacted fill would cause differential settlement, and

2. It was not economically feasible to fill a trench with concrete. While grouting spillway, one or two holes connected to tunnel below.

Mr. P. Aberle

Lake sediments between stations 32+80 and 34+00 had high takes. Construction assumed that because of the back pressures associated with these takes that hydrofracturing or displacement of material was occurring to allow the takes. Sediments and associated takes were encountered at approximately 200 feet, above the 260 to 310 depth criterion for the curtain. Construction thought that grouting could be stopped 10 feet into the sediments without impairing the depth criterion of the curtain enough to matter. This then became the standard practice. The design considerations gave instructions to limit travel of grout to within 100 feet of the curtain. In order to accommodate this, Construction used $CaCl_2$ and sand where, and in the quantities, they thought to be appropriate. Mixes with up to 4 percent $CaCl_2$ were tested in the laboratory. When the contractor started using river water at temperatures as low as 34°F, more

$CaCl_2$ was added for temperature control. At 10 percent $CaCl_2$, the mix froze solid. Mixes with 8 percent were hard to control, so the normal maximum was approximately 6-1/2 percent.

Mr. L. Gebhart

Normal USBR practice is to limit $CaCl_2$ to 2-1/2 percent. The percentages used at Teton were unprecedented.

Mr. P. Aberle

The temperature of the grout at the pump was the criterion used for adding $CaCl_2$.

Mr. L. Gebhart

When large voids were discovered in the left abutment key trench, Liaison made a visit to the project to investigate. At that time, the key trench excavation was completed down to elevation 5200. The slabby and broken nature of the side walls of the upper part of the trench were observed. Discussion followed between Design and Construction, Geology not included, concerning the treatment of the features. Mr. Harber of Design wrote a memorandum asking for surface treatment by shotcreting. This memorandum did not go anywhere. The preference of Construction, slurry grouting, was decided upon. The specifics of the procedure were left up to Construction, and most were handled verbally. Liaison was not aware that surface treatment stopped at elevation 5205.

Mr. P. Aberle

Construction knew the designers were aware of the open nature of the abutment surface rock because they specified the key trench excavation. Further, Construction assumed that Design was not particularly concerned about surface treatment because there was nothing in the specifications for surface treatment.

(Mr. Aberle's evaluation of the geologic explanation for the large open fissures follows. These ideas appear to be his own, not necessarily those of the project geologist.)

Many of the large, nearly vertical, open or filled fissures below 5200 were behind large detached surface blocks which had moved toward the river. Most of these large detached blocks were left in place because the specifications called only for surface cleanup under zone 1 material. The surface grouting program was tailored to treat these large features. Construction generally concurred with leaving abutment rock in place because of the large excavation quantities required for removal. Not sure if the designers knew of the displaced condition of these large blocks.

Above 5205, the rock was more slabby and the large fissures were replaced with hundreds of 1/4- to 1/2-inch-wide fractures which were left untreated.

United States Department of the Interior

BUREAU OF RECLAMATION
TETON PROJECT OFFICE
P.O. BOX 88
NEWDALE, IDAHO 83436

IN REPLY REFER TO:

February 5, 1977

Memorandum

To: Grouting Task Group U.S. Department of the Interior Teton Dam Failure Review Group

From: Peter P. Aberle - Field Engineer, Teton Basin Project, Newdale, Idaho

Subject: Comments pertaining to Grouting Subcommittee Task Group Report

This memo is in regard to comments asked for by the Grouting Task Group to their "Report of Findings".

Whirlpool: Original statements and memos stated that the whirlpool was located at dam station 13+00 at approximately elevation 5295. Through further discussion and observation of photographs the approximate station has been determined to be station 13+75, elevation 5295.

The percentage of calcium chloride used as an admixture to cement to increase the hydration rate of the grout mixture: In McCabe Brothers statement page 2, Appendix "A", McCabe Brothers stated that ten percent calcium chloride was used for a short time and later this was lowered to eight percent . Ten percent calcium chloride was used only on one occasion and the use of eight percent calcium chloride was not prevalent. Mixes containing an eight percent calcium chloride were too difficult to control in regard to set time and therefore it was used in an experimental manner only. McCabe requested that a maximum of six percent calcium chloride be used.

In the area of the failure between dam station 11+50 and 16+00 calcium chloride was used only on 4 occasions in an open joint at station 14+20 16 feet downstream of centerline. The maximum percentage was three percent for 250 cubic feet of cement, and one percent for an additional 1340 cubic feet of cement. Calcium chloride was not used in grout injected through a pipe nipple on any curtain within the limits of the stations stated above.

On page 14 of the Grouting Task Group Report the statement "Occasionally grouting was done through pipes to grout below a rock overhang". The statement was made by Mr. Ringel that there were a couple of cases where grout was poured through pipes. Mr. Jarvie and Mr. Hoyt do not recall using pipes to grout overhangs. The subject of grouting overhangs through pipes was discussed by the field forces for a few isolated overhangs but was never actually performed.

REVOLUTION BICENTENNIAL 1776-1976

On page 1 of Appendix "A" it stated, "There was very little grout surface leakage along the edges of grout cap". There was so little leakage along the grout cap that it can be termed insignificant. Only two occasions can be recalled.

On page 2, paragraph 2, Appendix "A", "If leaks occurred during grouting of a staged hole, the leaks generally reoccurred during grouting of successive stages". This situation did not exist. If surface leaks occurred from any stage of a hole the leaks were caulked and sealed. Grouting was then suspended for at least eight hours to give the sealed leaks sufficient time to heal before grouting was resumed. It is possible that leaks at new locations from successive stages could occur, however, this is a normal occurrence. Very few surface leaks occurred from stages at depth. Most leaks occurred when hooked to the nipple.

On page 2, paragraph 7, Appendix "A", "With high percentage of $CaCl_2$ the grout was in the hole within <u>one</u> minute after leaving the batch plant". At a maximum pumping rate of 250 cubic feet of cement per hour, the maximum pumping rate was 5.4 cubic feet of grout mixture per minute. With the time required to move the grout through the distribution lines and the time required to pump empty a 25 cubic foot capacity tub located at the pump site it was impossible to inject grout any sooner than six mimutes after it was mixed.

On page 2, paragraph 10, "Grouting was done with 100 and 300 PSI gauges isolated by diaphram gauge savers". The gauges were also internally filled with glycerine which dampened the pulsations from the duplex piston type pumps. This made the life expectancy of the gauges up to 20 times longer and much easier to read. Viscosity of the oil in the diaphram was adjusted to coincide with weather temperature.

On page 6, paragraph 1, "There was blasting close to the completed grout line on the left abutment (25 to 50 feet away)". This blasting was controlled blasting in the grout cap and occurred in the vicinity of station 19+50. Another blast in the vicinity of station 24+75 was made upstream of the grout cap where a small overhang was removed also under a controlled blast situation.

Peter Plaucki

APPENDIX C-2

RECOMMENDED FIELD EXPLORATIONS TO INVESTIGATE THE INTEGRITY OF THE GROUT CURTAIN

TETON DAM

Independent Panel

The Independent Panel exploration recommendations were presented in a 5 August 1976 report to the Secretary of the Interior and modified in a 24 August 1976 letter to the Teton Project Construction Engineers. The recommendations are quoted as follows:

5 August 1976 Report

"The purpose of the program is twofold: first, to determined 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° away from the river, and one at an angle of 30° toward 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."

24 August 1976 letter

"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 veritcal 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."

Federal Review Group, Grouting Task Group

Two successive exploration programs were recommended by the Grouting Task Group. The first recommendation outlined the program which the Task Group, as a whole, felt was the minimum program which would adequately characterize the right abutment grout curtain. The second recommendation is a reduction as a result of a determination by the Interior Review Group that the first recommended program was too extensive. Both recommended programs are quoted below along with reservations concerning the scope of the second, reduced program.

First Recommended Program

"The grouting subcommittee concurs with the independent panel's program to investigate the bottom of the key trench and grout curtain as outlined in paragraph B of the schedule appended to their 5 August 1976 report. In addition, the following are recommended.

Subsurface Explorations

Core Holes

"Recommend that 12 NX size wire line core borings be drilled at locations along the grout cap centerline and at locations up and downstream from the grout cap. Continuous core should be taken. The drilling of each hole should be continuously inspected by a geologist-inspector. The information required on USBR Geologic Log of Drill Hole, Form 7-1337 will be satisfactory. The inspector should pay special attention to and note lithology, weathering, water loss, color of returned drill water, core recovery, RQD, fracture frequency, occurrence and condition of grout, and location and description of each discontinuity. Fractures should be plotted in the log column. Color photographs should be taken of each core box as it is filled with a ruler in view for scale, and closeups taken with labels of questionable features.

Grout Cap Centerline Borings

"These borings should be drilled in the plane of the centerline grout row and should be inclined 30^{o} from the vertical toward the river. Locations and depths are:

Station	Depth (feet)
10+14	220
12+06	260
12+50	230
13+28	200
13+83	180

Upstream Grout Row Boring

"One 300 feet deep boring should be drilled in the plane of the upstream grout row at station 13+28. The boring should be inclined 30^{o} from the vertical toward the river.

Up and Downstream Crossing Borings

"These borings should be drilled across the grout curtain to provide a check on the up and downstream thickness of the effectively grouted area. Locations, inclinations and depths are:

Station	Offset	Inclination from Vertical	Depth (feet)
12+90	15′ D/S	27^{o} D/S	70
12+90	15′ U/S	27^{o} U/S	70
13+30	15′ U/S	27^{o} D/S	90
13+30	15′ D/S	27^{o} U/S	90
14+20	15′ U/S	27^{o} D/S	70
14+20	15′ D/S	27^{o} U/S	70

Water Pressure Tests

"Each of the above core borings should be water pressure tested. The pressure tests should be performed at approximate five feet intervals. Double packers should be used with the interval between packers at least one foot greater than the test interval. This will avoid blanking of a critical open fracture by successive packer settings. The packer string should be arranged so that testing can be performed between the two packers and below the bottom packer. Water test pressures should be set by the same criteria as used during grouting."

Borehole Photography

"All core boring walls should be continuously photographed, in color, with an 'NX Borehole Camera'. The developed film records should be analyzed and borehole camera logs should be prepared which show the depth and elevation, strike, dip, width, and filling of each and every fracture. In addition, comments on rock texture, lithology and condition should be included.

"Borehole TV surveys may be advantageous for rapid reconnaissance of the boring walls.

Calyx Holes

"In addition to the small diameter core borings, 30 or 36 inch vertical inspection borings may be necessary. The need for these borings will be established based on the results of the core boring, water testing, and a core drilling apparatus such as a shot or calyx drill. The use of a rotary drilling rig with air and/or water circulation should be avoided. If significant through going vertical features are observed in the up and downstream walls of the key trench, their condition below the grout cap can be checked with inspection borings.

Laboratory Testing

"A testing program should be designed by the USBR to analyze the strength characteristics of the various grout mixes using from two to ten percent $CaCl_2$. The tests should be set up to emulate field conditions where the grout was delivered at temperatures as high as 90°F and injected into a rock mass with temperatures on the order of 50° to 60°F. Set times and strength versus time characteristics should be observed at the ambient rock temperatures.

"Tests to determine the erodibility of the weaker grout mixes should be performed. The pinhole dispersion test appears to be applicable for this purpose. If the weaker mixes prove to be erodible, a more comprehensive testing program would need to be devised."

Second Recommended Program

Subsurface Explorations

Core Holes

"Recommended that 14 NX size core borings be drilled as follows and in the order of priority shown:

Priority I	The three clusters of three core holes each requested by the Independent Panel at Stations 12+65, 13+05, and 13+40.
Priority II	Two 100 feet deep core holes, inclined in the plane of the centerline row at 30° from vertical toward the river at Stations 12+–6 and 13+83.
Priority III	One 100 feet deep core hole inclined in the plane of the centerline row at 30° from vertical toward the river at Station 13+40.
Priority IV	Two 90 feet deep core holes: (1) 15 feet upstream from Station 13+30 and inclined 20° from vertical in the downstream direction, and (2) 15 feet downstream from Station 13+30 and inclined 20° from vertical in the upstream direction.

"The results of the core drilling, as it proceeds, may modify the above outlined program. Additional drilling may be required."

Water Pressure Tests

"All borings should be water pressure tested. The priority I holes should be tested as described by the Independent Panel. The priority II-IV borings should be water pressure tested in 25 foot stages. If leakage greater than 2 gpm occurs for any 25 foot stage, the stage should be re-tested in 5 foot intervals between double packers.

Borehole Photography

"All core boring walls should be continuously photographed in color with an NX Borehole Camera. The developed film records should be analyzed and borehole camera logs should be prepared which show the depth and elevation, strike, dip, width, and filling of each fracture. In addition, comments on rock texture, lithology and condition should be included."

Laboratory Testing

"A testing program should be proposed by the USBR to analyze the strength characteristics of the various grout mixes using from two to ten percent $CaCl_2$. The tests should be set up to emulate field conditions where the grout was delivered at temperatures as high as 90°F and injected into a rock mass with temperatures on the order of 50° to 60°F. Set times, bleeding, and strength versus time characteristics should be observed at the ambient rock temperatures."

Excerpt from Memo Transmitting Second Set of Recommendations to Chairman, Federal Review Group

"I (the Chairman, Grouting Task Group) am still concerned about the scope of the grout curtain field investigations as the program is now structured. The Independent Panel drilling program appears to be aimed at finding the specific grout curtain flaw (if one exists) which contributed to failure. The error in such an approach is that if the flaw in the grout curtain is not discovered no argument will have been developed for its absence. We will have merely missed it. A more general program, designed to characterize the condition of the right abutment grout curtain will allow us to make more supportable statements about the probable presence or absence of a 'fatal flaw', even if one is not actually drilled. If all the priority I-IV borings are drilled, we may be able to argue that we achieved general coverage to 60' in depth in the area of maximum interest. If we only get the priority I or the priority II borings we may end up with no apparent flaws and no basis for arguing that there are none."

APPENDIX C-3

TESTING PROGRAM FOR HIGH $CaCl_2$ GROUT MIXES

TETON DAM

Calcium chloride was used in unprecedented quantities in the grout mixes at Teton Dam to: (1) decrease set time and limit the flow of grout in the upstream and downstream curtains, and (2) control grout mix temperatures. Mixes with 6 to 6-1/2 percent by weight $CaCl_2$ were used routinely. Mixes with 8 percent $CaCl_2$ were used during the coldest winter months with some difficulty in controlling pumpability. Mixes with 10 percent $CaCl_2$ were tried without success. High $CaCl_2$ mixes with temperatures ranging from 70^o to 90^o were injected into a rock mass at much lower ambient temperatures with unknown results.

The McCabe Bros. Drilling, Inc., during their interviews with the Grouting Task Group and in their letter of August 25, 1976, to Morrison-Knudsen Company, Inc., expressed concern for the ability of the $CaCl_2$ mixes to flow sufficiently to form a continuous curtain. The experience of the McCabe Brothers indicates that flash sets and arching within holes did occur with only 3 percent $CaCl_2$, and that significant honeycomb areas may exist in the grout curtains.

The Grouting Task Group made the following recommendations for testing high $CaCl_2$ grout mixes in its August 27, 1976, Interim Report of Findings.

> A testing program should be designed by the USBR to analyze the strength characteristics of the various grout mixes using from 2 to 10 percent $CaCl_2$. The tests should be set up to emulate field conditions where the grout was delivered at temperatures as high as 90^oF and injected into a rock mass with temperatures on the order of 50^o to 60^oF. Set times and strength versus time characteristics should be observed at the ambient rock temperatures.
>
> Tests to determine the erodibility of the weaker grout mixes should be performed. The pinhole dispersion test appears to be applicable for this purpose. If the weaker mixes prove to be erodible, a more comprehensive testing program would need to be devised.

In response to this recommendation, Jim Pierce of the USBR Denver office prepared an outline of the proposed testing program. This outline was discussed with the Grouting Task Group on October 16 in Denver and several minor revisions were made. The following outline is currently being followed for development of the testing program. To date only a small number of drying shrinkage tests have been performed and no results have been presented. A final report should be issued during March 1977.

PROPOSED TETON DAM GROUT TESTING PROGRAM

Objective: Analyze physical properties of various grout mixes.

Test Program:

A. Variables:

1. Grout Mix Proportions

   a. Neat Cement Grout

      (1) 0.8:1

      (2) 5:1

      (3) 8:1

   b. Sand-cement Grout (2 percent bentonite)

      (1) 1:1:1

      (2) 1:1:1.4

      (3) 1:1:1.8

2. Calcium Chloride Dosage

   a. Control—0 percent—all mixes

   b. 2 percent

   c. 6 percent Mixes } 1.a.(1), 1.b.(1), 1.b.(2), 1.b.(3)

   d. 8 percent

3. Grout Temperature

   a. 35–40°F

   b. 50–60°F } All mixes

   c. 70–90°F

4. Strength Test Ages

   a. 7 days

   b. 28 days } All mixes

   c. 90 days

B. Testing

1. Time of set

2. Unconfined compressive strength (2 inches$^{\phi}$ x 4 inches)

3. Triaxial shear strength (2 inches$^{\phi}$ x 4 inches)

4. Drying shrinkage

5. Erodibility

6. Permeability

C. Test Procedures

1. Time of set: CRD-C82 or ASTM C 807

2. Unconfined compressive strength: ASTM C 39

3. Triaxial shear strength: USBR E-17

4. Drying shrinkage: ASTM C 157

5. Erodibility: Modified physical erosion test for soil

6. Permeability: Modified USBR concrete test or E-13

D. Operations

1. Mixing—Hobart Laboratory Mixer 1725 rpm

2. Ambient casting and curing temperature—60°F

3. Tin molds

4. Specimen test condition—dry

E. Proposed Schedule and Estimated DL + A Costs

1. Phase I—$6,650

   a. Addition literature search

   b. Time of set

   c. Unconfined compressive strength

   d. Drying shrinkage

2. Phase II—$1,500

   a. Triaxial shear strength tests

3. Phase III—$2,100

   a. Erodibility testing

4. Phase IV—$2,250

   a. Permeability testing

5. Phase V—$1,000

   a. Report

APPENDIX C-4

SUMMARY OF DESIGN
FOR
FOUNDATION PRESSURE GROUTING

TETON DAM

The information on the design of the Teton Dam pressure grouting program was obtained from: (1) USBR Design Considerations for Teton Dam, October 1971; (2) ASCE Specialty Conference Paper by Peter P. Aberle, "Pressure Grouting Foundation on Teton Dam," 1976; and (3) Teton Dam Contract Specifications DC-6910.

The design purpose of the Teton Dam pressure grouting program can be summarized by the following two quotations:

1. "Erosive seepage under the embankment will be eliminated by injecting the foundation with a grout mixture." (Design Considerations, page 9.)

2. " . . . to construct an impermeable curtain for the foundation of Teton Dam . . . " (Aberle's 1976 ASCE paper, first page).

It was anticipated that large grout quantities would be required to produce a tight curtain and that special procedures would be required to prevent travel of the grout beyond the limits of the impervious barrier. Grout more than approximately 100 feet from a vertical plane through the grout cap would serve no useful purpose. The grouting operation was designed to, in part, limit travel of grout.

To obviate massive grout takes on the abutments, 70-foot-deep key tranches were to be excavated in each abutment above elevation 5100 except under the spillway. The key trench was not extended under the spillway because of fear of differential settlement.

Critical areas in the foundation were to be treated by drilling and grouting three staggered rows of grout holes. The outer rows of holes were to be drilled at a specified spacing and injected with a limited volume of grout based on the probable volume of voids in the zone being grouted. No attempt would be made to completely seal the planes defined by the outer rows of holes by continuing to drill and grout intermediate (closeout) holes until they refused to take grout. After the outer rows were drilled and grouted, the centerline grout cap row was to be drilled and grouted by split spacing techniques until it was closed out.

In addition to the grout curtain, blanket grouting was to be provided to reinforce the curtain in areas where the bottom of the key trench or cutoff trench contains open fractures or other defects.

The critical areas requiring a three-row grout curtain were the abutments above elevation 5100 and an intracanyon basalt-rhyolite contact zone occurring under the valley floor between approximate dam stations 20+00 and 24+00. Areas which were anticipated to need blanket grouting were in the vicinity of the spillway where the key trench was omitted and basalt-rhyolite contact zones in the bottom of the cut-off trench.

The curtain grouting method was split spacing, packer grouting with provision for stage grouting where significant drill water loss (greater than 50 percent) or caving occurred during grout hole drilling. If a grout hole could be drilled to its planned depth, the hole would be successively water tested and grouted in sections, under a packer, from the bottom up. Each section would only be grouted if water losses during water pressure testing exceeded 1 cubic foot in 5 minutes.

The final hole spacing in the outer grout rows on the abutments was to be 20 feet to a depth of 60 feet, 40 feet to 160 feet deep, and 80 feet to 260 feet deep. Closure holes would not be drilled. The initial spacing for centerline grout cap holes on the abutment was to be 10 feet to a depth of 60 feet, 20 feet to 110 feet deep, 40 feet to 160 feet deep, and 80 feet to 260 feet deep. Closure holes would be drilled where grout takes in the last sequence of planned borings indicated that further grouting was necessary. In the areas where holes on 5-foot centers would fail to close out at depths exceeding 60 feet, additional closure holes were to be drilled 5 to 10 feet upstream from the centerline. On the right abutment, the 80-foot spaced primary holes would be extended from 260 to 310 feet in depth.

The holes in the upstream and dowstream rows were to be drilled vertically. The holes in the centerline row were to be inclined into each abutment at 30° from vertical.

Below elevation 5100, the hole spacings and depths in the single-line grout curtain were to be the same as those in the centerline row on the abutments. Closure holes would be drilled as needed.

The outer rows of grout holes in the intracanyon basalt area were to be drilled on 10-foot centers to a depth of 15 feet into the rhyolite underlying the basalt. No closure holes were to be drilled. The centerline row grout cap holes were to be drilled on the same initial pattern as the centerline row on the abutments, except that the 10-foot spacing was to extend 15 feet into the rhyolite. Closure holes would be drilled as needed.

The blanket grouting was provided to allow special treatment of open cracks, jointed areas, high grout takes, and other defects uncovered in the bottom of the cutoff trench or disclosed by the curtain grouting. The blanket grouting was to have been performed by low-pressure grouting through shallow drill holes.

The sequence of grouting where three grout rows were used was (for any section of the curtain) the downstream row first, the upstream row second, and the centerline row third. Where required, blanket grouting would be performed prior to curtain grouting.

It was anticipated that the grout water-cement ratio would vary from 10:1 to 0.8:1 by volume. The grouting maximum injection pressure, with certain exceptions, would be 10 psi or 0.75 psi per foot of depth measured normal to the ground surface from the packer to the ground surface, whichever was the greater.

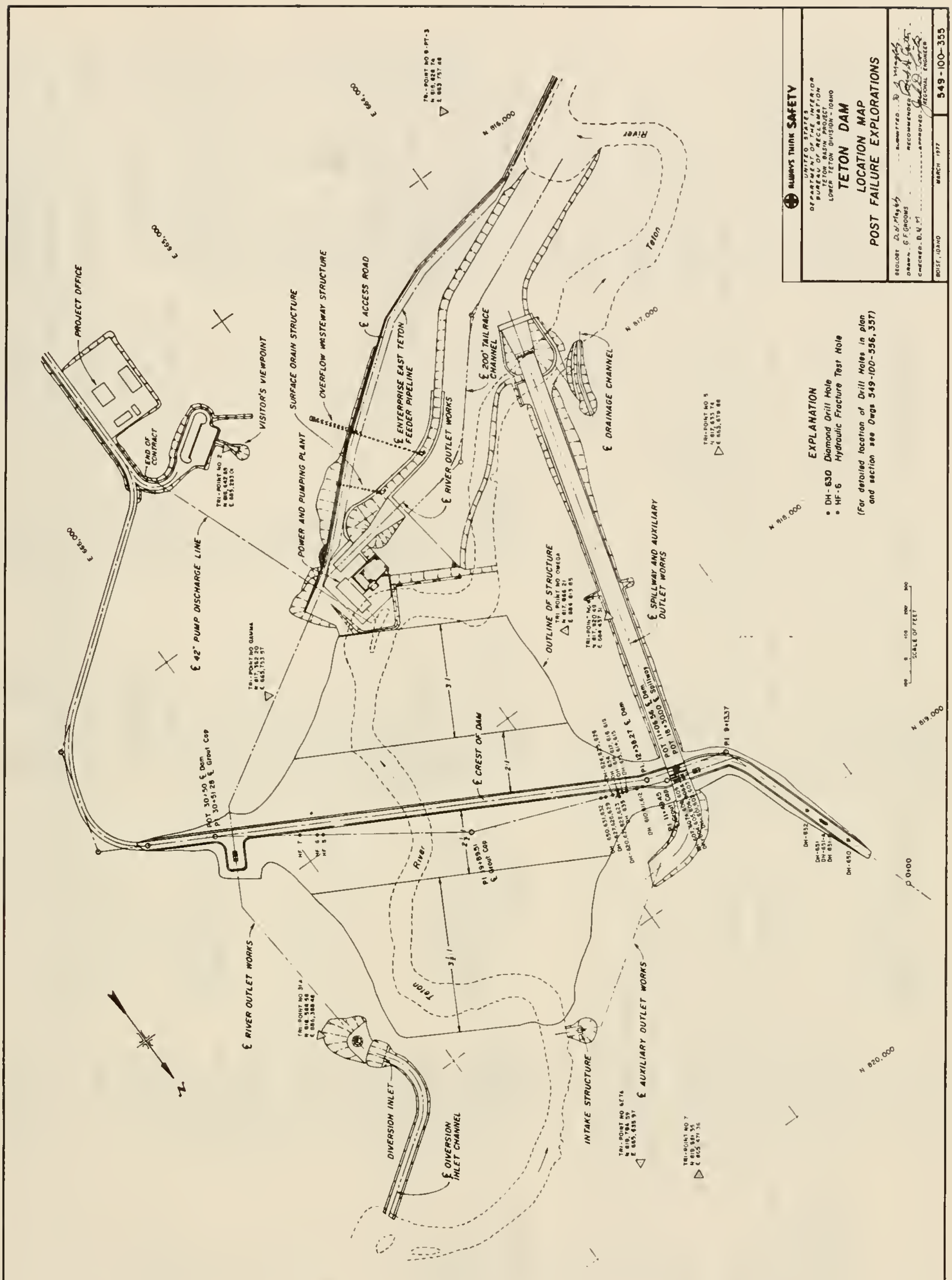
In order to minimize grout travel up and downstream from the planned grout curtain, the use of calcium chloride ($CaCl_2$), sand-cement grout mixes, and intermittent grout pumping was planned for grout holes which took large amounts of grout at less than normal pressures. The grout mix was to be progressively thickened and then sand and/or $CaCl_2$ added if the rate of grout take remained high. If the hole continued to take, consideration was to be given to intermittent pumping when a take of more than 500 sacks occurred in a 20-foot stage. Intermittent pumping would include flushing the hole with not more than 25 cubic feet of water, allowing the grout to set for up to 8 hours, and then continuing pumping. Closeout holes were required on each side of a centerline hole where intermittent pumping was performed.

The criterion for discontinuing grouting was a pumping rate of less than two sacks of cement per hour, a rate of less than 1 cubic foot of mix in 10 minutes at injection pressures up to 50 psi, in 8 minutes from 50 to 100 psi, and in 5 minutes if the injection pressure was greater than 100 psi.

APPENDIX C-5

BORING LOGS AND JOINT TRANSMISSIBILITY AND WATER PRESSURE TEST DATA

TETON DAM


ALWAYS THINK SAFETY
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TETON BASIN PROJECT
LOWER TETON DIVISION - IDAHO
TETON DAM
LOCATION MAP
POST FAILURE EXPLORATIONS
BOISE, IDAHO
MARCH 1977
549-100-355
EXPLANATION
• DH-630 Diamond Drill Hole
• HF-6 Hydraulic Fracture Test Hole
(For detailed location of Drill Holes in plan and section see Dwgs 549-100-356, 357)
SCALE OF FEET
PROJECT OFFICE
VISITOR'S VIEWPOINT
END OF CONTRACT
SURFACE DRAIN STRUCTURE
OVERFLOW WASTEWAY STRUCTURE
℄ ACCESS ROAD
℄ ENTERPRISE EAST TETON FEEDER PIPELINE
POWER AND PUMPING PLANT
℄ RIVER OUTLET WORKS
℄ 200' TAILRACE CHANNEL
℄ DRAINAGE CHANNEL
℄ 42" PUMP DISCHARGE LINE
OUTLINE OF STRUCTURE
℄ SPILLWAY AND AUXILIARY OUTLET WORKS
℄ CREST OF DAM
Teton River
℄ RIVER OUTLET WORKS
DIVERSION INLET
℄ DIVERSION INLET CHANNEL
INTAKE STRUCTURE
℄ AUXILIARY OUTLET WORKS
N 816,000
N 817,000
N 818,000
N 819,000
N 820,000
E 665,000
E 666,000
E 663,000

Status of Field Exploration Drilling - Teton Dam

Explanation

Inclusion of a total depth (TD) figure means the boring has been drilled.

Inclusion of a Date Drilled means a summary log has been prepared for the boring.

Inclusion of the term "Log" means the Grouting Task Group has received the summary log.

Inclusion of the term "WPT" means the Grouting Task Group has received the water pressure test results for the boring.

Requestor	Hole Number	Location	Inclination	Planned Depth	Date Drilled	Remarks
Independent Panel	DH-601	Right Spillway Bay	60° Rt	75'	9-11-76	TD109.2 Log WPT
	DH-602		Vertical	75'	9-8-76	TD94.7 Log WPT
	DH-603		60° Lt	75'	9-14-76	TD108.7 Log WPT
	DH-604	Center Spillway Bay	60° Rt	75'	9-21-76	TD109.7 Log WPT
	DH-605		Vertical	75'	9-18-76	TD96.0 Log WPT
	DH-606A		60° Lt	75'	9-24-76	TD110.0 Log WPT
	DH-607A	Left Spillway Bay	60° Rt	75'	10-2-76	TD109.5 Log WPT
	DH-608		Vertical	75'	9-28-76	TD125.8 Log WPT
	DH-609		60° Left	75'	10-6-76	TD145.0 Log WPT
	DH-612	Sta 12+65	Vertical	20'	10-31-76	TD23.5 Log WPT
	DH-611		67½° Rt	20'	11-3-76	TD23.9 Log WPT
	DH-610		45° Rt	20'	11-2-76	TD21.8 Log WPT

Status of Field Exploration Drilling - Teton Dam

Requestor	Hole Number	Location	Inclination	Planned Depth	Date Drilled	Remarks
	DH 615	Sta 13+05	Vertical	20'		TD 24.7 Log WPT
	DH 614		67½° Rt	20'		TD 24.7 Log WPT
	DH 613		45° Rt	20'		TD 24.8 Log WPT
	DH-622	Sta 13+40	Vertical	20'		TD 23.5 Log WPT
	DH 621		67½° Rt	20'		TD 23.5 Log WPT
	DH 620		45° Rt	20'		TD 100.0 Log WPT
	DH 623		67½° Lt	—		TD 21.4 Log WPT
	DH 616	Sta 13+20	45° Rt			TD 24.3 Log WPT
	DH 617		67½° Rt		11-8-76	TD 23.9 Log WPT
	DH-618		Vertical		11-4-76	TD 24.2 Log WPT
	DH-619		67½° Lt		11-6-76	TD 26.7 Log WPT
	DH-624	Sta 13+67	45° Rt			TD 24.3 Log WPT
	DH-625		Vertical			TD 25.3 Log WPT
	DH-626		67½ Lt			TD 26.4 Log WPT
	DH-627	Sta 14+00	45° Rt			TD 21.7 Log WPT
	DH-628		Vertical			TD 21.6 Log WPT
	DH-629		59° Lt			TD 21.0 Log WPT
	DH-630	Sta 14+16	45° Rt			TD 21.7 Log WPT
	DH-631		Vertical			TD 21.6 Log WPT
	DH-632		67½° Lt			TD 23.1 Log WPT

Status of Field Exploration Drilling - Teton Dam

Requestor	Hole Number	Location	Inclination	Planned Depth	Date Drilled	Remarks
	OH-650	Sta 3+00	59° Lt		10-15-76	TD 351.5 Log WPT
	OH-651	Sta 4+34	Vertical		10-12-76	TD 622.4 Log WPT
	OH-652	Sta 5+11	57° Rt		10-2-76	TD 450.0 Log WPT
Grouting Task Group		Sta 12+06	60° Lt	100'		
		Sta 13+40	60° Lt.	100'		
		Sta 13+83	60° Lt.	100'		
	OH-634	Sta 13+32	75° Upstream	90'		
	OH-633	Sta 13+33	70° Downstream	90'		TD 90.0 Log WPT

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-601 LOCATION Right Spillway Bay* GROUND ELEV. 5299.0 DIP (ANGLE FROM HORIZ.) 60°

COORDS. N. E.

BEGUN 9-8-76 FINISHED 9-11-76 DEPTH OF OVERBURDEN 0.0 TOTAL DEPTH 109.2 BEARING N74°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment

Longyear "34" drill, skid mounted.

Driller

Frank Martin

Drilling Methods

Advanced hole using NWD-3 split tube core barrel with clear water.

Drilling Conditions

0.0'-20.3': Fast and smooth.

20.3'-25.1': Rough and slow.

25.1'-109.2': Fast and smooth.

Water Return

0.0'-20.3': 100%

20.3'-40.0': 95%

40.0'-80.4': 90%, lost water at 80.4', regained after 4 min.

80.4'-109.2': 85%

Depth to Water ** During Drilling

Date	Depth Hole	Depth Water
9-9	4.0	3.0
9-10	49.6	35.6
9-11	93.8	80.4

TYPE AND SIZE OF HOLE: D Nx

CORE RECOVERY (%): 97, 100, 100, 100, 100, 100, 100, 100, 100, 96, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100

PERCOLATION TESTS

Tests were made using single mechanical packer.

Depth From (P, C, or Cm)	Depth To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)
P 4.0	40.0	6.5	8.5	20
P 7.0	40.0	6.5	8.5	20
P 39.6	74.6	0.25	8.5	10
P 74.2	109.2	0.34	8.5	20

100

Elevation (Feet)	Depth (Feet)
5294.3	5.4
5281.7	20

GRAPHIC LOG

SAMPLES FOR TESTING

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-5.4': CONCRETE; hard, 1" rebar at 4.4'. Contact with underlying rock is irregular and tight.

5.4'-20'±: WELDED ASH-FLOW TUFF; light to medium gray, fairly hard (requires light hammer blow to break), light weight, slightly porphyritic with phenocrysts of white feldspar mostly up to 1/8". Moderately Jointed. Joints are from 1" to 1' apart averaging about 0.8' apart. Most joints are about 60° to core axis and stained with iron and carbonate minerals. Foliation is indistinct.
Light gray grout is present in the following joints:
9.3': Soft "chalky" grout 1/16" to 1/8" in planar joint about 60° to core axis.
10.8': Scattered grout stain in planar joint about 60° to core axis.
16.4': Trace of grout in calcite filled joint (1/32" thick) about 30° to core axis.

20'±-109.2': WELDED ASH-FLOW TUFF; fairly hard (requires light to moderate hammer blow to break), slightly porphyritic with white feldspar phenocrysts up to 1/4". Lightly to Moderately Jointed. Average joint spacing is about 0.8'. Most joints are planar and cross the core at an angle of about 60° with a few prominent joints crossing the core at an angle of about 30°. Many joints are stained with iron and manganese oxides and some are filled with calcium carbonate. Foliation is faint to distinct and is caused by flattened, wavy pumice fragments up to 1/4" wide and some flattened cavities or vugs which cross the core at an angle of 60° to 80°. Color is variable from a light purple gray to pinkish gray.
67.9'-69.9': Breccia zone and a interflow zone.
67.9'-68.8': Flow breccia, dark gray welded tuff fragments up to 2" across cemented fairly tightly with silt and calcium carbonate.
69.3'-69.9': Interflow of medium to coarse, semi-rounded sand (mostly quartz) cemented with calcium carbonate.
Grout is present in the following joints:

EXPLANATION

CORE LOSS

CORE RECOVERY

*LOCATION: Sta. 10+60.4 offset 8.5' u/s from ℄ grout cap.

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx - 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-601 LOCATION Right Spillway Bay* COORDS. N. E. GROUND ELEV. 5299.0 DIP (ANGLE FROM HORIZ.) 60°

BEGUN 9-8-76 FINISHED 9-11-76 DEPTH OF OVERBURDEN 0.0 TOTAL DEPTH 109.2 BEARING N74°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
Hole Completion Cemented hole back to surface. Purpose of Hole Determine permeability of rock within grout curtain beneath spillway crest.		100 100 100	0.0	109.2	2.6	Gravity	5	5204.4	109.2			See below

Depth scale: 110, 120, 130, 140, 150, 160, 170, 180, 190

31.8': Scattered chalky grout up to 1/32" thick in broken zone which appears to be at intersection of 2 joints.
33.2': Hard grout 1/8" thick cementing planar, smooth joint which is about 60° to core axis.
40.0'-40.7': Grout partially filling 1/8" to 1/16" rough joint which is 80° to 90° to core axis.
43.4' & 43.6': Thin grout coating on two smooth, planar joints about 60° to core axis.
44.2': Grout 1/16" thick cementing joint which is about 60° to core axis.
44.8': Grout stain in smooth, planar, carbonate stained joint about 60° to core axis.
45.9'-46.2': Lens of grout 3/8" thick along side of core.
50.0' & 50.2': Grout 1/4" thick cementing smooth, planar joint about 85° to core axis.
50.3' & 50.9': Blobs of grout up to 1/4" thick partially filling a 1" wide zone of vesicular welded tuff about normal to core axis.
52.2'-54.7': Grout, partly chalky, up to 1/4" thick cementing a planar, rough joint which is nearly parallel to the core.
54.7': Grout 1/8" to 3/8" thick in irregular, rough joint 60°-80° to core axis.
59.2': Scattered blobs of grout in very irregular, rough fracture about 30° to core axis.
59.4'-59.7': Grout 1/16" wide cementing planar, rough joint about 30° to core axis. Joint terminates in vuggy zone which contains grout up to 3/8" wide about 80° to core axis.
61.7': Grout up to 3/8" thick partially filling smooth, planar joint about 60° to core axis. Part of joint is filled with brown fine sand.
62.3'-63.5': Grout 1/8" thick filling a planar, slightly rough joint.
67.9'-68.3': Grout up to 1/4" in irregular zone lined with calcite which is 70° to 80° to core axis.
68.9': Scattered small blebs (1/8" thick) of grout in a silty breccia zone.

EXPLANATION

*LOCATION: Sta. 10+60.4 offset 8.5' u/s from ℄ grout cap.

CORE LOSS

CORE RECOVERY

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-601 LOCATION Right Spillway Bay* COORDS. N. E. GROUND ELEV. 5299.0 DIP (ANGLE FROM HORIZ.) 60°
BEGUN 9-8-76 FINISHED 9-11-76 DEPTH OF OVERBURDEN 0.0 TOTAL DEPTH 109.2 BEARING N74°W
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes** LOGGED BY D. N. Magleby LOG REVIEWED BY R. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
												70.2' & 70.9': Paper-thin grout partially filling a smooth joint about 60° to core axis. Joint is also filled with $CaCO_3$ up to 1/32" and is iron and manganese stained. 76.4': Chalky grout stain in smooth, planar limonite stained joint about 60° to core axis. 77.0': Chalky grout up to 1/8" thick in smooth, planar joint about 30° to core axis. 77.4'-78.2': Irregular filling of grout 3/4" to 1" thick in joint about 60° to core axis. Part of joint is filled with compacted brown, calcareous silty sand. 78.2'-109.2': No grout in core.

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90

CORE LOSS
CORE RECOVERY

EXPLANATION

*LOCATION: Sta. 10+60.4 offset 8.5' u/s from ℄ grout cap.

Type of hole . . . . . O = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 3 OF 3 HOLE NO. DH-601

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 2

FEATURE: Teton Dam — PROJECT: Teton Basin — STATE: Idaho

HOLE NO.: DH-602 — LOCATION: Right Spillway Bay* — COORDS. N. ... E. ... — GROUND ELEV.: 5299.0 — DIP (ANGLE FROM HORIZ.): 90°

BEGUN: 9/4/76 — FINISHED: 9/8/76 — DEPTH OF OVERBURDEN: 0.0 — TOTAL DEPTH: 94.7' — BEARING: --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED: See notes ** — LOGGED BY: D. N. Magleby — LOG REVIEWED BY: B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment
Longyear "34" drill skid mounted.

Driller
Frank Martin

Drilling Methods
Drilled with clear water using NWD-3 split-tube core barrel.

Drilling Conditions
0.0'-44.2'-Fast and smooth.
44.2'-58.4'-Slow.
58.4'-94.7'-Fast and smooth.

Water Return
0.0'-34.7': 100%
34.7'-55.4': 99%
55.4'-55.6': 0%
55.6'-94.7': 90%

Water Levels**
During Drilling

Date	Depth Hole	Depth Water
9/7	34.7'	33.3'
9/8	78.7'	59.4'

Hole Completion
Cemented hole back to surface.

Purpose of Hole
Determine permeability of rock within grout curtain beneath spillway crest.

TYPE AND SIZE OF HOLE

0 Nx

CORE RECOVERY (%)

100 throughout (0–94.7 ft)

PERCOLATION TESTS

Tests were made using single mechanical packer

Depth From (P, Ca, or Cm)	Depth To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)
4.7 P	34.7	2.92	8	10
5.7 P	34.7	1.07	8	10
34.7 P	64.7	2.1	8	10
64.7 P	94.7	0.38	8.5	10
0	94.7	4.5	gravity	3

Elevation (Feet)	Depth (Feet)
5294.3	4.7
5282.0	17.0
5204.3	94.7

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-4.7': CONCRETE. Tight contact with underlying rock, contact dips about 10°

4.7'-17'±: WELDED ASH FLOW TUFF. Fairly hard (requires light hammer blow to break), light weight, slightly porphyritic (white phenocrysts of feldspar up to 1/8"). Moderately Jointed. Joint spacing varies from 0.3' to 1.2' and averages about 0.8'. Most joints are nearly flat lying, some of which are coated with silt and stained with manganese oxides. Foliation is indistinct. Color varies from light gray to 11 feet then changing to a pinkish gray.

Grout is present in the following joints:

- 4.7'-7.9': Grout to 1/8" thick in near vertical joint, some grout stained with red-purple dye.
- 7.9'-8.1': Irregular blob of grout 1/8" thick surrounding fragment of lapilli. Most grout is stained with red-purple dye.
- 8.3': Wavy, slightly rough joint coated with 1/16" thick dye-stained grout. Joint dips about 20°.
- 8.7': Dye stained grout about 1/32" thick in irregular joint dipping about 75°.
- 9.5': Scattered grout up to 1/32" thick in broken zone.
- 10.5': Grout 1/32" thick in flat lying joint.
- 11.5'-13.1': Near vertical joint filled with 1/32" thick grout.
- 13.1': Same as 10.5' except joint dips about 10°.

17'±-94.7': WELDED ASH FLOW TUFF. Fairly hard (requires moderate hammer blow to break), slightly porphyritic (white feldspar phenocrysts up to 1/4"). Intensely to Lightly Jointed, as described below. Most joints are nearly flat lying and many are stained with iron and manganese oxides and calcium carbonate. Low dipping flow lineations caused mainly by flattened wavy pumice fragments and flattened vesicles are distinct in some sections of core but indistinct in other sections. Color is pinkish gray from 17.0' to 30.6', changing to light purple-gray from 30.6' to 94.7'.

71.0'-93.0': Faint indication of vertical flow lineation.

EXPLANATION

* Location: Sta. 10+63.4, offset 8.5' u/s from ℄ grout curtain.

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, N = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Ca = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE: Teton Dam — PROJECT: Teton Basin — STATE: Idaho — SHEET 1 OF 2 — HOLE NO. DH-602

GPO 854-925

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-602 LOCATION Right Spillway Bay* COORDS. N. E. GROUND ELEV. 5299.0 DIP (ANGLE FROM HORIZ.) 90°
BEGUN 9/4/76 FINISHED 9/8/76 DEPTH OF OVERBURDEN 0.0 TOTAL DEPTH 94.7' BEARING --
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
												17.0'-30.8': Intensely Jointed. Joints range from 0.1' to 0.5', mostly 0.2'. 30.8'-94.7': Moderately to Lightly Jointed. Joints range from 0.1 to 5.7', mostly 1.8' Grout was found in the following joints: 24.3': Grout filling 1/2" thick, irregular, rough, flat lying joint. 24.6': Trace of grout in flat lying joint. 26.6': Scattered chalky grout up to 1/16" thick in irregular, rough, flat lying joint partially filled with calcium carbonate up to 1/4" thick. 30.4'-30.5': Grout 1/32" thick in flat lying vesicular zone. 59.9': Grout 1/8" thick cementing smooth, planar joint, dips 75-80°. 77.9': Irregular occurrence of grout up to 1/4" thick partially filling rough and irregular crack which appears to be open 1/4" to 1/2" and dips about 80°. 92.0'-92.8': Scattered grout 1/32" thick in smooth, wavy joint which dips nearly 90° - terminates on 45° dipping joint.

(Depth scales 10, 20, 30, 40, 50, 60, 70, 80, 90)

EXPLANATION

*LOCATION: Sta. 10+63.4, offset 8.5' u/s from ℄ grout curtain.

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1887 (8-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 2

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-603 LOCATION Right Spillway Bay* GROUND ELEV. 5299.0' DIP (ANGLE FROM HORIZ.) 60°
COORDS. N. .... E. ....
BEGUN 9-11-76 FINISHED 9-15-76 DEPTH OF OVERBURDEN 0.0 TOTAL DEPTH 108.7' BEARING S74°E
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes** LOGGED BY D. N. Magleby LOG REVIEWED BY R. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment
Skid mounted Longyear "34" drill.

Driller
Frank Martin

Drilling Methods
Drilled with clear water using NWD-3 split tube core barrel.

Drilling Conditions
0.0'-57.9': Fast and smooth.
57.9'-80.0': Slow.
80.0'-81.2': Fast.
81.2'-108.7': Fairly slow.

Water Return
0.0'-22.5': 100%
22.5'-57.9': 95%
57.9'-80.0': 90%
80.0'-98.7': 80%
Lost water at 98.7' for 4 minutes, regained 75%.
98.7'-108.7': 75%

Water Level During Drilling **

Date	Depth Hole	Depth Water
9/13	3.6'	2.0'
9/14	57.9'	46.0'
9/15	98.6'	65.1'

Hole Completion
Hole was cemented back to surface.

Purpose of Hole
Determine permeability of rock within grout curtain beneath spillway crest.

TYPE AND SIZE OF HOLE

D Nx

CORE RECOVERY (%)

95, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100

PERCOLATION TESTS

Tests were made using single mechanical packer

	Depth (feet) From (P, Cs, or Cm)	Depth (feet) To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)
P	4.5'	39.5'	1.0	8.5	20
P	40.1'	75.1'	0.08	8.5	20
P	73.7'	108.7'	0.4	8.5	20
	0.0'	108.7'	2.3	Gravity	5

ELEVATION (FEET) / DEPTH (FEET)

Elevation	Depth
5294.6	5.1
5279.5	22.5

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-5.1': CONCRETE. Good tight bond with underlying rock, 60° to core axis, contact is planar and fairly smooth.

5.1'-22.5': WELDED ASH-FLOW TUFF; fairly hard (requires light hammer blow to break), light weight, slightly porphyritic (phenocrysts of white feldspar 1/8" to 1/4"), contains some scattered vesicular zones. Lightly jointed, most joints are from 8" to 2' apart and most are about 60° to core axis. (Strong joint at 15.4' is about 30° to core axis). Joints at 19.2' and 19.6' have calcite coatings up to 1/64" thick. Foliation is indistinct. Color is light gray.
Grout is present in the following joints:
- 5.4': Soft chalky grout 1/32" thick partly filling a joint which is about 90° to core axis.
- 7.4': Hard grout 1/8" thick filling and cementing a smooth planar joint about 60° to core axis.
- 11.2': Hard grout 1/4" thick nearly filling a planar, smooth joint about 60° to core axis.
- 13.1': Scattered soft chalky grout up to 1/8" thick partially filling irregular, rough, iron stained joint which is about 60° to core axis.
- 14.3': Trace of chalky grout in irregular joint which is about 60° to core axis.

22.5'-108.7': WELDED ASH-FLOW TUFF; fairly hard (requires moderate hammer blow to break), slightly porphyritic (white phenocrysts of feldspar from 1/4" to 1/2"), scattered lapilli up to 1" across. Quartz lined cavity 1/2" across occurs at 55.5'. From 22.5' to 39'± the foliation is very distinct due to flattened vesicles and wavy flattened pumice which cross the core at an angle of about 60°. The joints which are from 1" to 3" apart parallel the foliation. From 39'± to 84.5'± the foliation is very faint. From 84.5'± to 108.7' the foliation is indistinct. Color is light purple gray with scattered streaks of red from 22.5' to 63'± and medium gray from 63'± to 108.7'.

EXPLANATION

CORE LOSS / CORE RECOVERY

*LOCATION: Sta. 10+66.4, offset 8.5' u/s ℄ grout cap

Type of hole . . . D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 2 HOLE NO. DH-603

GPO 854-625

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 2 OF 2

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-603 LOCATION Right Spillway Bay* GROUND ELEV 5299.0' DIP (ANGLE FROM HORIZ.) 60°

COORDS. N. E.

BEGUN 9-11-76 FINISHED 9-15-76 DEPTH OF OVERBURDEN 0.0 TOTAL DEPTH 108.7' BEARING S74°E

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See notes** LOGGED BY R. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
		100										Grout was found in the following joints:
		100										
		100						5204.9	108.7			

Grout was found in the following joints:

37.4': Grout up to 1/8" thick in planar, slightly rough joint about parallel to the foliation which is about 30° to core axis.

38.3': Same as 37.4'.

43.3': Grout to 1/8" thick filling and cementing smooth joint.

49.0': Grout to 1/8" thick filling smooth joint about 50° to core axis. (Joint is about normal to faint foliation).

56.9': Slightly chalky grout 1/8" to 1/4" thick filling smooth, planar joint.

63.9'-64.7': Scattered grout up to 1/16" thick present along a few surfaces in a 1-1/2" wide broken zone of rock which is partially cemented with calcite up to 3/8" thick.

65.8'-69.3': Grout 1/2" to 3/4" thick filling and cementing a planar smooth joint which is nearly parallel to core axis.

74.7': Chalky grout up to 1/8" partially filling planar, smooth, calcium carbonate coated joint about 60° to core axis.

88.4'-88.8': Grout 1/4" to 1" nearly completely filling and cementing a brecciated zone. Silt occurs along some surfaces.

91.0'-91.2': Wedge shaped occurrence of grout about 1" thick at intersection of 2 joints. One joint is irregular, rough, near normal to core; other joint is irregular, rough and about 30° to core axis.

102.5': Scattered, paper thin chalky grout in silt coated planar, rough joint about 60° to core axis.

110, 20, 30, 40, 50, 60, 70, 80, 90

CORE LOSS

CORE RECOVERY

EXPLANATION

* LOCATION: Sta. 10+66.4, offset 8.5' u/s ℄ grout cap.

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) .. Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton PROJECT Teton Basin STATE Idaho SHEET 2 OF 2 HOLE NO. DH-603

GPO 854-625

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-604 LOCATION Middle Spillway Bay* COORDS. N. E. GROUND ELEV. 5299.0 DIP (ANGLE FROM HORIZ.) 60°

BEGUN 9/18/76 FINISHED 9/21/76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 109.7' BEARING N74°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	CLASSIFICATION AND PHYSICAL CONDITION
Drill Equipment Skid mounted Longyear "34" Drill. Driller Frank Martin Drilling Methods Drilled with clear water using NWD-3 split tube core barrel. Drilling Conditions 0.0'-109.7': Fast and smooth. Water Return 0.0'-31.9': 100% 31.9'-109.7': 95% Water Level During Drilling** Date / Depth Hole / Depth Water 9-20 / 31.9 / 28.8 9-21 / 84.6 / 58.1 Hole Completion Hole was cemented back to surface. Purpose of Hole Test permeability of rock within grout curtain beneath spillway crest.	D Nx	100 100 100 100 100 100 100 100 100 100 100 100 100 100 100 100 100 100 100 100 100 100 100 100	Tests were made using single mechanical packer. P 5.5 P 40.1 P 74.7	40.5 75.1 109.7	0.77 0.5 0.13	10 10 10	20 20 20	5293.8 5280.0 5259.2	6.0 22.0 46.0	0.0'-6.0': CONCRETE. Contact with underlying rock is smooth and uncemented. 6.0'-22.0': WELDED ASH-FLOW TUFF; light gray, fairly hard (requires light hammer blow to break), light weight, slightly porphyritic with phenocrysts of white feldspar up to 1/4", scattered lapilli up to 3" across, Lightly to Moderately Jointed. Joints are 0.3' to 2.0' apart averaging about 1.0', most joints are smooth and planar and about 60° to core axis. From 15.4' to 16.4' is a calcite stained, fairly smooth, planar joint about 15° to core axis. Foliation is indistinct. Grout was found in the following joints: 7.8'-8.3': Hard grout 1/8" thick filling planar, rough joint about 15° to core axis. Terminates at 8.3' with smooth, planar joint about 35° with grout filling core up to 1/2". 8.6': Grout stain in very irregular, rough joint about 80° to core axis. 8.8'-9.0': Hard grout 3/8" thick in smooth, planar joint about 15° to core axis. Terminates at 9.0' with smooth planar joint about 45° to core axis with 1/4" grout. 10.0': Grout 1/4" thick filling a fairly smooth, planar, silt and $CaCO_3$ stained joint about 45° to core axis. 10.5': Grout stain in planar, rough joint about 60° to core axis. 11.5': Hard grout up to 1/4" in a planar, fairly smooth joint which is 60° to core axis. 12.4': Grout stain in rough, irregular joint which is about 60° to core axis. 22.0'-46'±: WELDED ASH-FLOW TUFF; fairly hard (requires light to moderate hammer blow to break), slightly porphyritic. Moderately to Intensely Jointed. The joint spacing varies from 0.1' to 1.0', averaging about 0.3', most joints are hematite stained. Most joints cross the core at an angle of 60° which is about parallel to the foliation. Color is reddish gray from 22.0' to 39'± and medium gray from 39'± to 109.7'.

EXPLANATION

CORE LOSS / CORE RECOVERY

*LOCATION: Sta. 10+86, offset 8.5' u/s from ℄ grout cap.

Type of hole . . . D = Diamond, N = Haystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 2 OF 2

FEATURE Tetoп Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-604 LOCATION Middle Spillway Bay* GROUND ELEV. 5299.0 DIP (ANGLE FROM HORIZ.) 60°

COORDS. N. E.

BEGUN 9-18-76 FINISHED 9-21-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 109.7' BEARING N74°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	D Nx	100										Foliation caused by flattened streaky pumice fragments and some flattened vesicles about 60° to the core axis, is prominent from 22'± to 32'± and faint from 32'± to 46'±.
		100										Grout was found in the following openings:
		100						5204.0	109.7			26.3': Irregular, vesicular zone about 80° to core axis containing grout up to 1/4" thick.
												32.2': Grout stain in rough planar joint about 60° to core axis, (about opposite to flow lineation).
												33.1': Intersection of 2 irregular rough joints, 45° and 60° to core axis, containing grout up to 3/8" thick.
												44.2': Grout stain in irregular rough fracture about normal to core axis.
												46'±-109.7': WELDED ASH-FLOW TUFF; fairly hard (requires moderate hammer blow to break), slightly porphyritic. Lightly Jointed. Joint spacing varies from 0.5' to 4.0' and averages about 2.0'. Most joints are 60° to the core axis. Prominent calcite filled (1/8") joints at 54' and 62' are about 30° to core axis. Vesicular zones are at 50.0' to 50.1' and 62.3' to 62.5'. Foliation is faint to indistinct.
												Grout occurs in the following openings:
												79.1': Planar, smooth joint 30° to core axis filled with 1/32" thick grout.
												81.4'-83.0': Irregular filling of grout from 1/4" to 1" thick, filling and cementing a broken zone in the core, terminates on a smooth planar joint 30° to core axis.
												95.5': Hard grout up to 1/4" thick in a rough, irregular joint about normal to core axis.
												95.6'-97.3': Grout 1/8" to 1/4" thick in a smooth, planar, iron stained joint about 10° to core axis.
												108.0'-109.0': Grout 1/4" thick in smooth, planar, iron stained joint about 10°-15° to core axis.

Depth scale: 110, 20, 30, 40, 50, 60, 70, 80, 90

EXPLANATION

* LOCATION: Sta. 10+86, offset 8.5' u/s from ℄ grout cap.

CORE LOSS / CORE RECOVERY

Type of hole . . . D = Diamond, N = Haystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 2 OF 2 HOLE NO. DH-604

GPO 854-625

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 3

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-605 LOCATION Middle Spillway Bay* GROUND ELEV. 5299.0 DIP (ANGLE FROM HORIZ.) 90°
COORDS. N. E.
BEGUN 9-16-76 FINISHED 9-18-76 DEPTH OF OVERBURDEN 0.0 TOTAL DEPTH 96.0' BEARING --
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment
Skid mounted Long-year "34" drill.
Driller
Frank Martin
Drilling Conditions
0.0'-45.3': Fairly fast and smooth.
45.3'-96.0': Fairly slow and smooth.
Water Return
0.0'-18.0': 100%
18.0'-96.0': 95%
**Water Level During Drilling

Date	Depth Hole	Depth Water
9-17	45.3	30.3
9-18	88.9	48.6

Hole Completion
Cemented hole back to surface.
Purpose of Hole
Determine permeability of rock within grout curtain beneath spillway crest.

TYPE AND SIZE OF HOLE: D Nx

CORE RECOVERY (%): 100 in all runs (0.0'-96.0').

PERCOLATION TESTS

Tests were made using single mechanical packer.

From (P, Cs, or Cm)	To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)
P 6.0'	36.0'	0.2	10	20
P 36.0'	66.0'	0.2	10	20
P 66.0	96.0	0.03	10	20

ELEVATION / DEPTH (FEET)

Elevation	Depth
5293.2	5.8
5277.5	21.5
5261.0	38.0±
5203.0	96.0

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-5.8': CONCRETE. Poor bond with underlying rock along a smooth, wavy contact.

5.8'-21.5': WELDED ASH-FLOW TUFF: fairly hard,(light hammer blow will break), light weight, slightly porphyritic, scattered lapilli up to 3/8" across, indistinct foliation. Moderately Jointed. Joint spacing varies from 0.4' to 2.2', averages about 1.0'. Most joints are smooth and planar, lightly calcite stained and are 70° to 80° to the core axis. Color is light gray 5.8' to 16'± and light reddish gray 16'± to 21.5'.
Grout was found in the following openings:
- 10.8': Grout 1/8" thick in planar rough joint about 70° to core axis.
- 13.8'-13.9': Scattered grout stain in broken zone of core.
- 14.0': Grout 1/16" thick in planar, smooth joint about 40° to core axis.
- 14.8': Planar, smooth joint about 15° to core axis filled with 1/4" thick grout.
- 15.2': Scattered grout stain in smooth, wavy joint about normal to core axis.
- 15.2'-15.4': Grout up to 1/8" thick in smooth, planar joint about parallel to core.
- 17.2': Grout up to 1/4" (partly chalky) in planar, smooth joint about 40° to core axis.
- 18.7': Trace of grout in smooth, wavy joint about 15° to core axis.
- 19.9': Trace of grout in smooth, wavy joint about 15° to core axis(not parallel to joint at 18.7').
- 20.8': Planar, slightly rough joint about 75° to core axis filled with 1/4" grout.

21.5'-38'±: WELDED ASH-FLOW TUFF; fairly hard, (requires moderate hammer blow to break), slightly porphyritic. Intensely Jointed. Joint spacing is from 0.1' to 1.0' averaging about 0.4'. Most joints are about 80° to core axis and are parallel to the foliation. Distinct foliation caused by flattened pumice fragments and flattened vesicles cross the core at an angle of about 80°. Color is light reddish gray from 21.5' to 27'± and medium gray from 27'± to 38'±.

EXPLANATION

*LOCATION: Sta. 10+89, offset 8.5' u/s ℄ grout cap.

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 2 OF 3

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-605 LOCATION Middle Spillway Bay* COORDS. N. E. GROUND ELEV 5299.0 DIP (ANGLE FROM HORIZ.) 90°
BEGUN 9-16-76 FINISHED 9-18-76 DEPTH OF OVERBURDEN 0.0 TOTAL DEPTH 96.0 BEARING --
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	10, 20, 30, 40, 50, 60, 70, 80, 90								10, 20, 30, 40, 50, 60, 70, 80, 90			Grout was found in the following openings: 21.7': Grout stain in 1/8" wide joint partially filled with calcite, joint is about 80° to core axis. 23.8': Grout and silt stain in irregular, rough joint about 45° to core axis. 24.0': Grout stain in planar, smooth, silt stained joint about 80° to core axis. 24.4': Chalky grout up to 1/8" thick in irregular, rough joint about 80° to core axis. 24.8': Grout up to 1/8" thick in rough, irregular joint which is 30° to 40° to core axis. 30.0': Chalky grout up to 1/8" thick in irregular to planar, smooth joint about normal to core axis. 33.3': Scattered grout up to 1/32" thick in smooth offset joint about 20° to core axis. 33.6': Scattered grout to 1/32" thick in smooth, planar joint about parallel to core. 37.2': Grout up to 1/8" in irregular rough joint about normal to core axis. 38'±-96.0': WELDED ASH-FLOW TUFF; medium gray with hematite staining in some joints. Fairly hard, (requires moderate hammer blow to break), slightly porphyritic. Contains scattered pumice and lapilli fragments (phenocrysts and pumice fragments give the rock a "speckled" appearance). <u>Lightly Jointed</u>. Joint spacing is from 0.4' to 4.8' and averages about 2.0'. Most joints are planar and smooth and cross the core at an angle of about 80°, calcite staining is prevalent in a few joints. Foliation is indistinct to faint and is from 30° to 80° to core axis. Grout was found in the following openings: 46.9'-47.0': Core contains vesicles from less than 1/32" to 3/8" in size. 52.4'-52.6': Same as interval 46.9' to 47.0'. 58.6'-58.8': Same as interval 46.9' to 47.0'. 60.9': Trace of grout in smooth, planar iron stained joint about 80° to core axis.

*LOCATION; Sta. 10+89, offset 8.5' u/s ℄ grout cap.

EXPLANATION

CORE LOSS
CORE RECOVERY

Type of hole .... D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series).. Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-605 LOCATION ~~Middle Spillway Bay~~* GROUND ELEV. 5299.0 DIP (ANGLE FROM HORIZ.) 90°
COORDS. N. E.
BEGUN 9-16-76 FINISHED 9-18-76 DEPTH OF OVERBURDEN 0.0 TOTAL DEPTH 96.0' BEARING --
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
												67.4'-68.4': Grout 1/8" thick filling a smooth, planar joint about 8° to core axis. 73.3': Grout 1/32" thick in smooth, wavy joint from 20° to 45° to core axis. 81.4'-84.2': Smooth, wavy joint about parallel to core axis containing scattered grout stain. 92.3': Grout 1/8" thick in planar, fairly smooth joint about 25° to core axis. 93.4': Irregular grout filling about 1/64" thick in a vesicular zone, about 80° to core axis.

EXPLANATION

*LOCATION: Sta. 10+89, offset 8.5' u/s ℄ grout cap.

CORE LOSS
CORE RECOVERY

Type of hole . . . D = Diamond, N = Noystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-606 LOCATION Middle Spillway Bay* COORDS. N. E. GROUND ELEV. 5299.0' DIP (ANGLE FROM HORIZ.) 60°

BEGUN 9-21-76 FINISHED 9-22-76 DEPTH OF OVERBURDEN 0.0 TOTAL DEPTH 41.1' BEARING S74°E

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment

Skid mounted Longyear "34" drill.

Driller

Frank Martin

Drilling Conditions

0.0'-41.1': Fairly fast and smooth.

Water Return

0.0'-19.0': 100%
19.0'-41.1': 95%

Water Level** During Drilling

Not measured.

Hole Completion

At 41.1' shell stripped out of core barrel leaving bit and shell in hole. Moved off hole--cemented hole and moved 0.7' d/s to DH-606A.

Purpose of Hole

Determine permeability of rock within grout curtain beneath spillway crest.

TYPE AND SIZE OF HOLE / CORE RECOVERY

Type and size of hole: D Nx

Core recovery (%)
96
100
100
100
100
100
100
100
100

PERCOLATION TESTS

Tests were made using single mechanical packer.

Depth (feet) From (P, Cs, or Cm)	To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of test (min.)
P 5.6	40.6	1.1	10	10

ELEVATION (FEET) / DEPTH (FEET)

Elevation (feet)	Depth (feet)
5292.8	7.1
5278.2	24.0
5263.4	41.1

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-7.1': CONCRETE. Poor bond with underlying rock in rough, irregular broken contact.

7.1'-24.0': WELDED ASH-FLOW TUFF; light gray to 17'±, medium gray streaked and blotched with hematite red from 17'± to 39'±. Fairly hard (requires light hammer blow to break), light weight, slightly porphyritic, scattered lapilli up to 1-1/2" across. Lightly to Moderately Jointed. Joint spacing varies from 0.2' to 4.2', averaging 1.3'. Most joints cross the core at angle of 60° to 80° to core axis. Foliation is indistinct to faint. (Faint foliation from 19'-23' is caused by flattened streaky pumice that crosses the core in 1/4" wide bands about 60°-90° to core axis.)
Grout was found in the following openings:
- 7.3': Scattered grout stain in irregular, rough joint about 80° to core axis.
- 7.5': Grout 1/64" thick in smooth, planar joint about 80° to core axis.
- 12.7': Scattered grout stain in rough, irregular joint about 80° to core axis.
- 17.1'-17.8': Very rough irregular hematite stained fracture about 10° to core axis containing scattered grout stain.
- 18.1': Hard grout 1" thick filling smooth, planar, calcite stained joint which is about 45° to core axis.

24.0'-41.1': WELDED ASH-FLOW TUFF; fairly hard, (requires light to moderate hammer blow to break), light weight, slightly porphyritic. Moderately to Intensely Jointed. Joint spacing varies from 0.1' to 0.8', averaging 0.2'. Most joints are stained with calcite (some silt staining), and are parallel to the foliation. Color is medium gray, streaked and blotched with hematite red. Distinct foliation caused by flattened vesicles and flattened, wavy pumice fragments, cross the core at an angle of about 50° to 60°.
Grout was found in the following openings:
- 38.9'-39.0': Vesicular zone about 80° to core axis containing scattered grout up to 1/8" thick.

EXPLANATION

CORE LOSS / CORE RECOVERY

* LOCATION: Sta. 10+92, offset 8.5' u/s ℄ grout cap.

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) .. Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 2

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-606-A LOCATION Middle Spillway Bay* GROUND ELEV 5299.0' DIP (ANGLE FROM HORIZ.) 60°

COORDS. N. .... E. ....

BEGUN 9/22/76 FINISHED 9/24/76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 111.0' BEARING S74°E

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment
Skid mounted Longyear "34" drill.

Driller
Frank Martin

Drilling Methods
Advanced hole with NWD-3 split tube core barrel using clear water.

Drilling Conditions
0.0'-69.2': Fairly fast and smooth.
69.2'-111.0': Fairly slow and smooth.

Water Return
0.0'-13.0': 100%
13.0'-43.3': 95%
43.3'-69.2': 85%
69.2'-111.0': 90%

Water Level During Drilling**

Date	Depth Hole	Depth Water
9/23	13.0	0.0
9/24	69.2	40.6

Hole Completion
Cemented hole back to surface.

Purpose of Hole
Determine permeability of rock within grout curtain beneath spillway crest.

TYPE AND SIZE OF HOLE

D Nx

CORE RECOVERY (%)

100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 96, 100, 100, 90, 100, 100, 100, 100, 100

PERCOLATION TESTS

Tests were made using single mechanical packer.

Depth (feet) From (P, Cs, or Cm)	To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)
P 6.8'	41.8	0.35	10	20
P 41.4	76.4	0.64	10	20

ELEVATION (FEET) / DEPTH (FEET)

Elevation (feet)	Depth (feet)
5292.9	7.1
5278.0	24.2
5262.8	41.8

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-7.1': CONCRETE. Poor bond with underlying rock, silt staining on rock surface. 1/2" rebar at 1.3' and 4.2'.

7.1'-24.2': WELDED ASH-FLOW TUFF; fairly hard (requires light hammer blow to break), light weight, slightly porphyritic, scattered lapilli fragments up to 3" across. Most joints are 30° and 60° to core axis and some are stained with $CaCO_3$ and hematite. Lightly Jointed. Joint spacing varies from 0.3' to 3.0' averaging 1.5'. Indistinct foliation. Color is light gray from 7.1' to 16'± and then light to medium gray streaked and stained mostly in joints and vesicular zones with hematite red from 16'± to 40'±.

Grout was found in the following openings:

- 7.5': Thin coating of grout in rough planar joint about normal to core axis.
- 8.2': Grout up to 1/32" in rough planar joint about 60° to core axis.
- 10.2': Grout stain in planar, smooth joint about 60° to core axis. Joint is partially filled with $CaCO_3$ up to 1/16" thick.
- 12.6': Thin soft chalky grout in planar, fairly smooth joint about 60° to core axis.
- 16.0': Grout stain in rough planar calcite stained joint 60° to core axis.
- 17.7': Grout 3/4" thick in 1/4" thick calcite lined joint which is slightly rough and wavy and is about 35° to core axis.

24.2'-41.8'±: WELDED ASH-FLOW TUFF; fairly hard (requires moderate hammer blow to break), light weight, slightly porphyritic. Moderately to Intensely Jointed. Joint spacing varies from 0.1' to 1.0' and averages 0.3'. Core has distinct foliation that crosses the core at an angle of 50° to 60°. Most jointing parallels the foliation. Color is medium gray streaked with hematite red.

EXPLANATION

*LOCATION: Sta. 10+92, offset 7.8' u/s ℄ grout cap.

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-606-A LOCATION Middle Spillway Bay* COORDS. N. E. GROUND ELEV. 5299.0' DIP (ANGLE FROM HORIZ.) 60°

BEGUN 9/22/76 FINISHED 9/24/76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 111.0' BEARING S74°E

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
		100										
		100										
	110	100						5202.9	111			

Grout was found in the following openings:
31.2': Soft grout stain in rough, broken iron stained vesicular zone about 0.2' wide.
38.2': Scattered hard grout up to 1/8" thick in 0.2' wide vesicular zone about 30° to core axis.

41.8'±-111.0': WELDED ASH-FLOW TUFF; fairly hard, light weight, slightly porphyritic, scattered lapilli up to 3/4" across. Moderately to Intensely Jointed, as described below. Majority of joints are smooth, planar, iron, manganese and calcite stained and from 45° to 60° to core axis. Foliation is faint to indistinct. Color is medium to dark gray.
41.8'-51.0': Intensely Jointed. Joint spacing is 0.1' to 1.4', averaging 0.3'.
51.0'-111.0': Moderately Jointed. Joint spacing is 0.2' to 3.0', averaging 1.0'.
Grout was found in the following openings:
50.0'-50.6': Grout from 1/4" to 3/8" thick in smooth, planar joint about 20° to core axis. $CaCO_3$ up to 5/8" thick fills portions of the joint.
55.9'-58.0': Joint nearly parallel to core open to 1/4" partially filled with grout and calcite.
62.6': Irregular blob of grout up to 1/2" thick filling an irregular 0.2' wide vesicular zone.
64.6': Grout 1/8" thick filling iron stained smooth planar joint about 35° to core axis.
65.4': Grout, partially chalky, up to 1/16" in a planar, fairly smooth joint about 30° to core axis (this joint is not parallel to joint at 64.6').
68.5': Scattered grout stain in 0.2' vesicular zone about 45° to core axis.
91.7': Scattered grout stain in smooth, planar, iron and manganese stained joint which is 30° to core axis.
92.1': Scattered grout stain in smooth, planar joint about 60° to core axis. Joint is iron and calcite stained.
95.4': Grout 1/16" thick partially filling smooth, planar, iron stained joint about 60° to core axis.
99.3': Grout 1/32" thick in smooth, planar joint about 40° to core axis. Joint is iron and manganese stained.

EXPLANATION

CORE LOSS
CORE RECOVERY

* LOCATION: Sta. 10+92, offset 7.8 u/s ℄ grout cap.

Type of hole . . . D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-607 LOCATION Left Spillway Bay* GROUND ELEV. 5299.0' DIP (ANGLE FROM HORIZ.) 60°

COORDS. N. E.

BEGUN 9/29/76 FINISHED 9/29/76 DEPTH OF OVERBURDEN 0.0 TOTAL DEPTH 8.4' BEARING N74°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED No water levels taken LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
Drill Equipment Skid mounted Long-year "34" drill. Driller A. Allen Drilling Method Drilled with clear water using NWD-3 split tube core barrel. Water Return 0.0'-8.4':100% Hole Completion Twisted off bit at 8.4', cemented hole, moved 0.7' d/s to new hole location (DH-607A). Purpose of Hole Determine permeability of rock within grout curtain beneath spillway crest,	D Nx	100 100		N O N	E			5294.1 5291.7	5.7 8.4			0.0'-5.7': CONCRETE; tight bond with underlying rock, contact is smooth, planar and about 60° to core axis. 5.7'-8.4': WELDED ASH-FLOW TUFF: fairly hard, slightly porphyritic. Intensely Jointed. Joint spacing varies from core fragments to 0.4', averaging 0.1'. Most joints are 60° to 70° to core axis, rough to smooth and planar to irregular. Joint at 7.7' is smooth, planar, 60° to core axis and filled with 1/4" to 1/2" of hard clay-like material.

*LOCATION: Sta. 11+11.6 offset 8.5' u/s ℄ grout cap.

EXPLANATION

CORE LOSS / CORE RECOVERY

Type of hole . . . D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Hx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Hx = 3"

7-1337 (8-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 2

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-607-A LOCATION Left Spillway Bay* GROUND ELEV. 5299.0' DIP (ANGLE FROM HORIZ.) 60°

COORDS. N. E.

BEGUN 9/29/76 FINISHED 10/2/76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 109.5' BEARING N74°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment

Skid mounted Long-year "34" drill.

Driller

A. Allen

Drilling Methods

Drilled with clear water using NWD-3 split tube core barrel.

Water Return

0.0'-9.7': 100%
9.7'-109.5': 95%

Water Level**
During Drilling

Date	Hole Depth	Water Depth
9/30	25.7	19.8
10/1	74.9	35.9

Hole Completion

Hole was cemented back to surface.

Purpose of Hole

Determine permeability of rock within grout curtain beneath spillway crest.

TYPE AND SIZE OF HOLE: DNx

CORE RECOVERY (%): 100, 100, 100, 100, 82, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100

PERCOLATION TESTS

Tests were made using single mechanical packer.

Depth From (P, Cs, or Cm)	Depth To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)	Elevation (Feet)	Depth (Feet)
					5294.1	5.7
P 5.7	40.3	1.3	10	10	5278.2	
						24.0
					5265.2	39.0
P 39.9	74.9	0.25	10	20	5247.9	59.0
P 74.5	109.5	0.04	10	20		

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-5.7': CONCRETE; tight irregular contact with underlying rock. Contact is 70° to 80° to core axis.

5.7'-24.0': WELDED ASH-FLOW TUFF; fairly hard (requires light hammer blow to break), slightly porphyritic, light weight, indistinct foliation. Lightly Jointed. Joint spacing varies from 0.1' to 1.6', averaging 1.0'. Most joints are 60°-70° to core axis, planar, smooth with few fillings. Light gray.

Grout was found in the following openings:
- 8.4': Grout 1/8" to 3/8" filling irregular, slightly rough joint about 60° to core axis.
- 11.3': Fairly smooth, planar joint about 35° to core axis, partially filled with grout up to 1/16" thick.
- 11.7': Grout stain in a smooth, planar joint about 35° to core axis.
- 13.7': Grout stain in a rough, irregular joint 60°-70° to core axis.
- 14.0': Grout and calcite from 1/4" to 3/8" filling a rough and very irregular opening.

24.0'-39.0'±: WELDED ASH-FLOW TUFF; fairly hard (requires light to moderate hammer blow to break), slightly porphyritic. Moderately to Intensely Jointed, joint spacing varies from 0.05' to 1.0', averaging 0.3', light purple gray with some hematite staining to 30.0'. Distinct foliation at 60° to 70° to core axis is caused by flattened wavy streaky pumice and flattened vesicles. Most jointing is parallel to the foliation.

Grout was found in the following openings:
- 24.6': Grout 1/8" thick on side of joint filled with up to 1" of calcite. Joint is planar, rough and about 30° to core axis.

39.0'±-59.0'±: WELDED ASH-FLOW TUFF; fairly hard, slightly porphyritic, medium to dark gray. Faint to indistinct foliation crosses the core at an angle of 60° to 70°. Lightly to Moderately Jointed. Joint spacing is from 0.2' to 2.0', averaging 1.0'

Grout was found in the following openings:
- 43.4': Grout 1/8" thick filling a smooth, planar joint about 80° to core axis.

EXPLANATION

*Location: Sta. 11+11.6, offset 7.8' u/s ℄ grout cap.

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 2 OF 2

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-607-A LOCATION Left Spillway Bay* GROUND ELEV. 5299.0' DIP (ANGLE FROM HORIZ.) 60°
COORDS. N. E.
BEGUN 9-29-76 FINISHED 10-2-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 109.5' BEARING N74°W
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	D Nx	100	P 74.5	109.5	0.04	10	20					
		100										
		100						5204.2	109.5			

45.6': Grout stain in a smooth, planar joint which dips about 45° and is nearly normal to foliation.
50.5': Grout 3/8" thick filling a calcite lined, fairly smooth and planar joint about 35° to core axis.
53.8': Grout 1/16" to 1/8" thick in a 1/2" wide irregular, broken vesicular zone near normal to core axis.

59.0'-109.5': WELDED ASH-FLOW TUFF; fairly hard (requires light to moderate hammer blow to break), slightly porphyritic, light weight, medium to dark gray. Lightly Jointed. Joint spacing varies from 0.3' to 4.0', averaging 2.0'. Most joints are planar and smooth with little or no staining and are 60° to 70° to core axis.
59.0'-72.0'±: Foliation is fairly distinct caused mainly by wavy, streaky flattened pumice about 60° to 70° to core.
72.0'-109.5': Foliation is faint to indistinct. The faint flow lineation crosses the core at an angle of from 30° to 45°.

Grout was found in the following openings:
60.5': Grout 3/8" thick filling a smooth planar joint about 15° to core axis, terminates on joint normal to core.
68.6': Grout stain in a smooth, planar, hematite stained joint about 65° to core axis. 70.3': Vesicular zone 0.2' wide containing scattered grout up to 1/4" thick. Zone is about 60° to core axis.
75.3'-76.3': Grout 1/4" to 1" thick is irregular broken zone which is lined in places with calcite up to 1/4". Joint terminates at 76.3' on a planar, slightly rough iron and calcite stained joint about 45° to core axis.
92.4': Grout stain in a planar, smooth joint about 20° to core axis.
93.0': Grout stain in a planar, smooth joint about 30° to core axis - not parallel to joint at 92.4'.
101.4': Grout stain in a smooth, planar joint about 10° to core axis.
102.9'-103.4': Grout stain in planar, rough, manganese stained joint which is nearly parallel to core.
105.0': Grout stain in a smooth, planar, calcite stained joint about 20° to core axis.

EXPLANATION

CORE LOSS
CORE RECOVERY

*Location: Sta. 11+11.6, offset 7.8' u/s ℄ grout cap.

Type of hole . . . D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 2 OF 2 HOLE NO. DH-607-A

GPO 854-626

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-608 LOCATION Left Spillway Bay* COORDS. N. ... E. ... GROUND ELEV 5299.0' DIP (ANGLE FROM HORIZ.) 90°

BEGUN 9/25/76 FINISHED 9/28/76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 125.8' BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drilling Equipment

Skid-mounted Longyear "34" drill

Drillers

A. Allen and F. Martin

Drilling Methods

Drilled open hole with clear water using NWD-3 split tube core barrel.

Water Return

0.0'-33.4': 100%
33.4'-125.9': 95%

Water Levels** During Drilling

Date	Hole Depth	Water Depth
9/27	33.4	20.6
9/28	87.9	43.3

Drilling Conditions

0.0'-19.7': Drilled fairly fast and smooth.
19.7'-45.1': Drilled fairly slow and smooth.
45.1'-73.9': Drilled fairly fast and smooth.
73.9'-83.1': Drilled fairly slow and smooth.

Hole Completion

Hole was cemented back to surface

Purpose of Hole

Determine permeability of rock within grout curtain beneath spillway crest.

TYPE AND SIZE OF HOLE

DNx

CORE RECOVERY (%)

100 (all runs, 0.0' to 100')

PERCOLATION TESTS

Tests were made using single mechanical packer

Depth From (P, Cs, or Cm)	Depth To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)
P 5.8	35.8	1.3	10	20
P 35.8	65.8	0.34	10	20
P 65.8	95.8	0.1	10	20

ELEVATION (FEET) / DEPTH (FEET)

Elevation (Feet)	Depth (Feet)
5292.8	6.1
5277.8	21.2
5266.0	33.0

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-6.1': CONCRETE; poor bond in irregular contact with underlying rock.

6.1'-21.2': WELDED ASH-FLOW TUFF; light gray 6.1' to 15'±, reddish gray 15'± to 21.2', fairly hard (requires light hammer blow to break), light weight, slightly porphyritic. Lightly Jointed. Joint spacing is 0.1' to 2.0', averaging 1.0', foliation is indistinct, core contains some scattered lapilli. Grout was found in the following joints:
- 7.7': Chalky grout stain in joint dipping 30°.
- 10.2': Grout up to 3/8" thick in smooth, planar, $CaCO_3$ stained joint that dips about 30°.
- 11.6': Grout 1/16" to 3/8" thick filling a very rough and irregular joint that dips about 40°. Calcite from 1/8" to 1/4" is also present in the joint.

21.2'-33'±. WELDED ASH-FLOW TUFF; reddish gray, fairly hard (requires light to moderate hammer blow to break). Intensely Jointed. Joint spacing varies from 0.1' to 0.5' and averages 0.3'. Most joints are normal to core. Foliation is very distinct with white bands of pumice about normal to core. Core is slightly porphyritic and slightly vesicular. Grout was found in the following joint:
- 26.65': Scattered grout up to 1/16" thick in fairly planar and smooth, iron and carbonate stained joint dipping about 10°.

33'±-104.2': WELDED ASH-FLOW TUFF; fairly hard (requires moderate hammer blow to break), slightly porphyritic. Moderately Jointed with local areas intensely fractured. Core lengths are from 0.1' to 2.5', average length about 1.0'. Foliation, mostly flattened pumice streaks, is faint to indistinct (some foliation is normal to core and some appear to be near parallel to the core). Grout was found in the following locations:
- 63.2'-64.1': Grout (partly chalky) from 1/16" to 1/8" thick in a hematite stained, fairly rough joint that dips about 80°. Joint terminates at lower end on a smooth, planar, iron-stained joint that is about normal to core axis.

*Location: Sta. 11+14.6, offset 8.5' u/s from ℄ grout cap.

EXPLANATION

CORE LOSS
CORE RECOVERY

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (8-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-608 LOCATION Left Spillway Bay* GROUND ELEV. 5299.0' DIP (ANGLE FROM HORIZ.) 90°
COORDS. N. ... E. ...
BEGUN 9/25/76 FINISHED 9/28/76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 125.8' BEARING --
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)
100						5194.8	104.2
100							
100	P 95.8	125.8	0.3	10	20		
100	0	125.8	0.6	Gravity	5		
100							
100						5173.2	125.8

CLASSIFICATION AND PHYSICAL CONDITION

72.8': Calcite and grout up to 1/2" thick filling a smooth planar joint that dips about 30°.
73.3': Intersection of 30° and 60° dipping joints that are smooth and planar and calcite stained--at junction of joints is wedge shaped piece of grout up to 1-1/4" thick.
74.7': Grout stain and grout up to 1/8" thick in two intersecting joints, one dipping about 85°, the other dipping about 70°. The joints are stained with calcium carbonate and a hard clay-like material.
76.0': Grout up to 1/16" in smooth, planar, iron and $CaCO_3$ stained joint which dips about 25°.
76.1'-76.5': Grout stained, irregular broken fracture nearly parallel to core with $CaCO_3$ up to 1/4" thick.
76.5'-77.0': Smooth, planar joint 1/2" wide filled with grout and angular rock fragments up to 1/2" across. $CaCO_3$ occurs as a thin lining on parts of the joint surface.
77.0': Irregular, fairly rough, $CaCO_3$ stained, 20° dipping joint that is filled with grout up to 1/2" thick.
77.1'-77.7': Grout up to 1/2" thick in wavy, fairly smooth joint dipping 75°-80°. One side of joint is lined with calcite 1/8" thick.
78.6'-78.7': Grout about 1" thick filling 1/8" thick calcite lined irregular, fairly smooth joint about normal to core axis.
78.7'-79.2': Planar, smooth, 1/8" thick calcite lined, 80° dipping joint filled with 1" of solid grout.
79.6': Grout stain in smooth, planar joint that dips about 75°. Joint contains $CaCO_3$ 1/8" thick.
79.7'-82.5': Two nearly parallel joints that are nearly parallel to the core contain some broken rock fragments up to 1/2", $CaCO_3$ up to 1/4" hard clay-like materials up to 1/8" and grout up to 1" thick. Parts of joints are smooth, parts are rough.
82.3'-83.1': Wavy, fairly smooth joint dips 75° to 80°, contains $CaCO_3$ and grout up to 1/4" thick.
83.8'-85.1': Irregular to wavy, slightly rough joint that is near parallel to core is filled with $CaCO_3$ up to 3/8" thick and grout up to 1/2" thick. A clay-like material stains some of the surfaces.
87.9': Grout stain in irregular, rough, iron stained joint about normal to core.

EXPLANATION

*Location: Sta. 11+14.6, offset 8.5' u/s from ℄ grout cap.

CORE LOSS
CORE RECOVERY

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series).. Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-608 LOCATION Left Spillway Bay* COORDS. N. ... E. ... GROUND ELEV. 5299.0' DIP (ANGLE FROM HORIZ.) 90°
BEGUN 9/25/76 FINISHED 9/28/76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 125.8' BEARING --
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
												90.5'-91.2': Grout 1/2" to 1-1/2" thick filling fairly planar $CaCO_3$ coated joint that dips about 85°. 91.5'-93.8': Rough, irregular opening containing scattered grout from 1/8" to 1/2" thick. Opening is stained with $CaCO_3$, iron and a clay-like material. 95.4': Itersection of two planar, slightly rough joints both dipping about 70° (near normal to each other) filled with grout 1/4" to 1/2" thick. Joints are stained with a silt-like material. 97.7': Grout 1/4" thick filling smooth, planar, clay (?) stained joint which dips about 10°. 97.8'-98.3': Grout 1/8" to 1/4" thick filling fairly smooth, planar joint. Locally a clay-like material fills joint up to 3/8" thick.
												104.2'-125.8': WELDED ASH-FLOW TUFF; medium to dark gray, hard (requires moderate hammer blow to break), fairly prominent foliation caused by streaky flattened pumice and some flattened vesicular zones that dip from 0° to 10°. Moderately Jointed Most joints are smooth, planar and 70°-80° to core axis, average spacing is 0.8'. Most of the joints are parallel to the foliation, but others cross the core at various angles. Grout occurs in the following openings: 105.2'-105.7': Grout up to 1-1/4" thick filling a smooth, planar joint that dips about 80°. 107.5'-108.4': Grout 1" to 1-1/2" filling a planar, smooth 70° dipping joint that is lined with calcite from 1/4" to 1/2". 111.5'-112.9': Grout locally up to 1/8" thick in irregular, rough, iron and calcium carbonate stained joint which is nearly parallel to core. 115.9'-116.8': Planar, smooth, $CaCO_3$ stained joint filled with grout 1" thick. 123.3'-123.9': Fairly smooth and planar joint dipping 60° filled with a chaotic mixture of calcite up to 1" thick and grout up to 1/4" thick.

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90

EXPLANATION

CORE LOSS
CORE RECOVERY

*Location: Sta. 11+14.6, offset 8.5' u/s from ℄ grout cap.

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 2

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-609 LOCATION Left Spillway Bay* GROUND ELEV. 5299.0' DIP (ANGLE FROM HORIZ.) 60°

COORDS. N. E.

BEGUN 10/2/76 FINISHED 10/6/76 DEPTH OF OVERBURDEN 0.0 TOTAL DEPTH 145.0' BEARING S74°E

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes** LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment

Skid mounted Long-year "34" drill.

Driller

A. Allen

Drilling Methods

Drilled with clear water using NWD-3 split tube core barrel.

Water Return

0.0'-8.1': 100%
8.1'-145.0': 95%

Water Levels**
During Drilling

Date	Hole Depth	Water Depth
10/4	50.1	Dry
10/5	98.1	90.0
10/6	141.8	103.0

Hole Completion

Hole was cemented back to surface.

Purpose of Hole

Determine permeability of rock within grout curtain beneath spillway crest.

TYPE AND SIZE OF HOLE: DNx

CORE RECOVERY (%): 90, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100

PERCOLATION TESTS

Tests were made using single mechanical packer.

Depth (feet) From (P, Cs, or Cm)	To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)
P 5.7	40.7	0.9	10	20
P 40.3	75.3	1.8	10	20
P 74.9	109.4	2.2	10	20

Elevation (feet)	Depth (feet)
5293.6	6.2
5276.7	25.8
5244.6	62.8

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-6.2': CONCRETE. Tight bond with underlying rock, contact is about 30° to core axis.

6.2'-25.8': WELDED ASH-FLOW TUFF; medium gray, fairly hard (breaks with light hammer blow), light weight, slightly porphyritic. Lightly Jointed. Joints are spaced 0.1' to 2.9', averaging 1.2'. Most joints are 30° to core axis, smooth and planar, some are iron stained. Foliation is indistinct. Grout was found in the following openings:
9.15': Grout 1/8" thick filling fairly planar, rough joint about 30° to core axis.
12.4': Intersection of three smooth, planar joints with grout, iron and manganese staining. Joints are about 60° to core axis.
16.2': Grout up to 1/16" partially filling a calcite, iron, and silt stained, smooth, planar joint about 45° to core axis.
24.9': Grout up to 1/32" thick partially filling an iron stained, smooth, planar joint about 45° to core axis.

25.8'-62.8'±: WELDED ASH-FLOW TUFF; medium to dark gray, streaked with red 25.8'-37'±, fairly hard (breaks with light to moderate hammer blow), slightly porphyritic. Intensely Jointed. Joint spacing varies from 0.05' to 1.0', averaging 0.3'. Most joints are parallel to foliation which is 50° to 60° to core axis. Foliation from 25.8 to 51.2' is very distinct with flattened wavy pumice crossing the core at 45° to 60°. From 51.2' to 62.8' the foliation is very faint to indistinct. Grout was found in the following intervals:
28.1': Grout stain in places in a planar, rough joint about 45° to core (parallel to foliation).
28.7': Grout stain in a planar, rough joint. Joint is calcite and hematite stained and about 45° to core axis (parallel to foliation).

EXPLANATION

*Location: Sta. 11+17.6, offset 8.5' u/s of ℄ grout cap.

CORE LOSS
CORE RECOVERY

Type of hole .... D = Diamond, H = Noysrellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) .. Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 2 HOLE NO. DH-609

GPO 854-628

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE: Teton Dam PROJECT: Teton Basin STATE: Idaho

HOLE NO.: DH-609 LOCATION: Left Spillway Bay* COORDS. N. ... E. ... GROUND ELEV: 5299.0' DIP (ANGLE FROM HORIZ.): 60°

BEGUN: 10/2/76 FINISHED: 10/6/76 DEPTH OF OVERBURDEN: 0.0 TOTAL DEPTH: 145.0' BEARING: S74°E

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED: See Notes** LOGGED BY: D. N. Magleby LOG REVIEWED BY: B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)
		100							
		100							
		92							
		100							
		100							
		100							
		100	P 110.0	145.0	0.3	10	20		
		100							
		100	0.0	145.0	3.1	Gravity	5		
		100							
		100						5173.4	145

CLASSIFICATION AND PHYSICAL CONDITION

62.8'±-145.0': WELDED ASH-FLOW TUFF; medium gray, fairly hard (breaks with light to moderate hammer blow), slightly porphyritic. Moderately Jointed. Joint spacing is 0.05' to 3.0', averaging about 1.0'. Most joints are smooth and planar and nearly parallel to foliation which is 60° to 80° to core axis. Some prominent joints are 30° to core axis. Foliation, which is 60° to 80° to core axis, is very faint and is the result mainly of flattened vesicles.
Grout was found in the following openings:
66.3': Irregular grout filling up to 1/4" thick in a rough vesicular zone about 60° to core axis.
73.0': Grout stain (and grout blob up to 1/4" across) in a very irregular vesicular zone that is stained with calcite and a silt-like material. The vesicular zone is about normal to core axis.
94.05': Grout up to 1/16" in a smooth planar, manganese and iron stained joint about 60° to core axis.
122.3': Grout stain in an irregular rough joint about 30° to core axis.
127.3': Grout 3/8" thick filling a smooth, planar, calcite lined (1/16") manganese stained joint about 25° to core axis.

CORE LOSS
CORE RECOVERY

EXPLANATION

*Location: Sta. 11+17.6, offset 8.5' u/s of ℄ grout cap.

Type of hole . . . D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-610 LOCATION Right Abutment Keyway* GROUND ELEV. 5222.1 DIP (ANGLE FROM HORIZ.) 48°

COORDS. N. E.

BEGUN 11-1-76 FINISHED 11-2-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 24.8' BEARING N68°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment

Mark 1X portable drill.

Driller

A. Allen

Drilling Methods

Drilled with NxD-3 split tube core barrel, using clear water.

Water Return

0.0'-24.8' - 100%

Hole Completion

Hole was cemented back to surface.

Purpose of Hole

Test effectiveness of grout curtain.

TYPE AND SIZE OF HOLE

D Nx

CORE RECOVERY (%)

100, 100, 100, 100, 100, 100, 100

PERCOLATION TESTS

Tests were made using single mechanical packer.

DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)
P 3.4	14.8	0.38	10	20
P 14.8	24.8	0.05	10	20

ELEVATION (FEET)	DEPTH (FEET)
5218.6	4.7
5203.7	24.8

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-4.7': CONCRETE; hard, tight, irregular contact with underlying rock. (Grout Cap)

4.7'-24.8': WELDED ASH-FLOW TUFF; dark gray, hard (core breaks with moderate hammer blow and scratches with heavy knife pressure). Slightly porphyritic, non vesicular, indistinct foliation. Intensely to Moderately Jointed. Joint spacing is from 0.2' to 1.4', averaging 0.7'. Most joints are planar and smooth and are about 35° to 45° to core axis. Many joints are hematite and manganese stained. No grout was observed in core.

Prominent joints:

5.6'-7.5': Smooth, planar, iron and manganese stained joint about 10° to core axis.

17.0': Intersection of 2 planar and rough joints, one about 70° to the core axis, the other one about 25° to the core axis.

17.2': Smooth, planar, iron and manganese stained joint about 15° to core axis.

19.3': Planar, rough, iron stained joint about 40° to core axis.

EXPLANATION

* LOCATION: Sta. 12+73
℄ grout cap.

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, N = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO. DH-610

GPO 884-828

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-611 LOCATION Right Abutment Keyway* COORDS. N. E. GROUND ELEV. 5222.1 DIP (ANGLE FROM HORIZ.) 70°

BEGUN 11-3-76 FINISHED 11-3-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 23.9' BEARING N68°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY S. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment

Mark IX portable drill.

Driller

A. Allen

Drilling Methods

Drilled with NxD-3 split tube core barrel using clear water.

Water Return

0.0'-23.9' - 100%

Hole Completion

Photographed hole with down-hole camera, then back-filled with grout.

Purpose of Hole

Test effectiveness of grout curtain.

TYPE AND SIZE OF HOLE: D Nx

CORE RECOVERY (%): 97, 100, 100, 100, 100, 100, 100, 100

PERCOLATION TESTS

Tests were made using single mechanical packer.

Depth (feet) From (P, Cs, or Cm)	To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of test (min.)
P 2.5	13.9	0.48	10	20
P 13.9	23.9	0.09	10	20

Elevation (feet)	Depth (feet)
5216.5	3.9
5197.7	23.9

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-3.9': CONCRETE; (Grout Cap), hard, broken in 1-foot lengths by drilling. Contact with underlying rock is fairly well cemented and is about normal to core axis.

3.9'-23.9': WELDED ASH-FLOW TUFF; dark gray, fairly hard (requires moderate hammer blow to break and heavy knife pressure to scratch), slightly porphyritic, non vesicular. Foliation is indistinct. Intensely Jointed. Joint spacing varies from 0.2' to 0.8', averaging 0.4'. Most joints are near-parallel, smooth, planar, manganese and hematite stained and 40°-45° to core axis. No grout was observed in core.
Prominent joints:
6.3': Smooth, wavy joint, manganese, hematite and calcite stained which is 35°-40° to core axis.
6.8': Calcite up to 1/8" partially filling a smooth planar joint 40° to core axis.
9.0'; 12.7' & 20.3': Planar rough joints with strong hematite staining, orientation is 10° to 15° to core axis.
22.4': Rough, irregular hematite stained joint about 80° to core axis.

EXPLANATION

*LOCATION: Sta. 12+74 ℄ grout cap.

CORE LOSS / CORE RECOVERY

Type of hole . . . D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO. DH-611

GPO 834-628

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE: Teton Dam — PROJECT: Teton Basin — STATE: Idaho

HOLE NO.: DH-612 — LOCATION: Right Abutment Keyway, Sta. 12+75 ℄ Grout Cap. — COORDS. N. ... E. ... — GROUND ELEV.: 5222.1 — DIP (ANGLE FROM HORIZ.): 90°

BEGUN: 10-26-76 — FINISHED: 11-1-76 — DEPTH OF OVERBURDEN: 0.0' — TOTAL DEPTH: 23.5' — BEARING: ...

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED: Not Measured — LOGGED BY: D. N. Magleby — LOG REVIEWED BY: B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	D Nx	100	Tests were made using Single Mechanical Packer.					5218.6				0.0'-3.5': CONCRETE; (Grout Cap); hard, good bond with underlying rock, contact is normal to core.
Drill Equipment		100	P 2.1	13.5	0.5	10	20					3.5'-23.5': WELDED ASH-FLOW TUFF; dark gray, hard (core breaks with moderate hammer blow and scratches with heavy knife pressure), slightly porphyritic, non vesicular, indistinct foliation. Intensely Jointed. Joint spacing is from 1/2" (fragments) to 1.3', averaging 0.3' to 0.4'. Most joints are near parallel, planar, smooth, hematite and manganese stained and dip from 0° to 20° (others dip between 45° and 60°).
Mark IX portable drill.		100										Prominent joints:
		100										6.2': Smooth, planar, iron stained joint dipping 35°.
Driller		100										19.4': Smooth planar, iron stained joint dipping about 35°.
A. Allen		100	P 13.5	23.5	0	10	10					22.9': Grout stains in a smooth, planar, iron and manganese stained joint that dips about 45°.
		100						5198.6	23.5			
Drilling Methods												
Drilled with NxD-3 split tube core barrel using clear water.												
Water Return												
0.0'-23.5' - 100%												
Hole Completion												
Cemented hole back to surface.												
Purpose of Hole												
Determine effectiveness of grout curtain.												

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90

EXPLANATION

CORE LOSS / CORE RECOVERY

Type of hole . . . D = Diamond, N = Noystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE: Teton Dam — PROJECT: Teton Basin — STATE: Idaho — SHEET 1 OF 1 — HOLE NO. DH-612

GPO 854-825

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE: Teton Dam — PROJECT: Teton Basin — STATE: Idaho

HOLE NO. DH-613 — LOCATION: Sta. 13+14.6 ℄ Grout Cap* — GROUND ELEV. 5206.1 — DIP (ANGLE FROM HORIZ.) 45°

COORDS. N. — E.

BEGUN 11-13-76 — FINISHED 11-14-76 — DEPTH OF OVERBURDEN 0.0 — TOTAL DEPTH 24.8' — BEARING N68°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED: Not measured — LOGGED BY: D. N. Magleby — LOG REVIEWED BY: B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	CLASSIFICATION AND PHYSICAL CONDITION
Drill Equipment Mark IX portable drill. Driller A. Allen Drilling Methods Drilled with clear water using NWD-3 split tube core barrel. Water Return 0.0'-24.8': 100% Hole Completion Backfilled with grout. Purpose of Hole Test permeability of grout curtain.	D Nx	100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100	Tests were made using single mechanical packer. P 3.4	14.8	1.1	10	20	5202.7	4.8	0.0'-4.8': CONCRETE (Grout cap); hard, broken by drilling into 1-foot lengths. Contact with underlying rock is fairly planar, smooth and tight.
			P 14.8	24.8	0.2	10	20	5188.5	24.8	4.8'-24.8': WELDED ASH-FLOW TUFF; light gray 4.6' to 10'±, pinkish gray 10'± to 24.8', hard (requires moderate hammer blow and heavy knife pressure to scratch core), slightly porphyritic, nonvesicular, faint foliation 20° to 30° to core axis caused mostly by streaky flattened pumice fragments. Lightly to Locally Moderately Jointed. Joint spacing varies from 1/2" core fragments up to 2.5 feet, average length 1.2 feet, many joints are about 30° to core axis, smooth and planar, others have random orientation and are mostly rough. No grout was noted in the core. The major jointing is as follows: 5.6': Smooth, planar to wavy, manganese stained joint about 20° to core axis. 7.5'-8.3': Slightly wavy, smooth, manganese and hematite stained joint 10° to core axis. 8.9'-9.8': Smooth, planar, manganese, hematite and calcite stained joint about 10° to core axis. 10.5': Smooth, planar, iron and manganese stained joint 25° to core axis, contains scattered calcite up to 1/16" thick. 11.3': Smooth, wavy, iron, manganese, calcite and silt stained joint about 10° to 20° to core axis. 11.6': Smooth, planar, hematite stained joint about 30° to core axis and about parallel with the faint foliation. 13.5'-14.0': Smooth, curved joint from parallel to 10° to core axis partially filled with calcite up to 3/8" thick. 16.9': Smooth, planar, hematite stained joint about 30° to core axis. 19.1': Planar, rough, hematite and calcite stained joint about 55° to core axis. 20.1': Smooth, planar joint 30° to core axis with calcite to 1/8" thick. 20.3': Smooth, planar limonite stained joint parallel to 20.1' joint.

EXPLANATION

* Right abutment keyway

CORE LOSS
CORE RECOVERY

Type of hole . . . D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam — PROJECT Teton Basin — STATE Idaho — SHEET 1 OF 1 — HOLE NO. DH-613

GPO 864-825

7-1337 (8-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-614 LOCATION Sta. 13+14.8 C. Grout Cap** GROUND ELEV. 5206.1 DIP (ANGLE FROM HORIZ.) 68°

COORDS. N. E.

BEGUN 11/11/76 FINISHED 11/13/76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 24.4' BEARING N68°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment

Mark IX portable drill.

Driller

A. Allen

Drilling Methods

Drilled with clear water using NwD-3 split tube core barrel.

Water Return

0.0'-24.4': 100%

Hole Completion

Surveyed with bore-hole camera, then backfilled with grout.

Purpose of Hole

Test permeability of grout curtain.

TYPE AND SIZE OF HOLE / CORE RECOVERY / PERCOLATION TESTS

Type and size of hole: D Nx

Core recovery (%): 100, 100, 100, 100, 100, 100, 100, 100, 100, 100

Depth From (P, Cs, or Cm)	Depth To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)
P 3.3	14.4	1.1	10	20
P 13.4	24.4	1.1*	10	20
P 3.3	24.4	0.9	10	20
0.0	24.4	0.5	Gravity	5

*Leakage out of top of hole at rate of about 1 gpm.

Elevation (Feet)	Depth (Feet)
5202.0	4.4
5183.4	24.4

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-4.4': CONCRETE (Grout cap); hard, broken into 1-foot lengths by drilling. Contact with underlying rock is smooth, planar, tight and about 30° to core axis.

4.4-24.4': WELDED ASH-FLOW TUFF; medium gray, slightly porphyritic, non-vesicular, hard (requires moderate hammer blow and heavy knife pressure to scratch core), foliation is indistinct. Lightly to Locally Intensely Jointed. Joint spacing varies from 1/2" core fragments to 3.3', average about 1.0'. Most jointing is smooth, planar, manganese stained and 30° to 60° to core axis.

Prominent joints:

- 4.6'-5.6': Smooth, planar, manganese and silt stained joint nearly parallel to the core.
- 5.6': Smooth, planar, manganese stained joint 50° to core axis.
- 5.9': Smooth, planar, manganese stained joint about 50° to core axis.
- 6.3': Same as joint at 5.9'.
- 6.7': Smooth, planar, manganese stained joint about 30° to core axis and near normal to joint at 5.6'.
- 7.3': Smooth, planar, manganese and silt stained joint about normal to joint at 6.7'.
- 9.5': Same as joint at 7.3'.
- 11.8': Smooth, planar, manganese stained joint about 30° to core axis.
- 12.2': Rough, planar, manganese and hematite stained joint about 40° to core axis.
- 12.3': Smooth, wavy to planar joint about 40° to core axis and near normal to joint at 12.2'.
- 13.0': Broken, iron stained zone 0.2' wide with several joints 20°-30° to core axis.
- 14.5'-15.5': Smooth, slightly wavy, manganese stained joint from 10° to near parallel to core.
- 15.6': Rough, planar, limonite stained joint 30° to core axis.
- 16.7': Smooth, planar, manganese stained joint about 20° to core axis.
- 19.0': Smooth, planar, manganese stained joint about 30° to core axis, contains grout 1/8" thick.
- 21.8': Rough, planar, limonite stained joint near normal to core axis.

EXPLANATION

**Right abutment keyway

CORE LOSS / CORE RECOVERY

Type of hole: D = Diamond, N = Hoystellite, S = Shot, C = Churn
Hole sealed: P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series): Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series): Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series): Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series): Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO. DH-614

GPO 854-625

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE: Teton Dam PROJECT: Teton Basin STATE: Idaho

HOLE NO. DH-615 LOCATION: Right Abutment Keyway* COORDS. N. ... E. ... GROUND ELEV 5206.0 DIP (ANGLE FROM HORIZ.) 90°

BEGUN 11-10-76 FINISHED 11-11-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 24.9' BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED: Not Measured LOGGED BY: O. N. Magleby LOG REVIEWED BY: B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	D Nx	100, 100, 100	Tests were made using single mechanical packer.					2501.1	4.9			0.0'-4.9': CONCRETE (Grout Cap); hard, broken by drilling; contact with underlying rock is uncemented, broken, rough and about 30° to core axis.
Drill Equipment Mark IX portable drill.		100, 100	P 3.5	14.9	0	10	20					4.9'-24.9': WELDED ASH-FLOW TUFF; light to medium gray, hard (breaks with mod. hammer blow and scratches with heavy knife pressure). Lightly to Locally-Intensely Jointed. Joint spacing varies from fragments of core 1/2" up to core lengths of 4.8', average length is 1.3'. Most joints are smooth, planar, manganese and limonite stained and dip 45° to 60°. Very faint foliation (wavy, streaky flattened pumice and some flattened vesicles) dip 75° to 80°.
Driller A. Allen		100, 100, 100, 100	P 14.9	24.9	0	10	20	2481.1	24.9			Prominent joints: 5.2'-5.4': Irregular, broken rough joint dips about 60°. 5.6': Smooth, planar, manganese stained joint dips 10°. 5.6'-5.7': Rough, planar joint that dips about 75°. 11.1': Smooth, planar, silt stained joint that dips 50° and contains grout up to 1/16" thick. 12.4': Very rough, planar, manganese stained joint dipping about 75°. Rock is bleached for 3/8" each side of joint. 14.9'-15.2': Rough, irregular, manganese stained joint that dips about 80°.
Drilling Methods Drilled with NwD-3 core barrel using clear water. Water Return 0.0'-24.9' - 100% Hole Completion Backfilled hole with cement. Purpose of Hole Test effectiveness of grout curtain.												

EXPLANATION

*LOCATION: Sta. 13+15 ℄ grout cap.

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO. DH-615

GPO 884-828

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-616 LOCATION Right Abutment Keyway* GROUND ELEV. 5197.4 DIP (ANGLE FROM HORIZ.) 43°

COORDS. N. E.

BEGUN 11-8-76 FINISHED 11-9-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 24.3 BEARING N68°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment

Mark IX portable drill.

Driller

A. Allen

Drilling Methods

Drilled with NwD-3 split tube core barrel using clear water.

Water Return

Partial water loss temporarily at 17.5'; otherwise 100% return 0.0' to 8.5' and 90% return 8.5' to 24.3'.

Hole Completion

Photographed hole with down-hole camera and then backfilled with grout.

Purpose of Hole

Determine effectiveness of grout curtain.

TYPE AND SIZE OF HOLE: D Nx

CORE RECOVERY (%): 100, 100, 100, 100, 100, 100, 100

PERCOLATION TESTS

Tests were made using single mechanical packer.

FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)
P 2.9	14.3	10.1**	10	10
P 6.4	14.3	0.23	10	10
0.0	14.3	3.5	Gravity	5
P 14.3	24.3	5.2***	10	20
P 18.9	24.3	0	10	10

ELEVATION (FEET)	DEPTH (FEET)
5194.3	4.6
5180.9	24.3

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-4.6': CONCRETE (Grout Cap); hard, broken into 0.3' to 1.0' core lengths by drilling, broken 4.5' to 4.6' next to contact with underlying rock. Contact is uncemented.

4.6'-24.3': WELDED ASH-FLOW TUFF; medium gray, hard (breaks with Mod. hammer blow, scratches with heavy knife pressure). Lightly to Locally-Intensely Jointed. Most joints are smooth and planar with manganese and some calcite staining. Most joints are about 40° to core axis, joint spacing varies from 1/2" fragments up to 3.5 feet, averaging 1.2 feet. Faint foliation caused by flattened vesicles and flattened pumice cross the core at an angle of about 20° to core axis.

Prominent joints:

4.6'-4.8': Rough, planar joint about 40° to core axis.

5.0'-5.4': Rough, irregular joint about 25° to core axis.

5.1'-5.9': Smooth, planar joint about 15° to core axis containing calcite and silt-like staining and scattered grout up to 1/16".

6.0'-6.4': Smooth, planar, manganese, hematite and calcite stained joint about 40° to core axis.

9.3' & 9.8': Smooth, planar, manganese stained, parallel joints about 35° to core axis.

13.6': Rough, irregular, limonite stained joint containing some sand-size rock fragments.

15.1': Rough, irregular joint about 45° to core. Limonite staining is prominent for 1/8" each side of joint.

17.5': Smooth, planar joint 30° to core axis filled with 1/4" thick grout.

21.5': Smooth, planar joint 30° to core axis filled with 1/8" thick grout.

22.4': Rough, planar manganese stained joint 60°-70° to core axis.

EXPLANATION

CORE LOSS / CORE RECOVERY

*LOCATION: Sta. 13+28.8, offset 0.4' d/s ℄ grout cap. Hole is also 0.6' d/s from grout nipple.

**Leakage noted from joints on d/s & u/s side of grout curtain, est. 5 gpm on d/s side, 2 gpm on u/s side.

***Leakage noted from joints as far away as 80' d/s (20' lower in elevation) from the drill hole.

Type of hole: D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed: P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series): Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series): Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series): Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series): Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-617 LOCATION Right Abutment Keyway* GROUND ELEV 5197.4 DIP (ANGLE FROM HORIZ.) 69°

COORDS. N. E.

BEGUN 11-7-76 FINISHED 11-8-76 DEPTH OF OVERBURDEN 0.0 TOTAL DEPTH 23.9' BEARING N68°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY R. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	D Nx	100	Tests were made using single mechanical packer									0.0'-3.9': CONCRETE; medium gray, hard, broken by drilling into 0.1'-1.3' lengths, good tight contact with underlying rock, contact is irregular, rough and about normal to core.
								5193.8	3.9			
Drill Equipment		100	P 2.5	13.9	0	10	10					3.9'-23.9': WELDED ASH-FLOW TUFF; hard (breaks with moderate hammer blow, scratches with heavy knife pressure), slightly porphyritic, non vesicular, faint flow lineations (caused by flattened vesicles and pumice) in a few sections of core orientated about 80° to core axis. Lightly Jointed. Joint spacing is from 0.1' to 5.3', averages 1.5'.
Mark IX portable drill.		100							10			
Driller		100	P 13.9	23.9	0	10	5					
A. Allen		100	0	23.9	0.4	Gravity	5		20			
		100						5175.1	23.9'			

Drilling Method

Drilled with NxD-3 split tube core barrel using clear water.

Water Return

0.0'-23.9' - 100%

Hole Completion

Cemented back to surface.

Purpose of Hole

Test effectiveness of grout curtain.

Prominent joints:

- 4.0'-4.7': Wavy, rough opening nearly parallel to core stained with silt, calcite and grout.
- 4.7'-6.7': Smooth, slightly wavy, manganese stained joint nearly parallel to core.
- 6.5'-7.2': Rough, planar, manganese stained joint nearly parallel to core.
- 8.4'-10.0': Smooth, planar, manganese and grout stained joint near parallel to core.
- 9.9' & 10.1': Parallel, smooth, planar joints 30° to core axis stained with manganese and grout.
- 10.8': Irregular, rough, hematite and manganese stained joint.
- 11.8' & 12.4': Irregular, rough, manganese and iron stained joints about parallel to faint flow lineations which are about 80° to core axis.
- 17.1': Smooth, planar, manganese stained joint about 40° to core axis.
- 18.0': Very rough, slightly planar opening, manganese stained, orientated about 40° to core axis.
- 23.4': Smooth, planar, manganese stained joint about 45° to core axis.

EXPLANATION

*LOCATION: Sta. 13+30, offset 0.9' u/s ℄ grout cap.

CORE LOSS / CORE RECOVERY

Type of hole . . . D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-618 LOCATION Right Abutment Keyway Sta. 13+30 S ℄ Grout Cap. GROUND ELEV. 5197.4 DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. E.

BEGUN 11-3-76 FINISHED 11-4-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 24.2' BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	CLASSIFICATION AND PHYSICAL CONDITION
	D Nx	100, 100, 100, 100, 100, 100, 100, 100	Tests were made using single mechanical packer.							
			P 2.8	14.2	0	10	10	5193.2	4.2	
			P 14.2	24.2	0.8	10	20	5173.2	24.2'	

Drill Equipment

Mark IX portable drill.

Driller

A. Allen

Drilling Method

Drilled with NxO-3 split tube core barrel using clear water.

Water Return

0.0'-24.2' - 100%

Hole Completion

Cemented hole back to surface.

Purpose of Hole

Test effectiveness of grout curtain.

Classification and Physical Condition

0.0'-4.2': CONCRETE; hard, broken into about 1-foot lengths by drilling, well cemented contact with underlying rock contact dips about 45°.

4.2'-24.2': WELDED ASH-FLOW TUFF; medium to dark gray, hard (requires Mod. hammer blow to break, scratches with heavy knife pressure), slightly porphyritic, non vesicular. Flow lineations are indistinct except 22.0'-24.2' where some flattened pumice streaks are about normal to core axis. Moderately Jointed: Joint spacing varies from fragments of core 0.1' long to 1.9', averaging about 1.1'. Most joints are planar, rough to smooth, lightly iron stained and are near normal to core axis. No grout is visible in the core.

Prominent joints:

- 4.6'-5.5': Smooth, planar, manganese and hematite stained joint dipping about 80°.
- 8.4': Rough, irregular opening (appears to be open 1/16") dipping about 70°.
- 16.2'-17.0': Rough, irregular, manganese stained opening dipping 85°-90°.
- 17.3': Rough, irregular, manganese stained opening dipping about 70°.
- 21.8': Irregular broken zone about 0.2' wide with manganese and hematite staining.
- 22.7': Smooth, planar joint partially filled with up to 1/4" thick calcite. Joint is about normal to core.
- 22.8': Rough, planar, hematite stained joint about normal to core axis.
- 23.8': Smooth, planar, hematite and manganese stained joint dipping about 55°.

EXPLANATION

CORE LOSS / CORE RECOVERY

Type of hole: D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed: P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series): Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series): Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series): Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series): Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO. DH-618

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 2

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-619 LOCATION Right Abutment Keyway. Sta. 13+30.7 ℄ Grout Cap.
COORDS. N. ... E. ... GROUND ELEV. 5197.4 DIP (ANGLE FROM HORIZ.) 59°
BEGUN 11-5-76 FINISHED 11-6-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 26.7' BEARING S68°E
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B.H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	CLASSIFICATION AND PHYSICAL CONDITION
	0 Nx	98	Tests were made with single mechanical packer.							0.0'-6.7': CONCRETE; hard, broken into about 1-foot lengths by drilling. Contact with underlying rock is smooth, planar, 60° to core axis and is uncemented.
Orill Equipment		93						5191.7	6.7	
Mark IX portable drill.		100	P 5.3	16.7	1.6*	10	20			6.7'-26.7': WELDED ASH-FLOW TUFF; medium to dark gray to reddish gray (17'±-26.7'), hard (breaks with hammer blow, scratches with heavy knife pressure), slightly porphyritic, non vesicular. Flow lineations are indistinct. Lightly to Moderately Jointed. Joint spacing varies from 0.1' to 2.5', averages about 1.0'. Many joints are planar, fairly smooth and about 60° to core axis but other joints are rough with random orientation.
Driller		100								
A. Allen		100								
		100	P 16.7	26.7	4.8**	10	20			
Drilling Method		100						5144.5	26.7	

Orill Equipment

Mark IX portable drill.

Driller

A. Allen

Drilling Method

Drilled with NxD-3 split tube core barrel using clear water.

Water Return

0.0'-26.7' - 100%

Hole Completion

Hole was photographed with downhole camera and then backfilled with grout.

Purpose of Hole

Test effectiveness of grout curtain.

Prominent joints:
8.6': Chalky grout stain in irregular, rough joint 45° to core axis.
10.0': Rough, irregular, grout stained joint about 45° to core axis.
10.3': Planar, smooth, manganese stained joint 25° to core axis.
11.0': Irregular, rough joint 65° to core axis.
11.9': Smooth, planar, manganese and chalky grout stained joint about 25° to core axis.
17.0': Planar, rough, hematite stained joint about 60° to core axis.
19.3': Smooth, planar joint 1" wide filled with 1/4" thick calcite on upper surface and 3/4" thick compact fine silty sand that contains a few 1/2" rock fragments. Joint is 45° to core axis.
19.7': Planar, rough, hematite and calcite stained joint about 45° to core axis. Joint appears to be open 1/8"±. (Faint foliation from 19.7' to 20.7' is parallel to core).
20.9': Irregular, rough joint stained with calcite and grout.
21.3'-21.8': Planar, rough, hematite stained joint about parallel to core.
21.9': Rough, irregular joint stained with calcite and hematite and containing a compact, fine silty sand up to 1/4".

EXPLANATION

* Leakage noted on d/s side of grout cap.
** Leakage noted on d/s and u/s side of grout cap.

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) .. Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 2 HOLE NO. DH-619

GPO 854-626

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-619 LOCATION Right Abutment Keyway, Sta. 13+30.7 ℄ Grout Cap GROUND ELEV. 5197.4 DIP (ANGLE FROM HORIZ.) 59°
COORDS. H. E.
BEGUN 11-5-76 FINISHED 11-6-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 26.7' BEARING S68°E
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
												23.0': Smooth, planar, hematite stained joint about 45° to core axis. 24.2' & 24.4': Parallel, rough, planar, hematite stained joints about 60° to core axis. 24.4'-25.0': Smooth, planar, hematite stained joint about 15° to core axis. 25.0': Planar, fairly smooth, hematite stained joint about 80° to core axis. Joint is partially filled with calcite up to 3/8" thick and with a compact silty fine sand up to 3/8" thick. 25.7': Irregular, rough, limonite stained joint about 30° to core axis.

EXPLANATION

*Leakage noted on d/s side of grout cap.
** Leakage noted on d/s and u/s side of grout cap.

CORE LOSS
CORE RECOVERY

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) .. Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1887 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 2

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-620 LOCATION Right Abutment Keyway* GROUND ELEV. 5189.0 DIP (ANGLE FROM HORIZ.) 46°

COORDS. N. E.

BEGUN 11-13-76 FINISHED 11-15-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 100.0' BEARING N68°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment

S&H air drill.

Driller

R. Able

Drilling Methods

Drilled with NxHw core barrel using clear water.

Water Return

0'-100.0' - 100%

Drilling Conditions

0.0'-100.0' -Smooth

Hole Completion

Hole was photographed by downhole camera and then backfilled with grout.

Purpose of Hole

Test effectiveness of grout curtain.

TYPE AND SIZE OF HOLE

D
Nx

CORE RECOVERY (%)

96, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100

PERCOLATION TESTS

Tests from 4.4' to 34.2' were made with single mechanical packer.**

Hole sealed	Depth From (P, Cs, or Cm)	Depth To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)
P	4.4	14.4	0.1	10	10
P	14.4	24.4	0.1	10	10
P	24.4	34.2	0.1	10	10
P	33.0	38.0	0.04	10	5
P	38.0	43.0	0.02	10	5
P	43.0	48.0	0.0	10	5
P	48.0	53.0	0.0	10	5
P	53.0	58.0	0.0	10	5
P	58.0	63.0	0.0	10	5
P	63.0	68.0	0.0	10	5
P	68.0	73.0	0.0	10	5
P	73.0	78.0	0.0	10	5
P	78.0	83.0	0.0	10	5
P	83.0	88.0	0.0	10	5
P	88.0	93.0	0.1	10	5
P	93.0	98.0	0.0	10	5

ELEVATION (FEET) / DEPTH (FEET)

Elevation	Depth
5186.1	4.0
5175.3	19.0
5117.1	100.0

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-4.0': CONCRETE (Grout Cap); hard, broken in 2 places by mechanical breaks. Fairly good bond in irregular contact at 4.0'.

4.0'-19'±: WELDED ASH-FLOW TUFF; light to medium gray with local limonite staining, hard (requires moderate to hammer blow to break and heavy knife pressure to scratch), slightly porphyritic, non vesicular. Lightly to Locally-Moderately Jointed. Joint spacing is from 0.3' to 4.5', averages 1.8', joint orientation appears to be random.

Prominent joints:

7.4': Rough, irregular, hematite stained joint about 40° to core axis, filled with 1/4" of soft brown silt.

7.8': Smooth, planar, hematite stained joint about 20° to core axis, filled with 1/8" soft brown silt, stained in places with grout.

9.0': Smooth, planar, hematite stained joint about 30° to core axis.

19'±-100.0': WELDED ASH-FLOW TUFF; medium to dark gray with scattered hematite staining in joints and vugs, hard (requires moderate hammer blow to break, and moderate knife pressure to scratch), slightly porphyritic and vesicular. Foliation is very prominent (eutaxitic) and is caused mostly by flattened, wavy, streaky 1/4" wide bands of pumice and some flattened vesicles that cross the core at an angle of 40° to 60°. Most jointing is planar and smooth to rough; the more prominent joints cross the foliation. Moderately Jointed. Joint spacing varies from 0.1' to 2.0', averaging about 0.5'.

Prominent joints:

20.7': Rough, irregular, hematite stained joint about 40° to core axis (parallel to foliation), filled with calcite 1/8" to 1/2" thick.

24.9': Smooth, planar, heavy limonite and hematite stained joint 40° to core axis, parallel to the foliation.

25.2': Smooth, planar joint 40° to core axis (and nearly perpendicular to foliation), partially filled with calcite 1/8" thick.

EXPLANATION

CORE LOSS

CORE RECOVERY

*LOCATION: Sta. 13+46, offset 1.4' d/s from ℄ grout cap.

**PERCOLATION TESTS: 33.0' to 98.0' were made with inflatable straddle packers.

Type of hole .... D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) .. Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 2 HOLE NO. DH-620

GPO 884-625

7-1887 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 2 OF 2

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-620 LOCATION Right Abutment Keyway* GROUND ELEV. 5189.0 DIP (ANGLE FROM HORIZ.) 46°

COORDS. N. E.

BEGUN 11-13-76 FINISHED 11-15-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 100.0' BEARING N68°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
												25.6'-26.5': Smooth, planar joint 25° to core axis (near normal to foliation), containing a light grout stain and a heavy manganese and hematite stain. 26.3': Smooth, planar hematite stained joint about 40° to core axis (sub parallel to foliation) containing calcite 1/16" thick. 41.4': Smooth, planar joint about 35° to core axis (near normal to foliation) filled with fairly soft, brown silt 3/8" thick with some grout stain in the silt. 42.0': Smooth, planar joint parallel to joint at 41.4' filled with 1/4" fairly soft brown silt, calcite 1/16" thick occurs on lower joint surface, grout stain is present on parts of the joint surface. 48.6'-50.2': Smooth, offset joint about 10° to core axis. Joint is offset 1/2" along the foliation at 49.6'. Joint is filled with grout 1/4" to 3/8" thick, paper-thin calcite occurs along joint surfaces and soft brown silt up to 1/4" thick occurs in scattered areas in the joint. 79.0': Irregular, rough joint (broken into several pieces by drilling) which is 10° to 20° to core axis, filled with up to 1/4" of fairly soft silt. Grout stains some of the silt. 91.6': Smooth, planar joint about 35° to core axis (near normal to foliation) that is filled with 1/4" of fairly soft, brown silt. Calcite 1/8" to 1/4" coats the lower joint surface.

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90

CORE LOSS / CORE RECOVERY

EXPLANATION

*LOCATION: Sta. 13+46, offset 1.4' d/s from ℄ grout cap.

**PERCOLATION TESTS: From 33.0' to 98.0' were made with inflatable straddle packers.

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 2 OF 2 HOLE NO DH-620

GPO 854-625

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 2

FEATURE: Teton Dam PROJECT: Teton Basin STATE: Idaho
HOLE NO.: DH-621 LOCATION: Rt. Keyway, Sta. 13+47 Offset 1.4' d/s ℄ Grout Cap. COORDS. N. E. GROUND ELEV.: 5189.0' DIP (ANGLE FROM HORIZ.): 68°
BEGUN: 11/12/76 FINISHED: 11/13/76 DEPTH OF OVERBURDEN: 0.0' TOTAL DEPTH: 23.5' BEARING: N68°W
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED: Not Measured LOGGED BY: D. N. Magleby LOG REVIEWED BY: B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment

S & H air drill.

Driller

R. Abel

Drilling Methods

Drilled with clear water using NwD-3 split-tube core barrel.

Water Return

0-9.7': 100%
9.7'-14.0': 95%
14.0'-23.5': 80%

Drilling Conditions

0.0'-23.5': Smooth

Hole Completion

Backfilled to surface with grout.

Purpose of Hole

Test effectiveness of grout curtain.

TYPE AND SIZE OF HOLE / CORE RECOVERY

Type and size of hole: Nx D

Core Recovery (%)
93
96
100
100
100
100

PERCOLATION TESTS

Tests were made with single mechanical packer.

Depth (feet) From (P, Cs, or Cm)	To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)
P 3.5'	13.5'	5.1*	10	20
P 13.5'	23.5'	0.1	10	20
P 3.5'	23.5'	7.9**	10	0

Elevation (feet)	Depth (feet)
5185.8	3.5
5167.2	23.5

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-3.5': CONCRETE (Grout Cap); hard, mechanical breaks 0.3' to 1.0' apart, tight contact with underlying rock. Contact is smooth, planar, and about 45° to core axis.

3.5'-23.5': WELDED ASH-FLOW TUFF; hard (breaks with moderate to sharp hammer blow and scratches with heavy knife pressure). Moderately to Intensely Jointed. Joint spacing varies from core fragments 1/2" to core lengths 1.2', average core length 0.5', joint orientation appears to be random, but many are about 80° to core axis, planar, rough to smooth and hematite stained. Foliation caused by streaky, wavy pumice from 80° to near parallel to the core is very faint from 3.5' to 13.5' and faint from 13.5' to 23.5'. Color is medium gray 3.5' to 10'± and reddish gray 10'± to 23.5'. The jointing is listed below:

- 3.6': Broken zone with scattered chalky grout stains. Some grout stains on broken joint which is smooth, planar and 45° to core.
- 4.0'-4.8': Several parallel joints 1/4" to 1/2" apart, 20° to core axis, stained in places with chalky grout.
- 6.0': Smooth, planar joint 40° to core axis, filled with calcite 1/16" thick, stained with grout.
- 6.3'-8.3': Wavy, slightly rough joint nearly parallel to core, containing calcite to 1/8" and grout from stain to 1/4" thick with some silt staining.
- 9.5': Rough, irregular grout stained joint 45° to core axis.
- 12.0'-12.7': Smooth, planar, hematite stained joint 25° to core axis filled with soft brown silt up to 1/8" thick and stained in some places with grout.
- 12.7': Smooth, planar, hematite stained joint 55° to core axis containing 1/8" grout in some places.
- 14.6': Smooth, planar joint 65° to core axis (parallel to foliation) filled with soft brown silt up to 1/4". Grout stain is present on some joint surfaces.
- 14.7': Planar, rough joint 60° to core axis filled with 3/8" grout. Joint is parallel to foliation and is stained in places with silt.

EXPLANATION

CORE LOSS
CORE RECOVERY

*Surface leak d/s grout cap.
**Water leaking out of DH-619, Sta. 13+30.7.

Type of hole . . . . D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 2 OF 2

FEATURE Teton Oam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-621 LOCATION Rt. Keyway, Sta. 13+47 Offset 1.4' d/s ℄ Grout Cap
COORDS. N. ... E. ... GROUND ELEV. 5189.0' DIP (ANGLE FROM HORIZ.) 68°
BEGUN 11/12/76 FINISHED 11/13/76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 23.5' BEARING N68°W
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
												18.4': Wavy, smooth, hematite and grout stained joint about 60° to core axis (parallel to foliation). 19.3': Rough, irregular joint about 80° to core axis (near parallel to foliation), with calcite up to 1/8" thick. 20.2'-20.7': Smooth, planar, hematite, silt, calcite, and grout stained joint about 20° to core axis. 20.9': Smooth, planar, hematite, calcite and grout stained joint. 22.2': Smooth, planar, hematite stained joint about 55° to core axis.

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90

EXPLANATION

*Surface leak d/s grout cap.
**Water leaking out of DH-619, Sta. 13+30.7.

CORE LOSS / CORE RECOVERY

Type of hole . . . D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO DH-622 LOCATION Right Abutment Keyway* COORDS. N. E. GROUND ELEV 5189.0 DIP (ANGLE FROM HORIZ.) 90°

BEGUN 11-11-76 FINISHED 11-12-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 23.5' BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	CLASSIFICATION AND PHYSICAL CONDITION
	D Nx	89								0.0'-3.6': CONCRETE (Grout Cap); hard, broken into 0.3' to 1.1' lengths by drilling. Tight contact with underlying rock is smooth, fairly planar and nearly parallel to core.
		100	P 2.1	13.5'	1.9	10	20	5186.2	3.6	
		89								
		100								
		100								
		100								
		79								
		100	P 13.5'	23.5'	0	10	5			3.6'-23.5': WELDED ASH-FLOW TUFF; medium gray with scattered limonite (yellow) staining in some joints, fairly hard (moderate hammer blow will break and heavy knife pressure will scratch), slightly porphyritic, scattered vesicles up to 1/4". <u>Moderately to Intensely Jointed</u>. Joint spacing is from core fragments up to lengths of core of 1.0', average core length outside of the highly broken areas is 0.4', most jointing is parallel to the foliation. Faint foliation is caused by flattened vesicles and some streaky, wavy, flattened pumice that cross the core at an angle of 70° to 80°.
		100						5165.5	23.5	

Drill Equipment

S&H air operated drill.

Driller

R. Able

Drilling Methods

Drilled with clear water using NwD-3 split tube core barrel.

Water Return

0.0'-23.5' - 95% to 100%.

Hole Completion

Backfilled with grout.

Purpose of Hole

Test permeability of grout curtain.

Tests were made using single mechanical packer.

Prominent broken zones and joints:
- 3.4'-7.4': Broken apparently by drilling into core fragments 1/2" to 2".
- 12.4': Rough, irregular, iron stained joint 80° to core axis.
- 12.5': Smooth, planar iron stained joint 30° to core axis.
- 12.6': Rough, irregular, iron stained joint about 80° to core axis.
- 13.0': Smooth, iron stained joint 35° to core axis.
- 13.4': Planar, rough calcite and grout stained joint about 80° to core axis.
- 13.4'-13.5': Grout stains in smooth planar joint nearly parallel to core axis.
- 15.1': Planar to wavy, iron stained joint.
- 16.0'-23.5': Joints 0.3' to 0.5' apart parallel to foliation (about 80° to core axis), mostly rough, planar with some limonite staining in joints.

EXPLANATION

* LOCATION: Sta. 13+48, offset 1.4' d/s ℄ grout cap.

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO DH-622

GPO 854-020

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-623 LOCATION Right Abutment Keyway; Sta.13+49, offset 1.4' d/s ℄ Grout Cap. GROUND ELEV. 5189.0 DIP (ANGLE FROM HORIZ.) 68°

COORDS. N. E.

BEGUN 11-16-76 FINISHED 11-17-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 21.4' BEARING S68°E

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
Drill Equipment S&H air operated drill. Driller D. Jess Drilling Methods Drilled with clear water using NwD-3 split tube core barrel. Water Return 0.0'-2.4' - 50% return, other 50% running out of cracks in rock below drill. 2.4'-21.4' - 0% return, 100% of drill water return running out of cracks in rock downslope of drill. Hole Completion Backfilled with grout. Purpose of Hole Test permeability of grout curtain.	D Nx	100 100 100 100 100	0.0' 3.4' 6.4' 11.4' Tests were made using single mechanical packer.	13.4' 13.4' 21.4' 21.4'	14.1* 12.7* 1.6 1.2**	Gravity 10 10 10	20 20 20 20	5187.7 5169.2	1.4 21.4'			0.0'-1.4': CONCRETE (Grout Cap); hard, tight planar, smooth contact 45° to core axis at 1.4'. 1.4'-21.4': WELDED ASH-FLOW TUFF; medium gray with limonite staining 1.4' to 2.5' and hematite staining 2.5' to 21.4', hard (requires moderate hammer blow to break and heavy knife pressure to scratch), slightly porphyritic, scattered vesicles up to 1/4". Moderately Jointed. Joint spacing is 0.1' to 1.6', averaging 0.6'. Foliation is fairly prominent with streaky, wavy flattened pumice and some flattened vesicles about 60° to core axis. Most jointing is planar, smooth to rough and parallel to the foliation. Prominent joints: 1.8': Smooth, planar joint 60° to core axis. 3.2': Smooth, planar joint 70° to core axis. 5.3': Smooth, planar, hematite stained joint 50° to core axis. 5.8': Rough, irregular, hematite stained joint 40° to core axis. 6.1': Smooth, planar joint 15° to core axis, heavily stained with hematite. 8.6': Rough, planar joint 40° to core axis. 11.3': Rough, planar joint about 35° to core axis. 13.3': Rough, irregular, hematite and grout stained joint about 15° to core axis. 13.8': Smooth, planar, silt and hematite and calcite stained joint about 65° to core axis. 14.8' & 15.2': Smooth, planar joint 60° to core axis. 20.6'-21.0': Rough, irregular, iron stained fracture 5°-15° to core axis.

CORE LOSS
CORE RECOVERY

EXPLANATION

* Seepage noted on d/s side of grout cap.
** Two small seeps noted on d/s side of grout cap.

Type of hole . . . D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO. DH-623

GPO 854-828

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO DH-624 LOCATION Right Abutment Keyway, Sta. 13+76 ℄ Grout Cap. GROUND ELEV 5170.0 DIP (ANGLE FROM HORIZ.) 45°

COORDS. N. E.

BEGUN 11-16-76 FINISHED 11-16-76 DEPTH OF OVERBURDEN D.D. TOTAL DEPTH 24.3' BEARING N68°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	D Nx	100	Tests were made using single mechanical packer.					5167.0	4.3			0.0'-4.3': CONCRETE (Grout Cap); hard, broken by drilling into pieces averaging 1.0' long. Contact at 4.3' is rough, irregular and uncemented.
Drill Equipment Mark IX portable drill.		97	P 2.9'	14.3'	3.5*	10	20					4.3'-24.3': WELDED ASH-FLOW TUFF: medium to dark gray, stained with limonite (yellow) 4.3' to 20' and stained with hematite (red) 20' to 24.3', hard (requires moderate to hammer blow to break and scratches with heavy knife pressure), slightly porphyritic and slightly vesicular. Foliation is fairly distinct and consists of flattened, wavy light colored pumice fragments and some flattened vesicles that cross the core at an angle of 50° to 60°. The core is Lightly to Locally-Moderately Jointed. Spacing is from 0.1' to 2.0', averaging about 1.0'. Most joints are smooth to rough, planar, limonite or hematite stained and near parallel to the foliation.
Driller A. Allen		100										Prominent joints:
		100										7.8': Smooth, planar, silt stained joint about 45° to core axis and about normal to the foliation.
		100										9.4': Smooth, planar, hematite and silt stained joint about 30° to core axis.
		98	P 14.3'	24.3'	0	10	5					12.6': Rough, planar, limonite stained joint about 45° to core axis.
		100	0.0'	24.3'	1.8	Gravity	5	5152.8	24.3			14.1': Smooth, planar joint about 50° to core axis and about parallel with the foliation.
Drilling Methods Drilled with clear water using NwD-3 split tube core barrel.												14.3': Rough, planar joint about parallel to the joint at 14.1'.
Water Return 0.0'-24.3' - 100%												23.0': Rough, planar joint about 25° to core axis and about normal to foliation containing scattered grout (some of which is soft) up to 1/8" thick and some soft, brown silt up to 1/8" thick. The grout and silt appears to be intermixed.
Hole Completion Backfilled to surface with grout.												
Purpose of Hole Test permeability of grout curtain.												

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90

CORE LOSS / CORE RECOVERY

EXPLANATION

* Leakage noted on u/s and d/s side of grout cap.

Type of hole . . . D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO. DH-624

GPO 854-625

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-625 LOCATION Right Abutment* GROUND ELEV. 5170.0 DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. E.

BEGUN 11/15/76 FINISHED 11/15/76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 26.3' BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment

Mark IX Portable Drill

Driller

A. Allen

Drilling Methods

Drilled with clear water using NwD-3 split-tube core barrel.

Water Return

0.0'-26.3' - 100%

Hole Completion

Backfilled with grout (dyed red).

Purpose of Hole

Test permeability of grout curtain.

TYPE AND SIZE OF HOLE: DNx

CORE RECOVERY (%): 100 100 100 100 100 100 100 100 100 100 100 100 100 100

PERCOLATION TESTS — Tests made with single mechanical packer.

From (P, Cs, or Cm)	To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)
P 3.9	15.3	0.4	10	20
P 15.3	26.3	0.0	10	5
0.0	26.3	0.2	Gravity	5

Elevation (Feet)	Depth (Feet)
5164.7	5.3
5143.7	26.3

GRAPHIC LOG

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-5.3': CONCRETE (Grout Cap); hard, broken by drilling into pieces averaging 1.0' long, broken at contact with underlying rock, apparently by drilling.

5.3'-26.3': WELDED ASH-FLOW TUFF; medium gray, streaked with limonite (yellow) in joints and vugs, hard (breaks with moderate hammer blow and scratches with heavy knife pressure), fairly strong foliation about normal to core caused by wavy, streaky bands of pumice and some elongated vesicles. Moderately to Intensely Jointed. Joint spacing varies from 0.2' to 1.0' averages 0.4', nearly all joints are parallel to the foliation and are rough to smooth, planar and stained with limonite. Core contains some scattered lapilli up to 3/4" across. Core contains no grout.

Only major joints of note are:

6.2': Smooth, irregular, manganese stained joint about 80° to core axis.

23.5': Smooth, irregular, iron stained joint 30° to 45° to core axis.

*Location: Sta. 13+76 ℄ grout cap.

EXPLANATION

CORE LOSS / CORE RECOVERY

Type of hole . . . D = Diamond, N = Haystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO. DH-625

GPO 854-625

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-626 LOCATION Sta. 13+77.3 ℄ Grout Cap* GROUND ELEV. 5169.3' DIP (ANGLE FROM HORIZ.) 68°

COORDS. N. ... E. ...

BEGUN 11/17/76 FINISHED 11/22/76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 26.7' BEARING S68°E

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	CLASSIFICATION AND PHYSICAL CONDITION
Drill Equipment	D Nx	100, 100, 100, 100, 100	Tests were made using single mechanical packer.							0.0'-6.7': CONCRETE (Grout cap); hard, broken by drilling into 1-foot lengths. Smooth planar contact at 6.7' is uncemented.
Mark 1X portable diamond drill.		100						5163.1	6.7	6.7'-26.7': WELDED ASH-FLOW TUFF; medium gray, slightly vesicular, slightly porphyritic, fairly hard (requires moderate hammer blow to break and moderate knife pressure to scratch), prominent foliation caused by flattened wavy pumice fragments and flattened, elongated vesicles (some of which are iron stained) that cross the core at an angle of 50°-60°. Moderately to Intensely Jointed. Joint spacing varies from 1/2" fragments of core up to 1.4' core lengths, average core length 0.4'. Most joints are parallel to the foliation, mostly planar, rough, hematite stained and 60° to core axis. About 80% of the joints are rough, about 20% are smooth. The only major joint of note is at 9.2', the joint apparently at which the drill water return was lost.
Driller		100	P 5.3	16.7	12.5**	10	20			9.2': Smooth, planar, manganese stained joint about 35° to core axis filled with 1/8" thick grout apparently intermixed with some soft, brown silt.
A. Allen		100								
Drilling Methods		100								
Drilled with clear water using NwD-3 split tube core barrel.		100	P 16.7	26.7	7.5	2	5			
		100	0	26.7	4.5**	Gravity	5	5144.5	26.7	
Water Return										
0.0'-9.0': 100%										
9.0'-26.7': 5%										
Hole Completion										
Hole left open for future borehole camera. Wooden plug put in top of hole.										
Purpose of Hole										
Test permeability of grout curtain.										

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90

EXPLANATION

*Right abutment

**Leakage from vertical joints on d/s side of grout curtain at Sta. 13+97.

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, N = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series).. Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO. DH-626

GPO 854-828

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-627 LOCATION Right Abutment* COORDS. N. E. GROUND ELEV. 5149.0' DIP (ANGLE FROM HORIZ.) 44°

BEGUN 11/24/76 FINISHED 11/24/76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 21.0' BEARING N68°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not measured LOGGED BY D. N. Magleby LOG REVIEWED BY S. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
Drill Equipment Mark 1X portable diamond drill. **Driller** A. Allen **Drilling Methods** Drilled with clear water using NxD-3 split tube core barrel. **Water Return** 0.0'-21.0': 100% **Hole Completion** Hole left open for future borehole camera. Wooden plug put in top of hole. **Purpose of Hole** Test permeability of grout curtain.	D Nx	100, 100, 100, 100, 100, 100, 100, 100, 100	Tests were made using single mechanical packer. P 1.6	11.0	0.2	10	20					0.0'-21.0': WELDED ASH-FLOW TUFF; medium gray (stained with hematite red 8'± to 11.8'±), slightly porphyritic and vesicular, fairly hard (requires moderate hammer blow to break and scratches with moderate knife pressure), prominent foliation caused mainly by flattened elongated vesicles and some streaky flattened pumice fragments that cross the core at an angle of about 50°. Moderately to Locally Intensely Jointed. Joint spacing is from 1/2" fragments of core up to core lengths 1.7', average 0.8'. Most joints are parallel to the foliation, rough, irregular, hematite and manganese stained. Prominent joints: 8.9': Planar, slightly rough, hematite stained joint about 35° to core axis. Rock is bleached about 1/2" each side of joint. 11.8': Planar, rough, hematite stained joint about 45° to core axis (parallel to foliation), filled with fairly soft brown silt up to 1/4" thick. 15.1': Planar, rough, manganese and grout stained joint about 45° to core axis. 19.4': Planar, fairly smooth, hematite stained joint about 25° to core axis. Joint is filled with 3/8" thick soft brown silt stained in some places with grout. Calcite coats some of the joint surfaces.
			P 11.0	21.0	0.1	10	20					
			0.0	21.0	0.0	Gravity	5	5134.5	21.0			

EXPLANATION

*Sta. 14+08.5 ℄ Grout Cap

CORE LOSS
CORE RECOVERY

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO. DH-627

GPO 854-625

7-1337 (8-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-628 LOCATION Right Abutment Keyway* GROUND ELEV. 5149.0' DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. ... E. ...

BEGUN 11/23/76 FINISHED 11/23/76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 21.0' BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
Drill Equipment Mark 1X portable diamond drill.	D Nx	100 100 100 100 100 100 100 100	P 1.0	11.0	Packer seal.	would	not					0.0'-21.0': WELDED ASH-FLOW TUFF; medium gray, slightly porphyritic, slightly vesicular, fairly hard (requires moderate hammer blow to break, scratches with moderate knife pressure), strong foliation about normal to core caused mostly by thin bands of flattened elongated vesicles and some flattened pumice fragments. Moderately to Intensely Jointed. Joint spacing is from fragments 0.1' to 0.9', average 0.4'. Most joints are fairly planar, rough and parallel to the foliation, some joints are manganese and hematite stained.
Driller A. Allen		100	P 4.6	11.0	0	10	5					Prominent joints: 0.8': Rough, irregular, hematite and silt stained joint about 15° to core axis. 1.1': Broken zone about 0.2' wide with some silt stain. 7.3': Smooth, planar joint about normal to core stained with soft chalky grout. 15.2'-16.2': Planar, rough, iron stained joint about 10° to core axis.
Drilling Methods Drilled using clear water and NWD-3 split tube core barrel.		100 100	P 11.0	21.0	0.1	10	20					
Water Return 0.0'-21.0': 100%		100	0.0	21.0	0.1	Gravity	5	5128.0	21.0			
Hole Completion Backfilled with grout.			Tests were made using single mechanical packer.									
Purpose of Hole Test permeability of grout curtain.												

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90

EXPLANATION

CORE LOSS / CORE RECOVERY

*Sta. 14+09.7 ℄ Grout Cap

Type of hole .... D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) .. Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO. DH-628

GPO 854-625

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-629 LOCATION Right Abutment, Sta. 14+07.5 offset 1.1' d/s ℄ Grout Cap GROUND ELEV. 5149.6 DIP (ANGLE FROM HORIZ.) 56°

COORDS. N. E.

BEGUN 11-26-76 FINISHED 11-26-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 21.0' BEARING S68°E

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	D Nx	100, 100, 100, 100	Tests were made using P single mechanical packer.									0.0'-21.0': WELDED ASH-FLOW TUFF; medium gray, slightly porphyritic, slightly vesicular, fairly hard (requires moderate hammer blow to break, scratches with moderate knife pressure). Prominent foliation caused by flattened and elongated vesicles and some flattened pumice fragments cross the core at an angle of 50° to 60° to the core axis.
Drill Equipment		100	1.0 P	11.0	7.1*	10	20					Moderately to Intensely Jointed. Joint spacing is from 0.1' to 1.2', averaging 0.4'. Most jointing is parallel to foliation, planar, rough and some stained with iron and manganese.
Mark IX portable diamond drill.		100	2.6	11.0	2.8*	10	20					Prominent joints:
Driller		100	P									2.3': Smooth, planar, grout and hematite stained joint about 30° to core axis.
A. Allen		100	11.0	21.0	0.7	10	20					15.8': Smooth, planar joint about 30° to core axis (near normal to foliation), with heavy manganese stain.
		100	0.0	21.0	0.3	Gravity	5	5131.6	21.0			17.6': Rough, irregular, manganese stained joint about 30° to core axis filled with soft brown silt up to 1/4" thick.
Drilling Methods												18.4': Planar, rough joint about 60° to core axis (parallel to foliation), stained with iron and some silt.
Drilled with clear water using NwD-3 split tube core barrel.												20.8': Similar to joint at 18.4'.
Water Return												
0.0'-21.0' - 100%												
Hole Completion												
Hole left open for future borehole camera. Installed wooden plug in top of hole.												
Purpose of Hole												
Test permeability of grout curtain.												

EXPLANATION

CORE LOSS

CORE RECOVERY

* Leakage from vertical and horizontal joints near DH-629 and some leakage from DH-628, Sta. 14+09.7.

Type of hole . . . D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Hx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Hx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Hx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO. DH-629

GPO 854-625

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-630 LOCATION Right Abutment, Sta. 14+27.0 ₵ Grout Cap COORDS. N. ... E. ... GROUND ELEV. 5141.3 DIP (ANGLE FROM HORIZ.) 43°

BEGUN 11-20-76 FINISHED 11-20-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 21.7' BEARING N68°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	CLASSIFICATION AND PHYSICAL CONDITION
		100, 100, 100, 100, 100	Tests were made using single mechanical packer					5140.1	1.7	0.0'-1.7': CONCRETE (Grout Cap); hard, contact with underlying rock is planar, rough, tight, 60° to core axis.
Drill Equipment Mark IX portable diamond drill.	D Nx	100	P 1.3	11.7	4.3*	10	20			1.7'-21.7': WELDED ASH-FLOW TUFF; medium gray, slightly porphyritic, slightly vesicular, fairly hard (breaks with moderate hammer blow, scratches with moderate knife pressure). Fairly strong foliation caused mostly by bands of flattened elongated vesicles that cross the core at an angle of about 60°. Moderately to Intensely Jointed. Joint spacing varies from 0.1' to 2.0', averaging 0.8'. About 80% of the joints (partings) are parallel to the foliation and are mostly rough, planar and iron and manganese stained. Prominent joints: 9.9': Smooth, planar, iron, manganese and silt stained joint about 30° to core axis (nearly normal to foliation). 10.7': Planar, slightly rough joint stained with iron, manganese and silt, 30° to core axis (parallel to joint at 9.9'). 21.6': Smooth, planar joint about 30° to core axis (near normal to foliation) stained with iron, calcite and silt.
Driller A. Allen		100, 100	P 11.7	21.7	0.4	10	20			
		100	0.0	21.7	0.9	Gravity	5	5126.5	21.7	
Method of Drilling Drilled with clear water using NwD-3 split tube core barrel.										
Water Return 0.0'-21.7' - 100%										
Hole Completion Left open for future borehole camera. Put wooden plug in top of hole.										
Purpose of Hole Test permeability of grout curtain.										

EXPLANATION

CORE LOSS / CORE RECOVERY

*Leakage from vertical joints u/s and d/s of grout cap.

Type of hole: D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed: P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series): Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series): Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series): Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series): Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO. DH-630

GPO 884-628

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-631 LOCATION Right Abutment, Sta. 14+26 ℄ Grout Cap GROUND ELEV 5141.6 DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. E.

BEGUN 11-18-76 FINISHED 11-18-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 21.6' BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY R. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment

Mark IX portable diamond drill.

Driller

A. Allen

Method of Drilling

Drilled with clear water using NwD-3 split tube core barrel.

Water Return

0.0'-21.6' - 100%

Hole Completion

Backfilled hole with colored grout.

Purpose of Hole

Test permeability of grout curtain.

TYPE AND SIZE OF HOLE

D Nx

CORE RECOVERY (%)

100, 100, 100, 100, 100, 100, 100, 100, 100, 100

PERCOLATION TESTS

Tests were made using single mechanical packer.

Depth (feet) From (P, Cs, or Cm)	To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)
P 1.2	11.6	0.3	10	20
P 11.6	21.6	0.2	10	20
0.0	21.6	0.0	Gravity	5

Elevation (feet)	Depth (feet)
5140.0	1.6
5120.0	21.6

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-1.6': CONCRETE (Grout Cap); hard, contact at 1.6' is planar, smooth, tight and normal to the core.

1.6'-21.6': WELDED ASH-FLOW TUFF; medium gray, slightly porphyritic, slightly vesicular, fairly hard (breaks with moderate hammer blow, scratches with moderate knife pressure). Fairly strong foliation about normal to core caused mostly by flattened and elongated vesicles and some flattened fragments of pumice. Moderately to Intensely Jointed. Joint spacing varies from 0.1' to 1.2' averaging 0.4'. Most of the jointing, parallel to the foliation, is planar, rough and stained some with iron and manganese.

Prominent joints:

2.1': Irregular, rough, grout stained joint 10°-20° to core axis.

2.3': Planar, rough, grout stained joint about normal to core axis.

2.4': Joint similar to joint at 2.3'.

10.3': Smooth, planar, iron stained joint about 85° to core axis.

10.5': Same as joint at 10.3'.

11.3': Same as joint at 10.3'.

13.1': Smooth, offset (1/4"), iron stained joint about 30° to core axis.

EXPLANATION

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO. DH-631

GPO 854-628

7-1337 (8-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 1

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO DH-632 LOCATION Right Abutment, Sta. 14+27.8 ℄ Grout Cap. GROUND ELEV 5141.3' DIP (ANGLE FROM HORIZ.) 56°

COORDS. N. E.

BEGUN 11-19-76 FINISHED 11-19-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 23.1' BEARING S68°E

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY R. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G P M)	PRESSURE (P S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
Drill Equipment Mark IX Portable diamond drill. Driller A. Allen Method of Drilling Drilled with clear water using NwO-3 split tube core barrel. Water Return 0.0'-23.1' - 100% Hole Completion Backfilled with grout. Purpose of Hole Test permeability of grout curtain.	D Nx	100, 100, 100, 100, 100, 100, 100, 100, 100, 100	P 1.7 P 13.1 0.0	13.1 23.1 23.1	2.6* 1.6 0.7	10 10 Gravity	20 20 5	5138.7 5122.2	3.1 23.1			0.0'-3.1': CONCRETE (Grout Cap); hard, broken by drilling into pieces from 0.1' to 0.9'. Contact with underlying rock is planar, smooth, tight and about 45° to core axis. 3.1'-23.1': WELDED ASH-FLOW TUFF; medium gray, slightly porphyritic, slightly vesicular, fairly hard (breaks with moderate hammer blow and scratches with moderate knife pressure). Fairly strong foliation 45° to 50° to core axis is the result mostly of flattened elongated vesicles and some flattened pumice fragments. Moderately to Intensely Jointed. Joint spacing varies from 0.1' to 1.4', averaging 0.6'. Most joints are planar, rough, iron and manganese stained and parallel to the foliation. Prominent joints: 13.6': Planar, rough, iron stained joint about 40° to core axis (about parallel to the foliation). 14.4': Rough, irregular, iron stained joint about 20° to core axis (crosses foliation).

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90

EXPLANATION

CORE LOSS

CORE RECOVERY

*Small leak on u/s side of grout cap at Sta. 14+41.

Type of hole . . . D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 1 HOLE NO DH-632

GPO 854-825

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 2

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-633 LOCATION Right Abutment, Sta. 13+43.5 offset 13.5' u/s ℄ Grout Cap GROUND ELEV. 5190.0 DIP (ANGLE FROM HORIZ.) 70°
COORDS. N. E.
BEGUN 11-19-76 FINISHED 11-23-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 90.0' BEARING S15°E
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment

S&H air operated drill.

Driller

R. Able

Method of Drilling

Drilled with clear water and NxHW core barrel.

Water Return

0.0'-77.0' - 100%
77.0'-90.0': 0%
Below 77.0' return water seeping from vertical joint located 17' u/s from grout cap, station 13+35.

Drilling Conditions

0.0'-90.0'- Smooth.

Hole Completion

Left hole open for future bore-hole camera. Put wooden plug in top of hole.

Purpose of Hole

Test permeability of grout curtain.

TYPE AND SIZE OF HOLE: D Nx

CORE RECOVERY (%): 100, 100, 100, 98, 100, 98, 100, 98, 91, 100, 100, 100, 92

PERCOLATION TESTS

Tests were made using pneumatic straddle packers.

From (P, Cs, or Cm)	To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)
P 5.0	10.0	0.1	10	5
P 10.0	15.0	0.9	10	10
P 15.0	20.0	0.4	10	10
P 20.0	25.0	0.0	10	5
P 25.0	30.0	0.0	10	5
P 30.0	35.0	0.0	10	5
P 35.0	40.0	0.0	10	5
P 40.0	45.0	0.0	10	5
P 45.0	50.0	0.0	10	5
P 50.0	55.0	18.3	10	10
P 55.0	60.0	17.3	10	10
P 60.0	65.0	0.4	10	10
P 65.0	70.0	0.1	10	5
P 70.0	75.0	0.1	10	10
P 75.0	80.0	0.1	10	5
0.0	90.0	27.2*	Gravity	20

ELEVATION (FEET): 5105.4 (at 90')

DEPTH (FEET): 10, 20, 30, 40, 50, 60, 70, 80, 90

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-90.0': WELDED ASH-FLOW TUFF; medium gray (stained with hematite red 40'± to 57'±) slightly porphyritic, slightly vesicular, fairly hard (requires moderate hammer blow to break and moderate knife pressure to scratch). Foliation is faint to distinct and is the result of flattened, elongated vesicles and flattened pumice fragments that cross the core at an angle of 60° to 80°. Moderately to Intensely Jointed. Joint spacing is from 0.1' to 1.5' averaging 0.4'. About 80% of the jointing parallels the foliation and is planar, smooth to rough and mostly stained with iron and manganese.

Prominent joints:

0.5': Smooth, planar, manganese stained joint about 40° to core axis.
6.3': Smooth, planar, manganese stained joint about 40° to core axis.
6.4': Same as joint at 6.3' except for some grout staining in the joint.
6.9': Same as joint at 6.3'.
8.1': Very rough and irregular iron stained joint about 35° to core axis.
9.6': Smooth, planar, manganese stained joint about 45° to core axis.
9.8': Same as joint at 9.6'.
10.3': Same as joint at 9.6'.
10.8': Rough, irregular iron stained joint about 30° to core axis, near normal to joint at 10.3'.
13.0': Smooth, planar, manganese stained joint about 30° to core axis.
13.9': Same as joint at 13.0'.
18.5' to 20.0': Smooth, planar, manganese stained joint nearly parallel to core.
20.4': Smooth, planar, manganese and calcite stained joint about 40° to core axis.
20.6' to 22.0': Smooth, planar joint nearly parallel to core stained with calcite and a pink grout stain.
30.8': Irregular, smooth, iron and manganese stained joint about 40° to core axis.

EXPLANATION

CORE LOSS / CORE RECOVERY

*Unable to raise water level to top of hole.

Type of hole . . . D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 2 OF 2

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-633 LOCATION Right Abutment, Sta. 13+43.5 offset 13.5' u/s ℄ Grout Cap. GROUND ELEV 5190.0 DIP (ANGLE FROM HORIZ.) 70°

COORDS. N. E.

BEGUN 11-19-76 FINISHED 11-23-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 90.0' BEARING S15°E

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	10 20 30 40 50 60 70 80 90								10 20 30 40 50 60 70 80 90			38.6'-40.3': Irregular, rough joint nearly parallel to core stained with hematite, calcite and some silt, grout partially filling joint up to 1/8" thick. 54.6': Fairly smooth, planar, iron and silt stained joint about 20° to core axis. 77.0': Smooth, planar silt stained joint about 10° to 20° to core axis. 77.5': Similar and parallel to joint at 77.0'. 78.3': Similar and sub parallel to joint at 77.5'. 79.0': Similar and parallel to joint at 78.3'. 87.0': Planar, fairly rough, iron and silt stained joint about 65° to core axis.

EXPLANATION

CORE LOSS
CORE RECOVERY

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series).. Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 2 OF 2 HOLE NO. DH-633

GPO 854-628

GEOLOGIC LOG OF DRILL HOLE

FEATURE: Teton Dam — PROJECT: Teton Basin — STATE: Idaho

HOLE NO. DH-634 — LOCATION: Right Abutment* — COORDS. N. .... E. .... — GROUND ELEV. 5188.7 — DIP (ANGLE FROM HORIZ.) 70°

BEGUN 12-4-76 — FINISHED 12-10-76 — DEPTH OF OVERBURDEN 0.0' — TOTAL DEPTH 91.0' — BEARING N 5° W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED: Not measured — LOGGED BY: D. N. Magleby — LOG REVIEWED BY: B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment
S&H air operated diamond drill

Driller
R. Able

Drilling Method
Drilled with clear water using NWO-3 split tube core barrel

Drilling Conditions
Mostly smooth

Water Return
0.0'-28.4' - 100%
28.4'-57.1' - 90%
57.1'-91.0' - 80%

Hole Completion
Hole left open for future borehole camera. Inserted wooden plug in top of hole.

Purpose of Drilling
Test effectiveness of grout curtain.

TYPE AND SIZE OF HOLE: DNx

CORE RECOVERY (%): 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 100, 95, 100, 100, 100, 100, 100, 100, 100, 100, 100

ELEVATION (FEET): 5174.6 (at 14.1'); 5103.2 (at 85.5')

PERCOLATION TESTS

Tests made with pneumatic straddle packers.

Hole sealed	Depth From (feet)	Depth To (feet)	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)
P	3.0	8.0	0	10	5
P	8.0	13.0	0	10	5
P	13.0	18.0	9.5	10	10
P	18.0	23.0	1.2	10	10
P	23.0	28.0	0	10	5
P	28.0	33.0	0	10	5
P	33.0	38.0	0	10	5
P	38.0	43.0	0	10	5
P	43.0	48.0	0.2	10	5
P	48.0	53.0	2.2	10	5
P	53.0	58.0	0	10	5
P	58.0	63.0	2.4	10	10
P	63.0	68.0	0	10	5
P	68.0	73.0	0	10	5
P	73.0	78.0	0	10	5
P	78.0	83.0	0	10	5
P	83.0	88.0	0	10	5

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-15.0'±: WELDED ASH-FLOW TUFF; medium gray 0.0' to 12'± changing to reddish gray 12'± to 15'±, hard (breaks with heavy hammer blow, scratches with heavy knife pressure), slightly porphyritic, nonvesicular, scattered lapilli up to 0.1'. Faint foliation caused mostly by flattened wavy pumice fragments is about parallel to core axis. Moderately Jointed with local areas up to 1 foot long of intensely jointed rock. Joint spacing varies from 0.1' to 2', averaging about 1'. Joints have variable orientation; however, major jointing is near parallel to core; most joints are planar, rough and stained with calcite and silt.

Major joints are as follows:

3.5'-4.6': Fairly planar, very rough, iron and silt stained joint nearly parallel to core. Core is moderately broken along the joint.

8.0': Smooth, planar, manganese stained joint orientated about 85° to core axis.

13.0': Planar, rough, silt stained joint about 40° to core axis filled with 1/8" thick calcite.

13.0'-15.0': Rough, planar joint nearly parallel to core, iron and silt stained and filled with calcite 1/4" thick. Some core at 14.5' appears to be brecciated and healed with calcite.

15.0'±-91.0': WELDED ASH-FLOW TUFF; reddish gray to gray (some joints stained hematite red), fairly hard (core breaks with moderate hammer blow and scratches with heavy knife pressure), strong foliation caused by flattened pumice fragments and flattened elongated vesicles that cross the core at an angle of 50° to 60° to core axis. Moderately Jointed. Joint spacing varies from 0.1' to 1.5', averages about 0.5'. Most joints are planar, rough and parallel to foliation - other jointing is smooth planar and nearly normal to foliation.

Prominent joints are described below:

15.9': Smooth, planar, iron-stained joint about 35° to core axis containing one small blob (1/4" wide) of grout.

20.0': Smooth to slightly rough, planar, iron and silt stained joint about 25° to core axis.

EXPLANATION

* Sta. 13+42, offset 11.0' d/s ℄ Grout cap.

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) .. Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 2 OF 2

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO DH-634 LOCATION Right Abutment* GROUND ELEV 5188.7 DIP (ANGLE FROM HORIZ.) 79°

COORDS. N. E.

BEGUN 12-4-76 FINISHED 12-10-76 DEPTH OF OVERBURDEN 0.0' TOTAL DEPTH 91.0' BEARING N 5° W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G P M)	PRESSURE (P S I)	LENGTH OF TEST (MIN)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
												27.8': Smooth, planar, hematite and manganese stained joint about 30° to core axis - near normal to foliation.
												29.4', 29.8' and 30.1': Smooth, planar, iron and manganese stained joints about parallel to foliation.
												35.3': Smooth, planar, iron and manganese stained joint about 70° to core axis (not parallel to foliation).
												38.0': Smooth, planar, manganese stained joint parallel to foliation.
												59.3': Smooth, planar, iron and silt stained joint about 20° to core axis.
												69.8': Smooth, planar, iron and manganese stained joint about 35° to core axis.
												70.8'-71.8': Smooth, planar joint nearly parallel to core filled with 1/8" soft brown silt.

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90

CORE LOSS / CORE RECOVERY

* Sta. 13+42, offset 11.0' d/s ℄ Grout cap.

EXPLANATION

Type of hole . . . . D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed . . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 2 OF 2 HOLE NO DH-634

GPO 854-625

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 4

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-650 LOCATION Far Rt. Abutment, Sta. 3+00, 3' u/s/ ℄ Dam GROUND ELEV. 5332.4* DIP (ANGLE FROM HORIZ.) 59° **

COORDS. N. ... E. ...

BEGUN 10/6/76 FINISHED 10/16/76 DEPTH OF OVERBURDEN 90.0' TOTAL DEPTH 351.5' BEARING** S21°E

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY J. Phillips LOG REVIEWED BY D. N. Magleby

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	CLASSIFICATION AND PHYSICAL CONDITION
Drill Equipment: Mobile B-40L.	6" Auger							5328.6	5.0	0.0'-5.0': GRAVEL, COBBLES & BOULDERS; as reported by drillers. (Road Fill)
Drillers: E. Claunts and D. Jess		0								5.0'-90.0': SILT; as reported by drillers (Zone I fill in keyway trench).
Drilling Methods: Advanced hole to 50' by using 6" auger. Rock bitted with 2-15/16" bit to 90', placing and driving Nx casing as advancing hole. Cored with 10' NxD core barrel 90.0'-351.5'.	6" Auger (to 50); $2\frac{15}{16}$ RB	0								
Drilling Conditions: 0.0'-5.0': Rough and smooth; 5.0'-70.0': Smooth; 70.0'-97.6': Smooth and slow; 97.6'-104.8': Smooth and hard, hit broken seam at 102.3'; 104.8-117.6': Hard and slow; 117.6'-127.4': Slow; 127.4'-312.5': Hard and slow; 312.5'-351.5': Smooth and fast			Tests were made using single mechanical packer.							
	DNx	91	P 90.0	97.6	32.1	10	10	5255.8	90	90.0'-150'±: WELDED ASH-FLOW TUFF; light gray-purple, fairly hard (breaks with moderate hammer blow, scratches with heavy knife pressure), contains scattered pumice zones and vuggy zones, indistinct foliation, slightly porphyritic, light weight. Lightly Jointed. Joint spacing varies from less than 0.1' to 6.1', averaging 2.0'. Most joints are 45° to 60° to core axis. Prominent joint 108.4' to 111.4' is wavy, rough, near parallel to core and silt stained.

Casing and Cementing

Date	Depth	Size
10/5	70.0	Nx(Cs)
10/6	90.0	Nx(Cs)
10/6	91.6	Cm
10/8	127.4	Cm

Water Return

Date	Interval	% Return
10/5	0.0-70.0	100
10/6	70.0-90.0	100
10/6	Lost water at 91.6' - no return 91.6'-351.5'	

EXPLANATION

CORE LOSS / CORE RECOVERY

*Measuring point is El. 5332.9'

RB= Rock Bit

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 2 OF 4

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-650 LOCATION Far Rt. Abutment, Sta. 3+00, 3' u/s/ ℄ Dam COORDS. N. E. GROUND ELEV 5332.4* DIP (ANGLE FROM HORIZ.) ** 59°
BEGUN 10/6/76 FINISHED 10/16/76 DEPTH OF OVERBURDEN 90.0' TOTAL DEPTH 351.5' BEARING ** S21°E
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY J. Phillips LOG REVIEWED BY D. N. Magleby

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Water Level After Drilling

Not measured.

Hole Completion

Purpose of Hole

Test effectiveness of grout curtain.

CORE RECOVERY (%): 100, 60, 77, 97, 97, 100, 100, 100, 100, 100, 97, 100

PERCOLATION TESTS

Hole sealed	Depth (feet) From	To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of test (min.)
P	99.8	104.8	32.9	10	5
P	103.0	127.4	28.3	10	10
P	127.6	162.6	13.4	10	10
P	160.6	197.2	8.4	10	10

ELEVATION (FEET): 5204.3 (at depth 150)

DEPTH (FEET): 110, 120, 130, 140, 150, 160, 170, 180, 190

CLASSIFICATION AND PHYSICAL CONDITION

Grout was found in the following openings:
109.1'-110.3': Grout locally up to 1/16" thick in irregular, rough joint about parallel to core. Joint contains a thin stain of a silt-like material.
112.9': Smooth, planar joint 30° to core axis containing 1/4" to 3/4" of regular and sandy grout.
113.2': Grout to 1/16" locally filling planar, rough joint at 60° to core axis.
118.4': Solid grout 0.1' thick filling a planar, rough joint which is about 45° to core axis. Grout contains scattered angular rock fragments up to 1/2" across.
119.0': Grout locally up to 1/16" thick in a rough, slightly planar joint about 45° to core axis.
128.5': Grout up to 1/4" in a rough, irregular joint which is about 45° to core axis. Joint locally contains silt-like material up to 1/8" thick.
129.1': Grout stain in irregular, rough opening about 30° to core axis.
132.4': Irregular, rough joint filled with 1/8" to 3/4" thick grout. Joint is about 30° to core axis.
132.4'-133.0': Grout up to 1/8" partially filling rough, irregular joints (2 & 3 parallel joints) which are about 30° to core axis.
150.0': Grout 1/4" to 1/2" thick in a rough, near planar joint about 60° to core axis.

150'±-308'±: WELDED ASH-FLOW TUFF; fairly hard (breaks with light hammer blow and scratches with moderate knife pressure), contains scattered pumice inclusions up to 0.2'.
Joint spacing:
150.9'-172.2': 0.1' to 1.2', averaging 0.6'.
172.2'-197.2': 0.2' to 2.5', averaging 0.7'.
197.2'-296.8': 0.1' to 2.8', averaging 0.6'.
296.8'-308': 0.1' to 3.1', averaging 0.7'.

*Measuring point is El. 5332.9'

EXPLANATION

CORE LOSS
CORE RECOVERY

Type of hole . . . . . D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 3 OF 4

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-650 LOCATION Far Rt. Abutment, Sta. 3+00, 3' u/s ℄ Dam GROUND ELEV 5332.4* DIP (ANGLE FROM HORIZ.) ** 59°

COORDS. N. ______ E. ______

BEGUN 10/6/76 FINISHED 10/16/76 DEPTH OF OVERBURDEN 90.0' TOTAL DEPTH 351.5' BEARING ** S21°E

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY J. Phillips LOG REVIEWED BY D. N. Magleby

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)
		100							
		100							210
			P 197.6	232.6	0.0	10	10		
		100							220
		100							230
		100							
									240
		100							
									250
		100	P 232.5	267.5	20.3	10	10		
									260
		100							
		96							270
		100	P 267.5	302.5	6.1	10	10		280
		100							290

CLASSIFICATION AND PHYSICAL CONDITION

Most joints are planar, smooth and mostly parallel to the foliation. Foliation is fairly distinct and is caused by flattened vesicles with some flattened, wavy pumice streaks about 45° to 60° to core axis. From 150'± to 207'± color is light purple-gray, from 207'± to 308'± color of core is light gray.

Grout was found in the following openings:

150.6': Grout 1/8" to 1/4" in rough, planar joint 50° to core axis. Some calcite staining occurs in joint.

173.7': Grout 1/8" to 1/4" thick in calcite stained, fairly smooth, planar joint about 30° to core axis.

175.35': Grout up to 1/16" in smooth, planar joint about 60° to core axis.

186.9': Grout locally up to 1/8" in a fairly planar, rough, hematite and manganese stained joint about 60° to core axis.

188.5': Grout up to 1/8" thick filling a rough, planar joint about 40° to core axis.

195.2': Grout up to 1/16" filling a rough, planar joint 45° to core axis (parallel to flow lineation). Joint locally contains calcite up to 1/16" thick.

206.5': Sand grout filling a 5/8" wide smooth, planar joint about 30° to core axis.

211.0': Grout locally up to 1/16" thick in a calcite stained, irregular, rough joint about 60° to core axis.

212.5': Grout up to 1/8" in a fairly planar, rough joint about 45° to core axis. Joint contains thin coating of calcite and some brown silt-like material.

308'±-351.5': WELDED ASH-FLOW TUFF; light to dark gray (banded) can be broken with moderate hand pressure, will scratch with light knife pressure, characterized by a wavy, streaky foliate structure (eutaxitic) and some flattened vesicles which are parallel to foliate structure and are 45° to 60° to core axis. Itensely to Moderately Jointed. Joint spacing 0.1' to 3.1', averaging 0.7'. Moist joints are planar, rough and parallel to the foliation.

*Measuring point is El. 5332.9'

EXPLANATION

CORE LOSS
CORE RECOVERY

Type of hole . . . . . . D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (8-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE: Teton Dam PROJECT: Teton Basin STATE: Idaho

HOLE NO. DH-650 LOCATION: Far Rt. Abutment, Sta. 3+00, 3' u/s ℄ Dam COORDS. N. E. GROUND ELEV. 5332.4* DIP (ANGLE FROM HORIZ.) ** 59°

BEGUN 10/6/76 FINISHED 10/16/76 DEPTH OF OVERBURDEN 90.0' TOTAL DEPTH 351.5' BEARING ** S21°E

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED: Not Measured LOGGED BY: J. Phillips LOG REVIEWED BY: D. N. Magleby

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
** SPERRY SUN DOWN-HOLE SURVEY		100										Grout was found in the following openings:
		98						5068.9				237.5': Grout 3/16" to 1/4" filling a smooth, planar joint about 30° to core axis.
		100	P 301.7	331.5	2.1	10	10					330.0'-330.2': Grout up to 1/4" partially filling a planar, smooth to rough joint 25° to core axis.
		100										
		100										
		100	P 331.5	351.5	0.0	10	10					
								5031.6	351.5			

Depth	Angle with Vertical	Bearing
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	

Depth scale: 310, 320, 330, 340, 350, 360, 370, 380, 390

CORE LOSS
CORE RECOVERY

*Measuring point is El. 5332.9'

EXPLANATION

Type of hole . . . . . D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx – 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

GPO 854-625

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-651 LOCATION Far Rt. Abutment, Sta. 4+34, ℄ GROUND ELEV. 5332.1* DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. .... E. ....

BEGUN 9/28/76 FINISHED 10/12/76 DEPTH OF OVERBURDEN 80.4 TOTAL DEPTH 622.4 BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED 314.1', 5018.6', 11-15-76 LOGGED BY John Phillips LOG REVIEWED BY D. N. Magleby

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Drill Equipment

Failing 1500 rotary drill.

Drillers

H. Davis and D. Jess

Drilling Methods

Advanced hole 0.0' to 4.4' with 7-7/8" rock bit, placed 6" casing. Advanced hole to 78.9' with 2-7/8" rock bit, chopped and drove 4" casing 0.0' to 78.9'. Drilled NxHw core barrel 78.9' to 622.4'. (20 foot core barrel and clear water).

Drilling Conditions as noted by drillers

457.2'-472.4': Hard rock.
495.4'-496.7': Very hard rock
496.7'-506.5': Very hard rock with soft silty sand.
526.7'-527.2': Ground up most of core.
534.0'-543.7': Very soft material, in interval 536'-538' rock washing down faster than drilling.
550.3'-552.3': No recovery, felt like large gravels.
563.5'-566.1': No recovery, drilled like gravel.
569.7'-571.1': No recovery, drilled smooth, medium sand in water return.
600.0'-622.4': No recovery, drillers reported sand and gravel.

TYPE AND SIZE OF HOLE / CORE RECOVERY / PERCOLATION TESTS / ELEVATION / DEPTH

Type and size of hole	Core recovery (%)	From (P, Cs, or Cm)	To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of test (Min.)	Elevation (feet)	Depth (feet)
RB 7-7/8"	0							
2-7/8" RB	0						5324.8	
	0							
	90						5253.8	
NX							5252.3	80
	100	P 80.0	100.0	.02	10	10		90

Tests were made using single mechanical packer.

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-7.9': GRAVEL AND COBBLES; as reported by drillers (road fill).

7.9'-78.9': SILT; as reported by drillers (Zone I fill in keyway trench).

78.9'-80.4': CONCRETE. (Grout cap in keyway trench).

80.4'-121.0'±: WELDED ASH-FLOW TUFF; light purple gray, hard (core breaks with moderate hammer blow normal to axis and scratches with heavy knife pressure), slightly porphyritic, slightly vesicular, foliation is very faint to indistinct. Lightly Jointed. Joints are spaced from 0.3' to 7.7', mostly about 2'. Most joints are near horizontal, but others dip up to 90°. Core contains some scattered thin vesicular zones.
Grout was found in the following openings:
- 82.9'-84.0': Regular grout and sand grout up to 1/2" partially filling an irregular broken zone.
- 88.3'-90.1': Solid sand grout in irregular rough joint dipping 75° to 80°.
- 90.3'-92.8': Irregular rough fracture near parallel to core axis, core is about half sand grout and half rock. Core from 91.9' to 92.5' is solid grout. Calcite up to 1/8" thick lines some of the rock surfaces.
- 96.0'-96.1': Grout up to 1/4" thick in 30° dipping rough irregular joint.
- 96.2'-97.7': Grout and sand grout 1/4" to 2" thick in a very irregular and rough fracture. Fracture is partly along a vesicular zone 0.2' long near 97.6'.
- 101.3': Blob of grout 1/16" thick and 1/2" wide along side of core.

121.0'±-352'±: WELDED ASH-FLOW TUFF; medium gray, fairly hard (core breaks with light hammer blow and scratches with moderate knife pressure), strong foliation is caused by wavy, streaky pumice fragments up to 1/4" wide and some lineation due to flattened vesicles; this foliation dips 15° to 40°. Moderately Jointed. Joint spacing varies from 0.1' to 4.1', averages 0.9'. Most joints parallel the foliation, some joints contain stains of calcite, silt, iron and a black clay-like alteration product up to 1/8" thick.

EXPLANATION

*Measuring point is El. 5332.7'

RB = Rock Bit

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, N = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-651 LOCATION Far Rt. Abutment, Sta. 4+34, Ç GROUND ELEV 5332.1* DIP (ANGLE FROM HORIZ) 90°
COORDS. N. E.
BEGUN 9/28/76 FINISHED 10/12/76 DEPTH OF OVERBURDEN 80.4' TOTAL DEPTH 622.4' BEARING --
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED 314.1', 5018.6', 11-15-76 LOGGED BY J. Phillips LOG REVIEWED BY D. N. Magleby

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Type and size of hole: Nx

Casing and Cementing

Date	Depth	Casing
9/29	4.4	6"
10/1	78.9	4"
10/1	78.9-80.4	Cm
10/12	530.0-534.0	Cm
10/14	547.1-550.3	Cm
10/18	546.7-571.1	Cm
10/20	540.0-621.3	Cm
10/21	546.1	Nx

Water Return

Date	Interval	%
9/28	0.0-78.9	100
Lost 80% of water return momentarily at 47.2'		
10/1	72.5-100.0	100
Lost 98% of water return momentarily at 75.0'		
**10/2	160.0-220.0	90
10/24	220.0-279.9	75
10/4	279.4-339.9	85
10/5	339.9-399.9	40
10/5	399.9-457.2	60
10/6	457.2-495.4	50
10/6	495.4-519.3	60
10/7	519.3-532.2	50
10/7	532.2-535.9	50
10/12	535.9-543.7	90
10/21	547.2-622.4	80

**Water return in interval 100.0' to 160.0' not recorded.

PERCOLATION TESTS

Core recovery (%)	From (P, Cs, or Cm)	To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of test (min)
100	P 100	120	.25	10	10
100	P 120	140	1.0	10	10
100	P 140	160	0.14	10	10
100	P 160	180	0.04	10	10
100	P 180	200	0.0	10	10

Elevation (feet): 5211.7 (at depth 120)

Depth (feet): 110, 120, 130, 140, 150, 160, 170, 180, 190

Graphic log:

CLASSIFICATION AND PHYSICAL CONDITION

Grout is present in the following openings:

126.9': Grout 1/16" to 1/8" filling an irregular, rough vesicular zone dipping about 10°.

128.8': Grout 1/16" to 1/8" partially filling a rough, planar joint dipping 20°.

141.9': Grout up to 1/8" thick partially filling a planar, rough joint which is about horizontal.

147.7': Grout up to 1/16" thick partially filling a rough, irregular joint dipping 30°.

152.1'-152.7': Grout 1/16" to 1/8" thick partially filling a rough, irregular joint dipping 10°.

161.1': Grout up to 1/4" thick filling a horizontal, planar rough joint. Calcite up to 1/16" thick lines sides of joint.

176.6': Grout up to 1/8" thick filling a smooth, planar joint dipping 45° (normal to foliation).

177.8': Grout up to 1/16" thick partially filling a horizontal, planar, rough joint.

200.0'-200.6': Grout up to 1/16" thick in a rough, irregular joint dipping 80°.

201.0'-201.2': Grout to 1/16" thick coating a rough, irregular joint which dips about 80° to 90°.

218.9': Grout up to 1/8" thick coating a planar, slightly rough joint dipping 10°.

276.2'-276.8': Grout up to 1/2" thick in a smooth, planar joint dipping 75° to 80°. Joint also contains a thin silt-like stain.

336.0': Grout to 1/16" thick coating a slightly rough, planar joint dipping 45°. Joint also contains some silt, fine sand and iron stains.

352'±-460'±: WELDED ASH-FLOW TUFF; core has a wavy, streaky foliation (eutaxitic) caused by bands of light and dark flattened pumice that crosses the core at a dip of 15° to 40°. Some lineation is also due to some zones of flattened vesicles in the same plane as the banded pumice. Fairly hard, core breaks with light hammer blow and scratches with light knife pressure. Core is slightly prophyritic and slightly to moderately vesicular. Moderately Jointed. Joints are spaced from 0.1' to 4.1', averaging about 0.9'. Most joints are iron and manganese stained and some contain a clay-like alteration product up to 1/16" thick.

*Measuring point is El. 5332.7'

EXPLANATION

CORE LOSS
CORE RECOVERY

Type of hole: D = Diamond, N = Naystellite, S = Shot, C = Churn
Hole sealed: P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series): Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Hx = 3"
Approx. size of core (X-series): Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series): Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series): Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1887 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 3 OF 7

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-651 LOCATION Far Rt. Abutment, Sta. 4+34, ℄ GROUND ELEV. 5332.1* DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. E.

BEGUN 9/28/76 FINISHED 10/12/76 DEPTH OF OVERBURDEN 80.4' TOTAL DEPTH 622.4' BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED 314.1', 5018.6', 11-15-76 LOGGED BY J. Phillips LOG REVIEWED BY D. N. Magleby

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Depth of Water

Date	Hole Depth	Water Depth
10/4	220.0	120.4
10/5	339.9	210.4
10/6	457.2	286.0
10/7	519.3	299.0
10/11	504.0	303.6
10/21	547.2	311.0
10/22	622.4	311.2
10/23	622.4	311.3
10/27	622.4	312.2
11/15	622.4	314.1

Hole Completion

Installed 597' of 3/4" plastic pipe in hole for water level measurements.

Purpose of Hole

Test effectiveness of grout curtain and sample materials beneath the welded tuff formation.

TYPE AND SIZE OF HOLE: NxD (200–300)

CORE RECOVERY (%)

Interval (ft)	Core recovery (%)
200–220	100
220–235	100
235–240	96
240–260	100
260–280	100
280–300	100

PERCOLATION TESTS

Depth From (P, Cs, or Cm)	Depth To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min.)
P 200	220	.50	10	10
P 219.9	239.9	.46	10	10
P 239.9	259.9	.50	10	10
P 259.9	279.9	.44	10	10
P 279.9	299.9	.55	10	10

DEPTH (FEET): 210, 220, 230, 240, 250, 260, 270, 280, 290

CLASSIFICATION AND PHYSICAL CONDITION

Most joints are rough and irregular, some are parallel to foliation, others dip 70° to 90°

EXPLANATION

*Measuring point is El. 5332.7'

CORE LOSS / CORE RECOVERY

Type of hole . . . . D = Diamond, N = Naystellite, S = Shot, C = Churn
Hole sealed . . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 3 OF 7 HOLE NO. DH-651

GPO 854-825

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 4 OF 7

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-651 LOCATION Far Rt. Abutment, Sta. 4+34, ℄ GROUND ELEV. 5332.1* DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. ... E. ...

BEGUN 9/28/76 FINISHED 10/12/76 DEPTH OF OVERBURDEN 80.4' TOTAL DEPTH 622.4' BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED 314.1', 5018.6', 11-15-76 LOGGED BY J. Phillips LOG REVIEWED BY D. N. Magleby

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	NxD											
			P 299.9	319.9	1.3	10	10					
		100										
	310								310			
	320								320			
	330	100							330			
			P 319.9	359.9	.64	10	10					
	340								340			
	350	100							350			
								4980.7				
	360								360			
	370	100	P 359.9	399.9	1.33	10	10		370			
	380								380			
		100										
	390								390			
	NxD											

CORE LOSS

CORE RECOVERY

*Measuring point is El. 5332.7'

EXPLANATION

Type of hole . . . O = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Hx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx – 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Hx = 3"

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

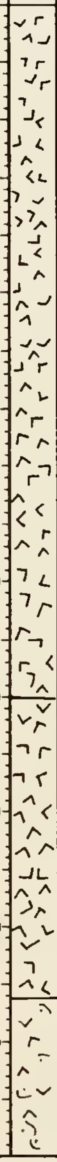
HOLE NO. DH-651 LOCATION Far Rt. Abutment, Sta. 4+34, ℄ GROUND ELEV 5332.1* DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. E.

BEGUN 9/28/76 FINISHED 10/12/76 DEPTH OF OVERBURDEN 80.4' TOTAL DEPTH 622.4' BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED 314.1', 5018.6', 11-15-76 LOGGED BY J. Phillips LOG REVIEWED BY D. N. Magleby

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	NxD											
		100	P 399.9	439.9	.50	10	10		410			
									420			
		100							430			
									440			
		100	P 432.4	472.4	0.0	10	5		450			
		98						4872.7	460			460'±-486.3'±: WELDED ASH-FLOW TUFF; medium to dark gray (mottled with black and red in lower half of section), hard, core breaks normal to core axis with moderate hammer blow and scratches with moderate to heavy knife pressure. Slightly porphyritic, nonvesicular. Moderately to Intensely Jointed. Joints are spaced from 1/2" (fragments) up to 3.2', averaging 0.7'. Most joints are smooth, planar and dip 20° to 40°. Some joints contain iron and manganese stains and coatings up to 1/16" thick and some contain a silt-like material up to 1/8".
									470			
		100	P 479.3	519.3	0.8	10	10		480			486.3'±-504'±: OBSIDIAN-LIKE ROCK; dark gray to black, fairly hard (requires moderate hammer blow to break), slightly porphyritic, nonvesicular, glassy texture. Moderately Jointed. Joint spacing averages about 0.7'. Most joints are 80° to 85° to core and are planar, smooth to rough, mostly stained with limonite and manganese and some silt-like material.
		95							490			
	NxD	69										



*Measuring point is El. 5332.7'

EXPLANATION

CORE LOSS

CORE RECOVERY

Type of hole .... D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 5 OF 7 HOLE NO. DH-651

7-1337 (8-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-651 LOCATION Far Rt. Abutment, Sta. 4+34, ℄ GROUND ELEV 5332.1* DIP (ANGLE FROM HORIZ.) 90°
COORDS. N. ... E. ...
BEGUN 9/28/76 FINISHED 10/12/76 DEPTH OF OVERBURDEN 80.4' TOTAL DEPTH 622.4' BEARING --
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED 314.1', 5018.6', 11-15-76 LOGGED BY J. Phillips LOG REVIEWED BY D. N. Magleby

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)
	NxD Nx Cs	69					
		29					
		75					
		76	P 518.2	527.2	2.9	10	10
		0					
		12	P 518.9	535.9	1.2	10	10
		100					
		0	Cm 535.9	543.7	6.2	10	10
		0					
		80					
	DNx	0					
		81	Cs 546.1	552.3	9.2	10	15
	Open hole	0					
		0					
		80					
		70	Cm 552.3	566.1	2.1	10	15
		67					
		0					
		30	P 559.3	571.1	1.5	10	15
		0					
		33					
		100	P 559.7	581.2	2.8	10	5
		54					
		0	P 560.0	600.0	35.0**	10	10
		100					
		100					
	NxD	100					

ELEVATION (FEET)	CLASSIFICATION AND PHYSICAL CONDITION
4828.2	504'±-508.7': SANDSTONE; fine grained, banded light brown to reddish brown (bands dip about 40°), lightly indurated, noncalcareous, slightly silty. Core breaks with moderate finger pressure. (Poor core recovery)
4824.0	508.7'-519.3': CONGLOMERATE; slightly indurated (can be fairly easily broken in the hands), consists of sand all sizes (tuffaceous) and subrounded to subangular gravel up to 1/2", most gravel is rhyolitic in composition. (Poor core recovery)
4813.4	519.3'-533.0'±: CONGLOMERATE; slightly indurated, consists of 30% sand all sizes and 70% subangular to rounded gravel of various composition up to 1" in size. Some core was recovered only as cored segments of gravel.
4799.7	533.0'±-559.9'±: SANDSTONE; tuffaceous, light brown, slightly indurated (core crumbles in fingers with little pressure), contains some scattered gravels up to 3/4". (Poor core recovery - driller reported 534.0'-543.7' as "very soft with silty sand return" and 550.3'-552.3' "drilled like large gravels").
4772.8	559.9'±-563.2': TUFF; light gray, lightly indurated (can be crushed with light finger pressure). Contact at 563.2' with underlying gravel is very sharp and dips about 45°.
4769.5	563.2'-582.9': GRAVEL; brownish red, composed mostly of subangular to subrounded rhyolite and quartzite 1/4" to 2". No apparent binder, except some clay was recovered from 574.7' to 574.9'. (Poor core recovery, driller reported 563.5' to 566.1' "drilled like gravels", 569.7'-571.1' "drilled smooth - medium sand in water return").
4749.8	582.9'-596.4': CLAY; medium brown, about 90% medium plastic fines, about 10% fine sand, compact - core can be easily crumbled in fingers, no bedding planes. (About 25% core recovery in this interval.)
4736.3	

DEPTH (FEET): 510, 520, 530, 540, 550, 560, 570, 580, 590

EXPLANATION

*Measuring point is El. 5332.7'
**Packer test questionable

CORE LOSS
CORE RECOVERY

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-651 LOCATION Far Rt. Abutment, Sta. 4+34, ℄ COORDS. N. ... E. ... GROUND ELEV. 5332.1* DIP (ANGLE FROM HORIZ.) 90°

BEGUN 9/28/76 FINISHED 10/12/76 DEPTH OF OVERBURDEN 80.4' TOTAL DEPTH 622.4' BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED 314.1', 5018.6', 11-15-76 LOGGED BY J. Phillips LOG REVIEWED BY D. N. Magleby

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	2-5/8" Rock Bit	w.s.										596.4'-622.4': GRAVEL; light to medium gray-brown, about 90% subrounded to subangular gravel to 2" (mostly hard quartzite and rhyolite composition), about 10% medium to coarse sand. Some core (0.3' long) consists of lightly compacted clayey sand and gravel. (Poor core recovery, driller reported 600.4' to 622.4' "sands and gravels").
	610	0							610			
	620								620			
	DNX	0						4710.3	622.4			
	30								30			
	40								40			
	50								50			
	60								60			
	70								70			
	80								80			
	90								90			

EXPLANATION

CORE LOSS

CORE RECOVERY

*Measuring point is El. 5332.7'

w.s. = wash sample

Type of hole . . . . . D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (8-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 5

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-651-A LOCATION Far Right Abutment* GROUND ELEV 5332.1** DIP (ANGLE FROM HORIZ) 90°

COORDS. N. E.

BEGUN 10-28-76 FINISHED 11-6-76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 500.6 BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	ELEVATION (FEET)	DEPTH (FEET)	CLASSIFICATION AND PHYSICAL CONDITION
Drill Equipment Failing 1500 rotary drill. Drillers H. Davis and D. Jes Drilling Methods Bentonite Mud 0.0'-5.0': 9-7/8" rock bit. 5.0'-78.5': 9" rock bit. 78.5'-82.3': 5-1/2" core barrel. 82.3'-84.8': 7-7/8' rock bit. 84.8'-93.5': 5-1/2" core barrel. 84.8'-93.5': 7-7/8" rock bit. 93.5'-98.7': 5-1/2" core barrel. 93.5'-495.0': 7-7/8' rock bit. 495.0'-500.6': 5-1/2" core barrel.	9-7/8" R.B.	0	5327.7		0.0'-5.0'±: GRAVEL AND COBBLES (Roadfill); as reported by drillers.
Drilling Conditions The following conditions were recorded: 0.0'-79.5' - Difficult to install 8" casing. 82.3'-84.8' - Had to ream hole to run 8" casing, welded tuff drilled slow. 219.0'-260.8' -Sand running in hole. 300.6'-331.0' - Sand running in hole. 495.0'-500.6' - Hole caved in - unable to wash to bottom.	9" R.B.	0		10	5.0'±-78.5': SILT (Zone I fill); as reported by drillers.
	D 5½"	100	5254.2	80	78.5'-81.9': CONCRETE (Grout Cap);
		35	5250.8		81.9'-500.6': WELDED ASH-FLOW TUFF; the following descriptions are based on cored intervals:
	R.B. 9"	0			
	D 5½"	100		90	84.8'-98.7': Medium gray, hard (requires heavy hammer blow to break, scratches with heavy knife pressure), slightly porphyritic, scattered vesicles from 1/8" up to 1/2"x1", faint foliation caused by flattened pumice is about normal to the core.
		100			
	R.B.	0			

Depth scale: 10, 20, 30, 40, 50, 60, 70, 80, 90

EXPLANATION

* Sta. 4+29 ℄ Grout Cap.
** Measuring point is elevation 5332.1'+0.6'= 5332.7'

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. 651-A LOCATION Far Right Abutment* COORDS. N. E. GROUND ELEV. 5332.1** DIP (ANGLE FROM HORIZ.) 90°
BEGUN 10-28-76 FINISHED 11-6-76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 500.6 BEARING --
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	R.8 7-7/8	0							110–190			

Casing Record

Date	Depth	Size
10-29	6.4'	10"
10-29	69.0'	8"
10-30	80.7'	8"

11-6-Broke 8" casing shoe - pulled 8" casing.
11-7-Tried to get 8" casing back in hole-wouldn't go beyond 78.8' - abandoned hole.

Mud Return
(Bentonite Mud)

0.0'-79.5' - 100%.
79.5'-362.0' - 95% to 100%.
362.0'-500.6' - 80% to 90%.

Water Level During Drilling

Not measured.

Hole Completion

Casing was pulled and hole was backfilled with silt.

Purpose of Hole

Intended purpose was to sample and test the sediments below the welded ash-flow tuff - hole aborted before purpose was accomplished.

Classification and Physical Condition

84.8'-85.1': Sand grout 1" thick along edge of core.
87.7'-: Smooth, planar joint 45° to core axis stained with grout.
92.1'-92.5': Solid sand grout, joint surfaces top and bottom are rough and irregular and about 80° to core axis (parallel to faint foliation).
92.5'-96.0': Irregular, rough fracture near parallel to core (only one side of joint is present), joint surface is coated with calcite up to 3/8" thick and grout is present from 0" to 3" thick.
96.0'-97.3': Solid sand grout. At 96.0' contact is irregular, 45° to core axis and coated with 1/4" thick calcite. Lower contact is about normal to core and is rough. The grout contains some angular rock fragments up to 4"x4".
97.3'-98.7': Irregular, rough joints open to 4" filled with sand grout and some angular rock fragments up to 1".

EXPLANATION

* Sta. 4+29 ℄ Grout Cap.
** Measuring point is elevation 5332.1'+0.6' = 5332.7'

CORE LOSS / CORE RECOVERY

Type of hole . . . D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

GPO 854-625

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 3 OF 5

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-651-A LOCATION Far Right Abutment GROUND ELEV 5332.1** DIP (ANGLE FROM HORIZ.) 90°
COORDS. N. E.

BEGIN 10-28-76 FINISHED 11-6-76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 500.6 BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D.N. Magleby LOG REVIEWED BY B.H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	7-7/8" rock bit	0										
	210								210			
	220								220			
	230								230			
	240								240			
	250								250			
	260								260			
	270								270			
	280								280			
	290								290			

EXPLANATION

* Sta. 4+29 ℄ Grout Cap.
** Measuring point is elevation 5332.1'+0.6'=5332.7'

CORE LOSS
CORE RECOVERY

Type of hole . . . D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 3 OF 5 HOLE NO. DH-651-A

GPO 854-628

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 4 OF 5

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-651-A LOCATION Far Right Abutment* COORDS. N. E. GROUND ELEV. 5332.1** DIP (ANGLE FROM HORIZ.) 90°

BEGUN 10-28-76 FINISHED 11-6-76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 500.6 BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D.N. Magleby LOG REVIEWED BY B.H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	7-7/8" rock bit	0										
	310								310			
	320								320			
	330								330			
	340								340			
	350								350			
	360								360			
	370								370			
	380								380			
	390								390			

EXPLANATION

* Sta. 4+29 ℄ Grout Cap.
** Measuring point is elevation 5332.1'+0.6'= 5332.7'

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, N = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 4 OF 5 HOLE NO. DH-651-A

GPO 854-625

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 5 OF 5

FEATURE: Teton Dam PROJECT: Teton Basin STATE: Idaho

HOLE NO: DH-651-A LOCATION: Far Right Abutment* GROUND ELEV: 5332.1** DIP (ANGLE FROM HORIZ.): 90°

COORDS. N. ... E. ...

BEGUN: 10-28-76 FINISHED: 11-6-76 DEPTH OF OVERBURDEN: -- TOTAL DEPTH: 500.6 BEARING: --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED: Not Measured LOGGED BY: D.N. Magleby LOG REVIEWED BY: B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	R.B. 7-7/8"	0										
	410								410			
	420								420			
	430								430			
	440								440			
	450								450			
	460								460			
	470								470			
	480								480			
	490								490			495.0'-500.6': Medium brown, slightly porphyritic, fairly hard (breaks with moderate hammer blow), brittle, broken from 0.1' fragments to core lengths 1.0'. Jointing is random, mostly smooth to rough and 30° to 60° to core axis stained with up to 1/16" of a brown clayey material.
	D 5½"	100						4832.1				

EXPLANATION

CORE LOSS / CORE RECOVERY

* Sta. 4+29 ℄ Grout Cap.
** Measuring point is elevation 5332.1'+0.6' = 5332.7'

Type of hole: D = Diamond, N = Noystellite, S = Shot, C = Churn
Hole sealed: P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series): Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series): Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series): Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series): Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE: Teton Dam PROJECT: Teton Basin STATE: Idaho SHEET 5 OF 5 HOLE NO. DH-651-A

GPO 854-625

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 9

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-651-B LOCATION Far Rt. Abutment* GROUND ELEV. 5332.0'** DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. ... E. ...

BEGUN 11-9-76 FINISHED 12-11-76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 885.3 BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
Drill Equipment 0.0'-89.2': Failing 1500 rotary drill. 89.2'-496.2': Ingersoll Rand down-hole hammer (Private contractor). 496.2'-885.3': Failing 1500 rotary drill. Drillers H. Davis, D. Jess, V. Bird Drilling Methods Bentonite mud was used in drilling except where otherwise noted. 0.0'-80.3': 9-7/8" R.B. 80.3'-89.2': 5-1/2" core barrel, then reamed to 7-7/8" with R.B. 89.2'-496.2': 8" down-hole hammer (by contractor). 496.2'-498.4': 7-7/8" R.B. 498.4-516.3': 5-1/2" core barrel, then reamed 7-7/8" R.B. to 516.3'. 516.3'-522.0': 5-1/2" core barrel, reamed 7-7/8" to 522.0'. 522.0'-527.0': 7-7/8" R.B. 527.0'-546.2': 5-1/2" core barrel, reamed 7-7/8" R.B. to 546.2'. 546.2'-558.0': 7-7/8" R.B. 558.0'-567.4': 5-1/2" core barrel, reamed 7-7/8" R.B. to 567.4'. 567.4'-596.7': 7-7/8" R.B. 596.7'-638.8': 5-1/2" core barrel, reamed 7-7/8" R.B. to 638.8'.	9-7/8" rock bit (80) D 5½" (later reamed to 7⅞") (90) 8" down hole hammer	0 100 0		NONE				5317.5 5254.3 5251.1	10, 20, 30, 40, 50, 60, 70, 80, 90			0.0'-5.0': GRAVEL AND COBBLES (Road Fill); as reported by drillers. 5.0'-78.2': SILT (Zone I fill in keyway); as reported by drillers. 78.2'-81.4': CONCRETE (Grout Cap); hard, tight contact with underlying rock, contact is normal to core axis. 81.4'-89.2': WELDED ASH FLOW TUFF; light gray, fairly hard (requires moderate hamme blow to break), slightly vesicular and slightly porphyritic. Lightly jointed, joint spacing varies from 0.9' to 2.5', averages 1.5'. From 82.0' to 85.5' is irregular, rough joint about parallel to core axis which is filled with grout and sand grout from 1/4" to 2" thick. Joint at 86.5' is planar, rough, silt stained and is about 30° to core axis.

EXPLANATION

* Sta. 4+19 ℄ Grout Cap.
** El. of measuring Pt. 5332.5'
RB = Rock Bit

CORE LOSS / CORE RECOVERY

Type of hole ... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed ... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Hx - 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 9 HOLE NO. DH-651-B

GPO 854-825

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. DH-651-B LOCATION Far Rt. Abutment* GROUND ELEV 5332.0'** DIP (ANGLE FROM HORIZ.) 90°
COORDS. N. E.
BEGUN 11/9/76 FINISHED 12/11/76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 885.3 BEARING --
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
638.8'-650.0': 4-1/2" RB. 650.0'-652.4': 3-7/8" core barrel, reamed 4-1/2" RB to 652.4'. 652.4'-660.0': 4-1/2" RB. 660.0'-665.0': 3-7/8" core barrel, reamed 4-1/2" RB to 665.0'. 665.0'-670.0': 4-1/2" RB. 670.0'-675.0': 3-7/8" core barrel, reamed 4-1/2" RB to 675.0'. 675.0'-682.2': Nx core barrel, reamed 4-1/2" RB to 682.2'. 682.2'-683.0': 4-1/2" RB. 683.0'-688.3': 3-7/8" core barrel, reamed 4-1/2" RB to 688.3'. 688.3'-700.0': 4-1/2" RB. 700.0'-702.0': 3-7/8" core barrel, reamed 4-1/2" RB to 702.0'. 702.0'-710.0': 4-1/2" RB. 710.0'-710.5': 3-7/8" core barrel, reamed 4-1/2" RB to 710.5'. 710.5'-885.3': 4-1/2" rock bit.	8" down hole hammer	0							110, 120, 130, 140, 150, 160, 170, 180, 190			89.2'-490'±: WELDED ASH-FLOW TUFF; as reported by drillers. Hole was drilled with down-hole air hammer - no core recovery. See log of DH-651 (Sta. 4+34) for log of similar rock in this interval.
SUMMARY OF DAILY DRILL REPORTS SHOWING: DRILLING CONDITIONS, MUD RETURN AND CASING ADVANCEMENT.												
<u>11/9/76 - Day Shift</u> Rock bit 9-7/8" hole 0.0'-22.0', 100% water return. 10" casing to 7.7'.												

EXPLANATION

RB = Rock Bit
* Sta. 4+19 ℄ Grout Cap.
** El. of measuring Pt. 5332.5'

CORE LOSS / CORE RECOVERY

Type of hole . . . D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

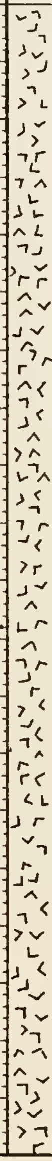
FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-651-8 LOCATION Far Rt. Abutment* GROUND ELEV 5332.0'** DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. .... E. ....

BEGUN 11/9/76 FINISHED 12/11/76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 885.3 BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
11/9/76 - Swing Shift Rock bit 9-7.8" hole 22.0'-80.3', 100% water return. Hit cement at 78.2'. Installed 80.3' of 8" casing.	8" down hole ham-mer	0										
	210								210			
11/10/76 - Day Shift Drilled 5-1/2" core barrel 80.3'-89.2', 100% mud return.	220								220			
Swing Shift Reamed hole to 7-7/8" from 80.3' to 89.2', 100% mud return.	230								230			
11/11/76 - Day Shift Cemented hole 89.2' to 79.3'.												
11/12/76 - Day Shift Reamed out cement to 7-7/8" from 79.3' to 89.2'.	240								240			
11/13/76 to 11/17/76 Hole was advanced with 8" down hole hammer (by private contractor) from 89.2' to 496.2'.	250								250			
11/18/76 - Day Shift Rock bit 7-7/8" from 494.2' to 498.4' - sand and gravel, 2' cave. Drilled with 5-1/2" core barrel 498.4' to 505.7'; 60% water return.	260 270								260 270			
Swing Shift Pumped in 1200 gals. revert but got back only 50 gals.	280								280			
11/14/76 - Day Shift Tried 3 more tanks revert - very little return. Started reaming hole to 192' with 7-7/8" rock bit using bentonite mud.	290								290			
Swing Shift Reamed 7-7/8" rock bit from 192'-505.7'												

CORE LOSS

CORE RECOVERY

EXPLANATION

* Sta. 4+19 ℄ Grout Cap.

** El. of measuring Pt. 5332.5'

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 4 OF 9

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-651-8 LOCATION Far Rt. Abutment* GROUND ELEV. 5332.0'** DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. .... E. ....

BEGUN 11/9/76 FINISHED 12/11/76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 885.3 BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
11/20/76 - Day Shift Drilled 5-1/2" core barrel from 505.7' to 516.3' (3 runs), no recovery 513.8' to 516.3', but drilled like gravels; 90% mud return. Swing Shift Drilled 5-1/2" core barrel 516.3' to 519.0', 100% mud return.	8" down hole hammer	0										
	310								310			
	320								320			
11/21/76 - Day Shift Drilled 5-1/2" core barrel from 519.0' to 522.1', then rock bit 7-7/8" to 527.0', 100% mud return. Swing Shift Drilled 5-1/2" barrel 527.0'-540.0' (2 runs), 95% mud return. Reamed hole with 7-7/8" rock bit to 540.0'.	330								330			
	340								340			
11/22/76 - Day Shift Drilled 5-1/2" core barrel 540.0' to 546.2', 7-7/8" rock bit to 558.0' - silt and sand with large gravels. Swing Shift 5-1/2" core barrel 558.0' to 567.4', 7-7/8" rock bit to 588.2'; 95% mud return.	350								350			
	360								360			
11/23/76 - Day Shift 7-7/8" rock bit 588.2' to 596.7'; 5-1/2" core barrel 596.7' to 606.2'. Swing Shift 5-1/2" core barrel 606.2' to 615.6', 95% mud return.	370								370			
	380								380			
11/24/76 - Day Shift 7-7/8" rock bit 596.7' to 615.6', 5-1/2" core barrel 615.6' to 625.6', 100% mud return.	390								390			

* Sta. 4+19 ℄ Grout Cap.
** El. of measuring Pt. 5332.5'

EXPLANATION

CORE LOSS
CORE RECOVERY

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Hx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 4 OF 9 HOLE NO. DH-651-8

GPO 854-825

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE: Teton Dam PROJECT: Teton Basin STATE: Idaho

HOLE NO. DH-651-8 LOCATION: Far Rt. Abutment* COORDS. N. .... E. .... GROUND ELEV. 5332.0'** DIP (ANGLE FROM HORIZ.) 90°

BEGUN 11/9/76 FINISHED 12/11/76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 885.3 BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED: Not Measured LOGGED BY: D. N. Magleby LOG REVIEWED BY: B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
11/24/76 Swing Shift 5-1/2" core barrel 625.6' to 636.4', lost 95% mud return 634.5' to 636.4'. Reamed 7-7/8" rock bit to 636.3'. 11/25/76 - Day Shift 5-1/2" core barrel 636.4' to 638.8', lost all mud return at 638.8'. Swing Shift Started running 6" casing to 120.5'. Had to drive casing 93' to 120.5'. 11/26/76 to 11/30/76 Installed 6" casing to 642.6' - had to drive casing in some areas. Casing separated at 231.8' but would still pass 4-1/2" collar. 12/1/76 - Day Shift 4-1/2" rock bit 642.6' to 650.0' - lost mud return. Swing Shift 3-7/8" core barrel 650.0' to 652.4', 4-1/2" rock bit to 660.0', no mud return. 12/2/76 - Day Shift 3-7/8" core barrel 660.0' to 665.0', 4-1/2" rock bit 665.0' to 670.0', 3-7/8" core barrel 670.0' to 675.0'. Swing Shift Nx core barrel 675.0' to 682.2' - no recovery; loose gravels and cobbles 4-1/2" rock bit to 683.0', 75% mud return. 12/3/76 - Day Shift Core barrel plugged off - couldn't force mud thru barrel. Swing Shift 3-1/2" core barrel 683.0' to 683.3', no recovery - silty sand and gravel; 90% mud return.	8" down hole hammer (410, 420, 430, 440, 450, 460, 470, 480, 490); 7-7/8" R.B.; D 5-1/2"	0; 0						4842.5	410, 420, 430, 440, 450, 460, 470, 480, 490			490'±-513.8': SANDSTONE; (as recovered in samples and reported by drillers), medium to light brown, fine to medium grained with scattered (10-15%) hard, rounded gravels up to 1", slightly indurated (can be broken with strong hand pressure), no bedding, some lense shaped gravels appear to be normal to core, non-calcareous.

CORE LOSS / CORE RECOVERY

EXPLANATION

RB = Rock Bit
* Sta. 4+19 ℄ Grout Cap.
** El. of measuring Pt. 5332.5'

Type of hole: D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed: P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series): Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series): Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series): Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series): Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (8-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 6 OF 9

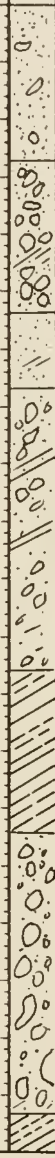
FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-651-8 LOCATION Far Rt. Abutment* COORDS. N. E. GROUND ELEV. 5332.0'** DIP (ANGLE FROM HORIZ.) 90°

BEGUN 11/9/76 FINISHED 12/11/76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 885.3 BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
12/4/76 - Day Shift 4-1/2" rock bit 683.0' to 700.0', 50 to 80% mud return Swing Shift 3-7/8" core barrel 700.0' to 702.0' (2 runs), 90% mud return.	D 5½"	53 62 0						4818.7	510			513.8'±-527.0'±: CONGLOMERATE (as recovered in samples and reported by drillers); dark gray to black with all colors of hard, rounded gravel up to 3" across. Slightly to moderately indurated (breaks with moderate hammer blow). Part of matrix is silt and fine sand; non-calcareous. Non-cored sections were reported by drillers as sand and gravel.
12/5/76 - Day Shift 4-1/2" rock bit 700.0' to 710.0', 3-7/8" core barrel 710.0' to 710.5', 100% mud return. Swing Shift 4-1/2" rock bit 710.0' to 726.0'; 95% mud return - sand and gravels, drilling smooth except in gravels.	7-7/8" Rock Bit D 5½"	100 77 0 95 100						4805.5 4798.9	520 530			527.0'±-533.6'±: SANDSTONE; medium brown, mostly fine to medium grained, silty, slightly to moderately indurated (can be broken with medium hand pressure), non-calcareous. 533.6'-558.0'±: Recovered samples are SILTY to CLAYEY SAND & GRAVEL, non-indurated to lightly indurated; sand all sizes, gravel is rounded, hard up to 1" across. Drillers reported non-cored sections as compacted silty sand and gravel.
12/6/76 - Day Shift 4-1/2" rock bit 726.0' to 740.0'; 50 to 70% mud return. Drilled smooth but slow to 733', then smooth and fast to 736', slow to 740'. Swing Shift 4-1/2" rock bit 740.0' to 784.2', 70% to 95% mud return (added cotton seed hulls). 740'-740.6' - Smooth drilling. 740.6'-741.7' - Soft gravel seam. 741.7'-778.1' - Firm smooth drilling. 778.1'-780.0' - Gravel seam. 780.0'-784.2' - Firm smooth drilling. 784.2'-790.0' - Hard material.	7-7/8" RB D 5½" 7-7/8" Rock Bit	39 0 74 0						4774.5 4760.3	540 550 560 570 580			558.0'±-572.2'±: SILTSTONE & CLAYSTONE (as recovered in a few samples); medium brown, compact (cannot be indented with fingers, scratches easily with knife, breaks with moderate hand pressure), non to slightly plastic; fine sand in some sections, non-bedded, non-calcareous. Drillers reported non-cored sections as silty sand. 572.2'-596.7'±: SAND, GRAVELS and some COBBLES; as reported by drillers; compact, some clay towards bottom of section.
12/7/76 - Day Shift 4-1/2" rock bit 790' to 821.1', 100% mud return. 790'-794' - Drilled smooth and slow. 794'-821' - Drilled hard and slow with few gravels.	D 5½"	67						4735.8	590			596.7'±-626.0'±: SILTSTONE & CLAYSTONE; light to medium gray to medium brown,



EXPLANATION

RB = Rock Bit
* Sta. 4+19 ℄ Grout Cap.
** El. of measuring Pt. 5332.5'

CORE LOSS
CORE RECOVERY

Type of hole .... D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed .... P = Pocket, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) .. Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 6 OF 9 HOLE NO. DH-651-8

GPO 884-625

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-651-B LOCATION Far Rt. Abutment* GROUND ELEV. 5332.0'** DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. .... E. ....

BEGUN 11/9/76 FINISHED 12/11/76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 885.3 BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

12/7/76 - Swing Shift
4-1/2" rock bit 821.1' to 841.1'; 90% mud return.
821.1'-825.6': Firm material.
825.6'-826.0': Gravels, lost 480 gals. mud.
826.0'-833.8': Firm material with some gravels.
833.8'-834.9': Cobbles, gravels and sand.
834.9'-841.1': Firm material with some gravels.

2/8/76 - Day Shift
4-1/2" rock bit 841.1' to 866.6'; 100% mud return. Drilled slow except in infrequent gravel areas.

Swing Shift
4-1/2" rock bit 866.6' to 883.8'; 95% mud return to 882.2' - firm, smooth drilling and then no mud return 882.2' to 883.0' in gravel and cobbles. Lost 980 gals. mud using 2 sack mix and 1/3 sack cotton seed hulls.

12/9/76 - Day Shift
4-1/2" rock bit 883.8' to 885.3', no mud return. Mixed 12 tanks mud with no return.

Swing Shift
Mixed 4 tanks of heavy mud, still no return.

12/10/76 - Day Shift
Pumped 3 tanks mud, still no return.

TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	ELEVATION (FEET)	DEPTH (FEET)
5½"	67		
	96		610
	98		620
		4706.5	
	100		630
		4697.5	
	83		
4½" Rock Bit	0		640
D 3-7/8"	100		650
4½" Rock Bit	0		
D 3-7/8"	88		660
4½" Rock Bit	0		
D 3-7/8"	30	4661.5	670
D Nx	0		680
D 3-7/8"	0		
4½" Rock Bit	0		690
		4633.5	

PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm) / TO; LOSS (G.P.M.); PRESSURE (P.S.I.); LENGTH OF TEST (MIN.) — no entries.

CLASSIFICATION AND PHYSICAL CONDITION

slight to medium pasticity, highly compacted (cannot be imprinted with finger pressure, scratches easily with knife, breaking with moderate hand pressure). No bedding planes apparent. Some sections contain some fine sand; interval from 622.5' to 622.8' contains scattered hard, rounded gravels up to 1". Core contains prominent joints or partings as follows:
604.6': Smooth, planar, manganese stained, slickensided joint that dips 45°.
607.0': Smooth, planar, slickensided joint dipping 40°
608.0', 609.0' and 610.2': Parallel, planar, slickensided joints dipping 45°.
611.0', 613.3' and 614.8': Smooth, planar, slickensided joints that dip about 45° and are near normal to each other.
618.7': Smooth, planar, slickensided joint that dips 40°.
619.7': Smooth, fairly planar, slickensided, manganese stained joint that dips 45° (not parallel to joint at 619.7').

626.0'±-635.0'±: CONGLOMERATE; medium brown, moderately indurated (can be broken with heavy hand pressure), consists of 60-70% hard, rounded gravel up to 2" across in a silty fine sand matrix, no apparent bedding.

635.0'±-671.0'±: SILTSTONE and CLAYSTONE; (as recovered in samples and reported by drillers), light gray to medium brown, medium plasticity, compact (cannot be indented with finger pressure, breaks with moderate hand pressure), unbedded. A smooth, planar, manganese stained joint at 635' has prominent slickensides and dips 45°. Drillers reported uncored sections as firm clay.

671.0'±-699.0'±: SAND, GRAVEL, COBBLES with some SILT; as reported by drillers.

EXPLANATION

* Sta. 4+19 ℄ Grout Cap.
** El. of measuring Pt. 5332.5'
RB = Rock Bit

CORE LOSS / CORE RECOVERY

Type of hole: D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed: P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series): Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series): Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series): Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series): Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-651-8 LOCATION Far Rt. Abutment* COORDS. N ... E ... GROUND ELEV. 5332.0'** DIP (ANGLE FROM HORIZ.) 90°

BEGUN 11/9/76 FINISHED 12/11/76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 885.3 BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	ELEVATION (FEET)	DEPTH (FEET)	CLASSIFICATION AND PHYSICAL CONDITION
12/10/76 - Swing Shift Pumped 1500 gals. mud and got 10% return before drilling, started turning rods and lost all return. Pumped in more mud - no return.	D 3⅞" 4½" Rock Bit D 3⅞" 4½" Rock Bit	70 0 100 0			699.0'±-712.8'±: SILTSTONE & CLAYSTONE (as recovered in cored sections); light brown to gray, slightly plastic, compact. Drillers indicated uncored sections as compact SILTY SANDS.
				710	Hole from 712.8' to 885.3' was advanced with 4-1/2" rock bit. Description of materials in this interval is based on drillers notes and some recovered cuttings.
12/11/76 Decided not to drill deeper because of no mud return. Pumped in 300 gals. of clean mud. Hole was capped off.			4619.7		712.8'±-719.2'±: GRAVEL.
			4613.3	720	719.2'±-724.1'±: SILTY SAND.
			4608.4		724.1'-729.0'±: GRAVEL
12/15/76 Down-hole geophysical log was made of hole.			4603.5	730	729.0'±-735.0'±: CLAY, compact.
			4597.5		735.0'±-736.0'±: GRAVEL.
			4596.5		736.0'±-740.6'±: SAND(?), compact.
			4591.9	740	740.6'±-741.7'±: GRAVEL
Purpose of Hole Determine characteristics of sedimentary materials below the welded ash-flow tuff.			4590.8		741.7'±-778.1'±: SILT & CLAY (?), compact, few scattered gravels. Mud return is mostly gray.
Hole Completion Casing was left in hole, hole was covered over with 5 gal. bucket. On 12/15/76 down-hole geophysical log was made of hole to depth of about 750', hole was partly caved below 750'.				750, 760, 770	
			4554.4		778.1'±-780.0'±: GRAVEL.
			4552.5	780, 790	780.0'±-825.6'±: SILTY CLAY(?), compact, few scattered gravels.

EXPLANATION

* Sta. 4+19 ℄ Grout Cap.
** El. of measuring Pt. 5332.5'

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series).. Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

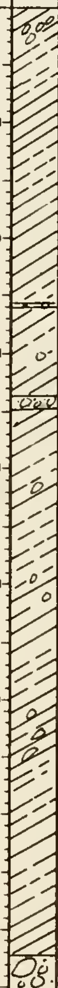
FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-651-B LOCATION COORDS. N. E. GROUND ELEV. 5332.0'** DIP (ANGLE FROM HORIZ.) 90°

BEGUN 11/9/76 FINISHED 12/11/76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 885.3 BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	4½" Rock Bit	0										
	810								810			
	820								820			
								4506.9				825.6'±-826.0'±: GRAVEL
								4506.5				826.0'±-833.7'±: SILTY CLAY(?); firm, few scattered gravel.
	830								830			
								4498.8				833.7'±-834.9'±: SAND, GRAVEL and COBBLES.
								4497.6				834.9'±-882.2'±: SILTY CLAY(?), firm, few scattered gravel.
	840								840			
	850								850			
	860								860			
	870								870			
	880								880			
								4450.3				882.2'±-885.3': GRAVEL AND COBBLES.
								4447.2				
	890								890			



EXPLANATION

* Sta. 4+19 ℄ Grout Cap.
** El. of measuring Pt. 5332.5'

Note: Description of materials from 712.8' to 885.3' is based on drillers notes and some recovered cuttings.

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 5

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-652 LOCATION Far Rt. Abutment, Sta. 5+11.2, 5.5' u/s/ ℄ GROUND ELEV 5332.0'* DIP (ANGLE FROM HORIZ.) ** 56°

COORDS. N. ... E. ...

BEGUN 10/2/76 FINISHED 10/21/76 DEPTH OF OVERBURDEN 90.0' TOTAL DEPTH 450.0' BEARING ** N19°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY J. Phillips LOG REVIEWED BY D. N. Magleby

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	6" Auger (0–70)	0						5328.3	5.0			0.0'-5.0': GRAVEL AND COBBLES (Road Fill); as reported by drillers.
												5.0'-90.0': SILT (Zone 1 fill in keyway), as reported by drillers.
	3" RB (70–90)	0						5257.8	90			
	NxD	29 / 100 / 93										90.0'-152.0'±: WELDED ASH-FLOW TUFF; hard (requires moderate hammer blow to break, scratches with heavy knife pressure), slightly vesicular, light weight, indistinct to faint foliation at 60° to core axis. Lightly Jointed. Joint spacing is from 0.3' to 5.5', averaging 2.5'. Most joints are smooth planar, and about 30° to 60° to core axis. Light purple gray.

Drill Equipment

Mobile B-40L

Driller

R. Abel

Drilling Methods

Advanced hole to 70' by using 6" auger. Placed Nx casing to 70'. Rock bitted with 3" bit 70'-90'. Cored with NqHw core barrel 90'-450', taking samples every 10'.

Drilling Conditions

0.0'-90.0': Smooth

Casing & Cementing

Date	Depth	Size
10/2	70.0	Nx(Cs)
10/4	90.0	Nx(Cs)
10/8	90.0-95.2	Conc.
10/14	94.0	Nx(Cs)
10/18	307.7-311.7	Conc.

Water Return

Date	Interval	% Return
10/4	70.0-90.0	100
10/8	90.0-95.2	95
10/9	95.2-107.7	95
10/11	107.7-165.0	80
10/12	165.0-177.7	80
10/13	177.7 197.7	40
10/14	197.7-235.0	20
10/16	235.0-297.2	20
10/18	302.9-307.7	0
10/19	307.7-327.7	0
10/19	327.7-347.7	90
10/20	347.7-387.7	30
10/20	387.7-427.4	80-90
10/21	427.4-450.0	30

EXPLANATION

*Measuring point is El. 5332.4'

RB= Rock Bit

CORE LOSS / CORE RECOVERY

Type of hole .... D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) .. Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) .. Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-652 LOCATION Far Rt. Abutment, Sta. 5+11.2, 5.5' u/s, ℄ Dam GROUND ELEV. 5332.0'* DIP (ANGLE FROM HORIZ.) ** 56°

COORDS. N. .... E. ....

BEGUN 10/2/76 FINISHED 10/21/76 DEPTH OF OVERBURDEN 90.0' TOTAL DEPTH 450.0' BEARING** N19°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY J. Phillips LOG REVIEWED BY D. N. Magleby

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

Water Level During Drilling

Not measured.

Water Level After Drilling

Not measured.

Hole Completion

Purpose of Hole

Test effectiveness of grout curtain.

TYPE AND SIZE OF HOLE: NxD

CORE RECOVERY (%)

Core recovery (%)
93
100
100
100
100
98
100
100
100
100
98
98

PERCOLATION TESTS

Hole sealed	Depth (feet) From	To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of test (Min.)
P	95.0	130.0	0.4	10	10
P	130.0	165.0	0.4	10	10
P	165.0	200.0	1.4	10	15

ELEVATION (FEET): 5206.4 **DEPTH (FEET):** 152.0

CLASSIFICATION AND PHYSICAL CONDITION

Grout was found in the following joints:
101.8': Grout 3/4" thick in smooth, planar joint about 30° to axis. Joint terminates on lower end in rough, irregular fracture.
103.5': Chalky grout stain in a rough, broken zone.
112.0'-112.5': Irregular, rough-sided opening about 45° to core, filled with grout 3" thick. Grout contains angular rock fragments up to 1/2" by 1-1/2".
118.0': Sand grout up to 1/2" filling an irregular, rough opening about 45° to core
129.0': Intersection of Ax open hole about 30° to core axis.
131.3': Grout up to 1/8" partially filling a very rough, irregular joint 45° to 60° to core axis.
132.2'-135.5': Concrete, hard, upper contact is about 10° to 15° to core axis and is well cemented. Lower contact is about 15° to core axis and fairly well cemented, but some openings up to 1/2". Contacts are fairly rough and irregular with up to 1/8" calcite on rock surfaces.
140.0': Grout up to 1/2" thick in an irregular, rough opening. Grout filling is rough.
145.7'-147.0': Grout up to 1/8" thick in a very rough, irregular opening from 10° to near parallel to core axis.
150.7': Irregular grout filling up to 1/4" thick, filling rough irregular opening.

152.0'-250.0': WELDED ASH-FLOW TUFF; light gray. Hard, core breaks with moderate hammer blow and scratches with heavy knife pressure. Some core length (6" long) can be broken with heavy hand pressure along foliation. Fairly distinct foliation 45° to 60° to core axis caused by flattened vesicles and some bands of pumice 1/4" wide. Intensely to Moderately Jointed. Joints spaced 0.1' to 2.4', mostly 0.5'. Most joints are parallel to foliation. Most breaks are rough and planar, 45° to 60° to core axis. Grout is present in the following joints:
162.0': Chalky grout up to 1/16" filling a smooth, planar joint about 50° to core axis.

EXPLANATION

*Measuring point is El. 5332.4'

CORE LOSS / CORE RECOVERY

Type of hole . . . D = Diamond, N = Noystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 3 OF 5

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO OH-652 LOCATION Far Rt. Abutment, Sta. 5+11.2, 5.5' u/s, ℄ Dam GROUND ELEV 5332.0' DIP (ANGLE FROM HORIZ.) ** 56°

COORDS. N. E.

BEGUN 10/2/76 FINISHED 10/21/76 DEPTH OF OVERBURDEN 90.0' TOTAL DEPTH 450.0' BEARING ** N19°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY J. Phillips LOG REVIEWED BY D. N. Magleby

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	NxD	100										
	210	100							210			
	220	99	P 200.0	235.0	.10	10	10		220			
	230	100							230			
		100										
	240	100							240			
	250	93	P 235.0	270.0	.80	10	10	5125.1	250			
	260	100							260			
	270	74							270			
		96										
	280	86							280			
	290	100	P 267.9	302.9	1.2	10	10		290			
	NxD							5086.2	297.0			

CLASSIFICATION AND PHYSICAL CONDITION:

166.0': Sand grout up to 3/8" thick filling a smooth, planar joint about 45° to core axis (nearly normal to foliation).
169.6': Sand grout 1/8" to 1/2" thick, filling a rough, slightly planar opening about 30° to core axis.
194.2': Chalky grout up to 1/16" thick, in a rough, planar to wavy joint about 50° to core axis. Some are parallel to the foliation.
208.1': Sand grout 3/8" thick in a smooth, planar joint about 30° to core axis which is about normal to the foliation.

250.0'-297.0': WELDED ASH-FLOW TUFF; light gray. Core breaks with moderate hand pressure and scratches with light knife pressure; lightly vesicular. Fairly distinct foliation between 45° to 60° to core axis caused by flattened vesicles and some wavy pumice banding up to 1/4" wide. Intensely Jointed. Joints spaced from less than 1/2" to 1.3', mostly between 0.3' to 0.8'. Most joints are parallel to the foliation which is 45° to 60° to core axis.
268.7'-270.7': Two high angle, rough to smooth joints about 20° to core axis.
Grout is present in the following joints:
253.4': Grout to 1/8" in a rough, planar joint 30° to 40° to core axis. Joint is parallel to the foliation.
257.3': Grout stains on irregular, rough joint 40° to core axis.
260.2': Grout 1/16" thick filling a smooth, planar joint 50° to core axis.
260.6': Grout stains in smooth, planar joint 50° to core axis.
272.3'-272.8': Grout up to 1" thick in an irregular joint intersecting some near parallel, smooth planar joints which contain 1/2" grout. Smooth joints are 30° to 35° to core axis and normal to foliation.
273.1': Grout to 1/8" thick in a planar, rough joint that is 50° to core axis and near normal to foliation.
273.3': Grout to 1/4" thick in a smooth, planar joint that is 45° to core axis and near normal to foliation.

*Measuring point is El. 5332.4'

EXPLANATION

CORE LOSS
CORE RECOVERY

Type of hole . . . D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-652 LOCATION Far Rt. Abutment, Sta. 5+11.2, 5.5, u/s, ℄ Dam GROUND ELEV. 5332.0'* DIP (ANGLE FROM HORIZ.)** 56°

COORDS. N. E.

BEGUN 10/2/76 FINISHED 10/21/76 DEPTH OF OVERBURDEN 90.0' TOTAL DEPTH 450.0' BEARING** N19°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY J. Phillips LOG REVIEWED BY D. N. Magleby

Type and size of hole: NxD (from top of sheet to bottom of sheet)

Core recovery (%): 100, 100, 100, 100, 93, 100, 99, 99, 99, 99, 99

Percolation test	From (feet)	To (feet)	Loss (G.P.M.)	Pressure (P.S.I.)	Length of test (Min.)
P	301.3	307.7	12.8	10	10
P	307.7	347.7	0.8	10	10
P	347.7	387.7	0.3	10	10

Elevation (feet): 5056.3 at Depth (feet): 333.0

Depth scale (feet): 310, 320, 330, 340, 350, 360, 370, 380, 390

CLASSIFICATION AND PHYSICAL CONDITION

291.8': Grout up to 1/16" in broken, irregular opening.
294.0': Grout stains on a smooth, planar joint 75° to core axis.
294.5': Grout stains on a rough, irregular joint 45° to core axis.
295.6': Grout stains in a fairly smooth joint 50° to core axis and normal to foliation.
296.8': Grout stains and silty clay stains in a rough, irregular joint which is 60° to core axis.

297.0'-333.0': WELDED ASH-FLOW TUFF; banded light and dark gray. Core breaks easily in hand with light pressure and scratches easily with a light knife pressure. Slightly vesicular. Streaky, wavy pumice bands (eutaxitic) and flattened vesicles give a strong foliation. Intensely to Moderately Jointed. Joints spaced from 0.1' to 1.3', mostly 0.7'. Most joints are parallel, planar and rough and near parallel to the foliation (50° to 60° to core axis).
309.8': Intersection of two joints 50° to 55° to core axis filled with brown calcite 1/2" thick. Joints are planar, smooth and irregular.
314.0': Irregular joint containing calcite in vuggy zone, 65° to core axis.
332.4'-333.7': Smooth, planar, silt stained joint 10° to core axis.
Grout is present in the following joints:
300.0': Grout up to 1/16" thick in rough, planar joint 25° with core axis. Joint contains silt and calcite stains.
300.9': Grout up to 3/8" thick in a planar, rough, silt-stained joint which is 60° to core axis (near parallel to flow lines). Part of grout is a chalky grout.

333.0'-450.0': WELDED ASH-FLOW TUFF; light gray-brown, streaked with limonite, especially along joints and in some vesicles, fairly hard, core breaks with moderate hammer blow and scratches with moderate knife pressure. Lightly to moderately vesicular. Distinct flattened vesicles and some streaky pumice zones gives a strong foliation.

EXPLANATION

*Measuring point is El. 5332.4'

CORE LOSS / CORE RECOVERY

Type of hole . . . D = Diamond, H = Hoyzellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Hx = 2"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 2"

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 5 OF 5

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. DH-652 LOCATION Far Rt. Abutment, Sta. 5+11.2, 5.5' u/s ℄ Dam GROUND ELEV 5332.0'* DIP (ANGLE FROM HORIZ) ** 56°

COORDS. N. E.

BEGUN 10/2/76 FINISHED 10/21/76 DEPTH OF OVERBURDEN 90.0' TOTAL DEPTH 450.0' BEARING ** N19°W

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED Not Measured LOGGED BY J. Phillips LOG REVIEWED BY D. N. Magleby

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS

**SPERRY SUN DOWN-HOLE SURVEY

Depth	Angle with Vertical	Bearing
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

CORE RECOVERY (%): 100, 98, 100, 93, 92, 93

PERCOLATION TESTS

Depth (Feet) From (P, Cs, or Cm)	To	Loss (G.P.M.)	Pressure (P.S.I.)	Length of Test (Min)
P 387.4	427.4	.13	10	10
P 425.0	450.0	0.5	10	10

ELEVATION (FEET): 4959.3 (at 450)

DEPTH (FEET): 410, 420, 430, 440, 450, 60, 70, 80, 90

CLASSIFICATION AND PHYSICAL CONDITION

Moderately to Lightly Jointed. Joints spaced from 0.1' to 3.7', mostly 0.7'-2.0'. Most joints are planar, rough and near parallel to the foliation.

406.5'-406.8': Manganese up to 1/4" thick in a planar, rough joint 20° to core axis. Joint also contains calcite.

408.4': Calcite and silt stains up to 1/16" on a smooth, planar joint 40° to core axis. Joint also contains manganese and iron stains up to 1/16" thick. 368'-437' most joints contain limonite staining.

Grout was found in the following joints:

350.0': Sand grout 1" thick in planar, smooth 1/8" thick calcite-lined joint 35° to core axis and normal to the foliation.

371.8': Intersection of 2 joints. Grout up to 3/8" thick filling planar, rough joint 35° to core axis, parallel to foliation. Other joint is 5° to core axis and contains no grout.

373.0': Irregular, rough joint 45° to core axis is stained with chalky grout.

389.9': Chalky grout 3/8" thick in a smooth, planar joint which is lined with calcite up to 1/16" thick. Joint is 30° to core axis and near normal to the foliation.

EXPLANATION

*Measuring point is El. 5332.4'

CORE LOSS / CORE RECOVERY

Type of hole . . . D = Diamond, N = Neyatellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1337 (8-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 2

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. HF-5 LOCATION Earth Fill Section of Dam* GROUND ELEV. 5313.0 DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. E.

BEGUN 10-22-76 FINISHED 11-22-76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 190.0' BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	ELEVATION (FEET)
Drill Equipment: Mobile 8-40L Drill	5-7/8" rock bit	0	5306.0

Drill Equipment
Mobile 8-40L Drill

Driller
E. Claunts

Drilling Methods
Drilled with 5-5/8" rock bit and clear water to 150'. Set 150' of 5" steel casing and 150' of 3" plastic casing. Poured grout plug from 150' to 135'. Drilled out grout plug with Nx core barrel to 150' with air. Used 2-15/16" rock bit and air to drill to 190'. Pulled out 3" plastic casing when removing rods from hole so set 153' of Nx steel casing and cemented inside 5" casing. Drilled out cement plug to 153', advanced hole to 184.4' with 3" drive sampler.

Casing Record
0.0'-150.0' - 5" steel pipe.
0.0'-150.0' - 3" plastic casing - pulled plastic casing and installed 153' of Nx steel casing.

Water Return During Drilling
0.0'-101.3' - 100%
Lost drill water return at 101.3' - pumped in about 3,000 gals. in 1 hr. - no return.

PERCOLATION TESTS

HYDRAULIC FRACTURE TESTS

Tests consisted of filling the open hole and nx casing with water to various levels and making measurements over a period of time of the rates which water seeped from the hole. Bottom of NxCs 153', bottom of open hole about 154', water level before starting test, 154'.

Test No. 1

Time	Depth to Water
9:50	92.1
51	94.3
52	96.0
53	97.5
54	99.0
55	100.3
56	101.5
58	103.5
59	104.6
10:00	105.6
01	106.5
02	107.3

Test No. 2

Time	Depth to Water
10:06	78.2
07	81.0
08	83.2
09	85.3
10	87.0
11	88.4
12	89.9
13	91.4
14	92.6
15	93.9
16	95.2
17	96.3
18	97.4
19	98.3
20	99.0

Test No. 3

Time	Depth to Water
10:24	68.7
25	71.8
26	74.3
27	76.6
28	78.6
29	80.7
30	82.6
32	85.6
33	87.0
34	88.4

Test No. 4

Time	Depth to Water
10:36	57.9
37	52.0
38	64.9
39	67.5
40	69.9
41	72.0
42	74.0
43	75.9
44	77.8

Test No. 5

Time	Depth to Water
10:46	45.3
47	50.6
48	54.2
49	57.4
50	60.4
51	63.0
52	65.4
53	67.7

Test No. 6

Time	Depth to Water
10:54	38.0
55	42.4
56	46.3
57	49.4
58	52.3
59	54.9
11:00	57.5
01	60.0
02	62.3

Test No. 7

Time	Depth to Water
11:05	19.1
06	30.0
07	35.3
08	39.7
09	43.5
10	46.8
11	49.9
12	52.8

CLASSIFICATION AND PHYSICAL CONDITION

0.0'-7.0': BOULDERS; as reported by drillers (Zone II fill).

7.0'-184.4': SILT; as reported by drillers and recovered in drive samples from 153.0' to 184.4' (Zone 1 fill).

EXPLANATION

* Sta. 26+00 ℄ grout cap, upstream face of dam.

CORE LOSS
CORE RECOVERY

Type of hole . . . . . D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed . . . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

7-1887 (8-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 2 OF 2

FEATURE: Teton Dam PROJECT: Teton Basin STATE: Idaho
HOLE NO.: HF-5 LOCATION: Earthfill section of dam* GROUND ELEV.: 5313.0 DIP (ANGLE FROM HORIZ.): 90°
COORDS. N. .... E. ....
BEGUN: 10-22-76 FINISHED: 11-22-76 DEPTH OF OVERBURDEN: -- TOTAL DEPTH: 190.0' BEARING: --
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED: See notes LOGGED BY: D. N. Magleby LOG REVIEWED BY: B. H. Carter

Notes on water losses and levels, casing, cementing, caving, and other drilling conditions

Drilling Condition

0.0'-7.0' - Slow and rough using rock bit and water.
7.0'-100.0' -Smooth and slow using rock bit and water.
100.0'-150.0' - Smooth and fast using rock bit and water.
150.0'-190.0' - Material very damp, used rock bit and air, bit stuck in 3" plastic casing and casing was pulled from hole. Reset with Nx steel casing to 153' and used 3" drive sample to clean out hole to 184.4'. Hole caved back to 172.0', muddy water and mud in hole at depth 154'.

Type and size of hole / Core recovery (%)

Depth (feet)	Type and size of hole	Core recovery (%)
100–153	5-3/8" rock bit	0
153–184.4	3" drive sample	100, 100, 100, 100, 100, 100
184.4–190	2 15/16" rock bit	0

Percolation tests

HYDRAULIC FRACTURE TESTS (Continued)

Test No. 8 Time	Depth to Water
11:14	24.0
15	44.2
16	52.7
17	58.7
18	63.7
19	67.9
20	71.4

Test No. 9 Time	Depth to Water
11:24	30.0
25	50.0
26	59.1
27	65.7
28	70.9
29	75.2
30	78.5
31	81.4
32	83.4

Test No. 10 Time	Depth to Water
11:36	26.6
37	47.3
38	64.2
39	71.3
40	75.9
41	80.0
42	82.1
43	84.6
44	86.4
45	87.9
46	89.4
47	91.0
48	91.9
49	92.7
50	93.5

Test No. 11 Time	Depth to Water
12:00	96.6
05	100.2
10	103.2
15	105.8
20	107.9
25	109.8
30	111.4
40	114.0
50	116.0
1:00	118.0
10	120.0
20	121.3
30	122.7
40	124.0
50	125.0
2:00	128.0
30	128.1
3:00	129.7
30	130.9
4:00	132.0
30	132.6

Depth (feet): 110, 120, 130, 140, 150, 160, 170, 180, 190. Graphic log hatched to 190.

Classification and physical condition

Notes: (Cont.)

Depth to Water During Drilling

Date	Depth of Hole	Depth to Water	Casing
10/23	150'	58.7	10'
10/24	150'	60.1	10'
10/30	150'	119	150'
10/31	190'	140	150'
11/22	184.4'	110	153'

Purpose of Hole

Conduct hydraulic fracture test in section of uncased hole in Zone 1 fill. Tests ran by Mr. C. Cortwright of the Independent Panel.

Hole Completion

Pulled NxCs, left 5" casing in hole.

EXPLANATION

* Sta. 26+00 ℄ grout cap, upstream face of dam

CORE LOSS
CORE RECOVERY

Type of hole . . . . . D = Diamond, H = Hoystellite, S = Shot, C = Churn
Hole sealed . . . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series) . Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE: Teton Dam PROJECT: Teton Basin STATE: Idaho SHEET 2 OF 2 HOLE NO. HF-5

GPO 854-028

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 1 OF 2

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. HF-6 LOCATION Earthfill section of dam* GROUND ELEV. 5313.0 DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. E.

BEGUN 11/2/76 FINISHED 11/5/76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 152.0' BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
Drill Equipment Mobile B40-L drill Driller E. Claunts Drilling Methods Drilled with 5-5/8" rock bit using air from 0.0' to 137.0'. Installed 137.0' of 3" plastic casing and cemented hole with 2 sacks of cement. Drilled out cement and advanced hole to 152.0' with 2-15/16" rock bit using air - had difficulties because silt was wet. Pulled plastic casing from hole when attempting to remove rock bit and rods from hole -- abandoned hole.	5-5/8" rock bit	0	NONE					5300.0				0.0'-13.0': BOULDERS; as reported by drillers (Zone 11 fill). 13.0'-152.0': SILT; as reported by drillers (Zone I fill).

Depth to Water During Drilling*

Date	Depth of Hole	Depth of Water
11/5	137'	122'
11/6	152'	115'

*Casing at depth 137'.

EXPLANATION

* Sta. 26+25 ℄ grout cap, upstream face of dam.

CORE LOSS
CORE RECOVERY

Type of hole . . . O = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 1 OF 2 HOLE NO. HF-6

GPO 834-628

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

SHEET 2 OF 2

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. HF-6 LOCATION Earthfill section of dam* GROUND ELEV 5313.0 DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. ... E. ...

BEGUN 11/2/76 FINISHED 11/5/76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 152.0' BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
Purpose of Hole Conduct hydraulic fracture test in section of uncased hole in Zone I fill. Test not run because silt in test section was saturated - probably as a result of water introduced into HF-5, Sta. 16+00. Hole Completion Pulled plastic casing, backfilled hole with silt.	5-5/8" rock bit	0							110 120 130			
	rock bit 2 15/16"	0						5161.0	140 150			
									60 70 80 90			

EXPLANATION

* Sta. 26+25 ℄ grout cap, upstream face of dam.

CORE LOSS

CORE RECOVERY

Type of hole .... O = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series). . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho SHEET 2 OF 2 HOLE NO. HF-6

GPO 854-825

7-1337 (8-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho

HOLE NO. HF-7 LOCATION Earth Fill Section of Dam* GROUND ELEV. 5316.5 DIP (ANGLE FROM HORIZ.) 90°

COORDS. N. E.

BEGUN 11-11-76 FINISHED 11-19-76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 127.0 BEARING --

DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
Drill Equipment Mobile B40-L auger. Driller E. Claunts Drilling Methods Drilled with 7-7/8" rock bit using air to 10.0'. Set 10' of 8" casing and augered with 6" flight augers 10.0' to 107.0'. Set 107.0' of Nx casing and cemented in bottom of hole. Drilled out cement plug with air and advanced hole 107.0' to 127.0' with 1-7/8" I.D. x 2-1/2" O.D. drive sampler. Drilling Conditions 0.0'-6.0': Slow and rough. 6.0'-10.0': Smooth. 10.0'-70.0': Damp, slow. 70.0'-107.0': Very damp and slow. Hole Completion Casing was left in hole, hole was capped. Purpose of Hole Conduct hydraulic fracture test in section of uncased hole in Zone I fill in dam. Tests were run by C. Cortwright of the Independent Panel.	0-10: 7-7/8" rock bit 10-: 6" auger	0 0	HYDRAULIC FRACTURE TESTS Tests consisted of filling the open hole and Nx casing with water to various levels and making measurements over a period of time of the rates which water seeped from the hole. Bottom of NxCs 107', bottom of hole 127'. No water in hole at start of test.	5310.5				0.0'-6.0': BOULDERS; as reported by drillers (Zone II fill). 6.0'-127.0': SILT, as reported by drillers and determined from drive samples taken from 107' to 127' (Zone I fill).

Time	Depth to Water		Time	Depth to Water
Test No. 1			Test No. 5	
10:38	112.7		11:30	86.0
48	112.3		31	86.4
			32	87.0
Test No. 2			33	87.5
10:50	107.7		34	88.0
54	106(?)		35	88.5
57	103.3		36	89.0
58	103.5		37	89.6
59	103.8		38	90.0
11:00	104.0			
01	104.2		Test No. 6	
02	104.4		11:41	80.6
			42	81.3
Test No. 3			43	82.0
11:08	99.6		44	82.7
09	99.7		45	83.3
10	100.0		46	83.9
11	100.3		47	84.5
12	100.5		48	85.1
13	100.8		49	85.7
14	101.0		50	86.3
15	101.2		51	86.8
Test No. 4			Test No. 7	
11:20	92.6		11:53	77.5
21	92.7		54	78.0
22	93.1		55	78.9
23	93.5		56	79.8
24	94.0		57	80.4
25	94.3		58	81.0
26	94.7		59	81.8
27	95.0		12:00	82.5
			01	83.1
			02	83.7

EXPLANATION

CORE LOSS / CORE RECOVERY

*Sta. 27+00 ℄ grout cap, upstream face of dam.

Type of hole . . . D = Diamond, H = Haystellite, S = Shot, C = Churn
Hole sealed . . . P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

GPO 854-825

7-1337 (6-74)
Bureau of Reclamation

GEOLOGIC LOG OF DRILL HOLE

FEATURE Teton Dam PROJECT Teton Basin STATE Idaho
HOLE NO. HF-7 LOCATION Earth Fill Section of Dam* GROUND ELEV. 5316.5 DIP (ANGLE FROM HORIZ.) 90°
COORDS. N. ... E. ...
BEGUN 11-11-76 FINISHED 11-19-76 DEPTH OF OVERBURDEN -- TOTAL DEPTH 127.0' BEARING --
DEPTH AND ELEV. OF WATER LEVEL AND DATE MEASURED See Notes LOGGED BY D. N. Magleby LOG REVIEWED BY B. H. Carter

NOTES ON WATER LOSSES AND LEVELS, CASING, CEMENTING, CAVING, AND OTHER DRILLING CONDITIONS	TYPE AND SIZE OF HOLE	CORE RECOVERY (%)	PERCOLATION TESTS: DEPTH (FEET) FROM (P, Cs, or Cm)	TO	LOSS (G.P.M.)	PRESSURE (P.S.I.)	LENGTH OF TEST (MIN.)	ELEVATION (FEET)	DEPTH (FEET)	GRAPHIC LOG	SAMPLES FOR TESTING	CLASSIFICATION AND PHYSICAL CONDITION
	6" auger	0										
	1-7/8" I.D. drive sample	100, 100, 100, 100, 100, 100, 100, 100, 100						5189.5	110–127			

PENETRATION RESISTANCE TESTS using 2-1/2" OD x 1-7/8" I.D. sampler and 250# hammer.

Test No.	Blows/foot
PR #1	55
PR #2	73
PR #3	65
PR #4	99
PR #5	65
PR #6	120
PR #7	43

HYDRAULIC FRACTURE TESTS (Cont.)

Test No.	Time	Depth to Water
8	12:03	74.0
	04	74.7
	05	75.6
	06	76.4
	07	77.2
	08	78.2
	09	78.7
	10	79.6
9	12:12	70.0
	13	78.7
10	12:15	61.5
	16	62.8
	17	64.2
	18	65.6
	19	66.8
11	12:20	58.5
	21	59.7
	22	61.1
12	12:23	52.9
	24	53.9
13	12:25	46.0
	26	47.7
14	12:27	40.0
	28	42.2
15	12:29	34.9
	30	36.7
16	12:31.5	29.6
	32.5	32.2
	33.5	34.9
	34.5	37.5
17	12:36	30.0
	37	32.4
	38	34.9
18	12:39	27.1
	40	29.6
	41	32.3
	42	34.7
	43	37.0
19	12:44	20.5
	45	24.0
	46	26.6
	47	29.7

HYDRAULIC FRACTURE TESTS (Cont.)

Test No.	Time	Depth to Water
20	12:49	14.0
	50	17.3
	51	20.6
21	12:53	6.0
	54	10.0
22	12:55	0
	55.5	2.5
	56	4.6
	57	9.2
	58	12.5
	59	16.8
	13:00	20.0
	01	23.0
	02	26.0
	03	28.6
	04	31.1
	05	33.5
	06	35.7
	07	37.8
	08	39.9
	09	41.8
	10	43.8
	12	47.0
	21	61.1
	26	66.6
	31	71.2
	36	75.1
	41	78.5
	46	81.5
	51	84.1
	56	86.4
	14:01	88.5
	06	90.3
	11	92.0
	16	93.6
	21	94.1
	51	100.7
23	14:57	51.8
	58	54.0
	59	55.7
	15:00	57.3
	01	58.9
	02	60.3
24	15:05	15.4
	06	19.2
	07	22.5
	08	25.3
	09	28.0
	10	30.6
25	15:13	12.0
	14	14.8
	15	17.8
	16	20.6
	17	23.2
26	15:28.5	0
	29	2.5
	29.5	4.8
	30	5.4
	30.5	8.1
	31	9.6
	31.5	11.4
	32.5	14.4
	33.5	17.2
	34.5	19.8
	35.5	22.2
	36.5	24.6
	37.5	26.8
	38.5	29.0
	39.5	31.0
	40.5	33.0

* Sta. 27+00 ℄ grout cap, upstream face of dam.

CORE LOSS
CORE RECOVERY

EXPLANATION

Type of hole .... D = Diamond, N = Haystellite, S = Shot, C = Churn
Hole sealed .... P = Packer, Cm = Cemented, Cs = Bottom of casing
Approx. size of hole (X-series) . . Ex = 1-1/2", Ax = 1-7/8", Bx = 2-3/8", Nx = 3"
Approx. size of core (X-series) . . Ex = 7/8", Ax = 1-1/8", Bx = 1-5/8", Nx = 2-1/8"
Outside dia. of casing (X-series). Ex = 1-13/16", Ax = 2-1/4", Bx = 2-7/8", Nx = 3-1/2"
Inside dia. of casing (X-series) . . Ex = 1-1/2", Ax = 1-29/32", Bx = 2-3/8", Nx = 3"

GPO 854-625

BY Cortright	DATE 10/17/76	PROJECT Teton Dam	SHEET 1 OF 2
CHKD BY	DATE	FEATURE Joint Transmissability Test	
DETAILS			

Joint No.	Dip	Strike	Location, Intersection w/ Grout Cap Sta	Submerged Length of Joint Ft	Max. Depth of Water Ft	Volume of Pond Gal	Loss GPM
U1	35	N25W	13+40	5.4	0.7	37.7	0.0065
U2	31	N25W	13+35	4.4	0.7	12.0	None
U3	49	N25W	13+31	3.4	0.6	3.3	None
U4	44	N30W	13+22				
Pond 1				2.6	0.5	2.4	None
Pond 2				1.8	0.4	2.3	"
Pond 3				2.1	0.6	4.6	"
U5	45	N20W	13+20				
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	45	N30W	13+17	2.2	0.5	18.0	1.09
U7	21	N25W	13+13				
Pond 1				4.1	0.5	4.4	0.01
Pond 2						2.1	None
U8	38	N20W	13+05	2.4	0.4	2.9	None
U9	30	N15W	12+93	2.7	0.4		None
U10	25	N8W	12+90	8.0	0.6	26.4	0.10
U11	25	N5W	12+83				
Pond 1				2.7	0.5	16.3	0.08
Pond 2				3.4	0.7	10.1	None
D1	25	N10W	13+40	5.8	0.8	14.2	0.58
D4	42	N13W	13+22	4.9	0.9		
D5	36	N25W	13+20	6.0	0.6	6.8	0.21
D7	78	N33W	13+13	3.9	1.0	32.1	0.09

BY Cortright	DATE 10/17/76	PROJECT Teton Dam	SHEET 2 OF 2
CHKD BY	DATE	FEATURE Joint Transmissability Test	
DETAILS			

Notes:

Date of test - 10/14/76
Test by - C. Cortright
Ponds numbered from upstream
Stations on grout cap ℄.
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+40, 20ft d/s, elev. 5185

Joint U6 - Return flow appeared from joint 6ft 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.

cc:
Jansen
Sherman

BY Marrelan	DATE 16 Nov 76	PROJECT TETON DAM	SHEET 1 OF 5
CHKD BY	DATE	FEATURE RIGHT ABUTMENT KEYWAY	
DETAILS Pressure Test Data			Rev

STATION 12+65±

DH-610, 45° Right

3.4' – 14.8'	: 0.38 gpm at 10 psi	$K = 1.3 \times 10^{-4}$ ft/min
14.8' – 24.8'	: 0.04 gpm at 10 psi	K = 0.1 " " "

DH-611, 22½° Right.

2.5' – 13.9'	: 0.48 gpm at 10 psi	K = 1.3 " " "
13.9' – 23.9'	: 0.09 gpm at 10 psi	K = 0.09 " " "

DH-612, Vertical

2.5' – 13.5'	: 0.5 gpm at 10 psi	K = 1.3 " " "
13.5 – 23.5'	: 0 gpm at 10 psi	K = 0 " " "

STATION 13+05±

DH-613, 45° Right.

3.4' – 14.8'	: 1.1 gpm at 10 psi	K = 3.0 " " "
14.8 – 24.8	: 0.2 gpm at 10 psi	K = 0.4 " " "

DH-614, 22½° Right.

3.3' – 24.4'	: 0.94 gpm at 10 psi	K = 1.3 " " "
13.4 – 24.4'	: 1.07 gpm at 10 psi	K = 2.7 " " "
0 – 24.7'	: 0.05 gpm at 0 psi	K = 0.2 " " "

DH-615, Vertical

3.5' – 14.9'	: 0 gpm at 10 psi	K = 0 " " "
14.9 – 24.9	: 0 gpm at 10 psi	K = 0 " " "

STATION 13+20±

✓ DH-616, 45° Right.

2.9' – 14.3'	: 10 gpm at 10 psi	K = 27. " " "
6.4 – 14.3'	: 0.23 gpm at 10 psi	K = 0.8 " " "
14.3 – 24.3'	: 5.2 gpm at 10 psi	K = 13. " " "
18.9 – 24.3	: 0 gpm at 10 psi	K = 0 " " "

DH-617, 21°± Right.

2.5' – 13.9'	: 0 gpm at 10 psi	K = 0 " " "
13.9' – 23.9'	: 0 gpm at 10 psi	K = 0 " " "
0 – 23.9	: 0.4 gpm at 0 psi	$K = 1.6 \times 10^{-4}$ ft/min

NOTE: Concrete Cap was cracked during failure at Stations 13+12 and 13+26

Incl No. 3

BY	DATE	PROJECT	SHEET 2 OF 5
[illegible]	16 Nov 76	TETON DAM	
CHKD BY	DATE	FEATURE RIGHT ABUTMENT KEYWAY	

DETAILS
Pressure Test Data Rev

STATION 13+20± (CON'T)

- DH-618, Vertical

2.8' – 14.2' : 0 gpm at 10 psi $K = 0 \times 10^{-4}$ ft/min
14.2' – 24.2' : 0 gpm at 10 psi $K = 0 \times 10^{-4}$ ft/min

DH-619, 30° Left.

* 5.3' – 16.7' : 1.64 gpm at 10psi K = 4.1 " " "
** 16.7' – 26.7' : 4.81 gpm at 10psi K = 10. " " "
0 – 26.7' : 4.9 gpm at 0 psi K = 18 " " "

* Water emerged from fracture downstream of grout cap
** Water emerged from fractures upstream and downstream of grout cap.

STATION 13+35±

- DH-620, 45° Right

4.4' – 14.4' : 0.1 gpm at 10 psi K = 0.3 " " "
14.4' – 24.4' : 0.1 gpm at 10 psi K = 0.25 " " "
24.4' – 34.2' : 0.1 gpm at 10 psi K = 0.2 " " "

Note: This hole extended to depth 98 feet. This part of hole requested by Independent Panel – Test data for entire hole shown on sheet 5 of this set.

DH-621, 22½° Right

0 – 23.5 : 8 gpm at 10 psi $K = 9 \times 10^{-4}$ ft/min
: 13.8 gpm at 18 psi. $K = 10 \times 10^{-4}$ ft/min

Pigment used during testing emerged from DH-619 and from fractures downstream.

DH-622, Vertical

2.1' – 13.5' : 1.9 gpm at 10 psi $K = 5.0 \times 10^{-4}$ ft/min
13.5' – 23.5' : 0 gpm at 10 psi K = 0

DH-623, 22° left

0 – 13.4', : 15.1 gpm at 0 psi $K = 170 \times 10^{-4}$ ft/min
3.4' – 13.4', : 12.7 gpm at 10 psi K = 35 " " "
11.4' – 21.4' : 1.2 gpm at 10 psi K = 3.0 " " "
3.4' – 21.4' : 1.6 gpm at 10 psi K = 4.0 " " "

BY Monahan	DATE 1 Dec 76	PROJECT Teton Dam	SHEET 3 OF 5
CHKD BY	DATE	FEATURE Right Abutment Keyway	
DETAILS Pressure Test Data			

Station 13+67±

DH-624, 45° Right

2.9'-14.3'	: 3.5 gpm at 10 psi	k= 10. x10^-4 ft/min
14.3'-24.3'	: 0 gpm at 10 psi	k= 0 " " "
0 - 24.3'	: 1.8 gpm at 0 psi	k= 10 " " "

DH-625, Vertical

8.9'-15.3'	: 0.4 gpm at 10 psi	k= 1.5 " " "
15.3'-26.3'	: 0 gpm at 10 psi	k= 0
0 - 26.3'	: 0.2 gpm at 0 psi	k= 0.7 " " "

DH-626, 22½° Left

5.3'-16.7'	: 12.5 gpm at 10 psi*	k= 28 " " "
16.7'-26.7'	: 7.5 gpm at 10 psi *	k= 75 " " "
0 - 26.7'	: 4.5 gpm at 0 psi*	k= 15 " " "

* Considerable leakage downstream

Station 14+00±

DH-627, 46° Right

1.6 - 11.0	: 0.18 gpm at 10 psi	k= 0.7 " " "
11.0 - 21.0	: 0.14 gpm at 10 psi	k= 0.4 " " "
0 - 21.7	: 0.03 gpm at 0 psi	k= 0.2 " " "

DH-628, Vertical

1.0' - 11.0'	— Not Valid Test —	
4.6' - 11.0'	: 0 gpm at 10 psi	k= 0 " " "
11.0' - 21.0'	: 0.14 gpm at 10 psi	k= 0.3 " " "
0 - 21.0'	: 0.3 gpm at 0 psi	k= 1.5 " " "

DH-629, 34° Left.

1.0' - 11.0'	: 7.1 gpm at 10 psi*	k= 23 " " "
2.6' - 11.0'	: 2.8 gpm at 10 psi**	k= 10 " " "
11.0' - 21.0'	: 0.7 gpm at 10 psi	k= 2.5 " " "
0 - 21.0'	: 0.39 gpm at 0 psi	k= 2.3 " " "

* Leaked from hole DH-628 and joint

** Leaked from Joint

BY Manahan	DATE 1 Dec 76	PROJECT Teton Dam	SHEET 4 OF 5
CHKD BY	DATE	FEATURE Right Abutment Keyway	
DETAILS Pressure Test Data			

Check

Station 14+24F

DH-630

1.3'–11.7'	: 4.3 gpm at 10 psi *	$K = 15 \times 10^{-4}$ ft/min
11.7–21.7'	: 0.4 gpm at 10 psi *	K = 1.0 " " "
0 – 21.7'	: 0.9 gpm at 10 psi *	K = 6.0 " " "

* Leakage U/S and D/S

DH-631, Vertical

1.2'–11.6'	: 0.26 gpm at 10 psi	$K = 7.5 \times 10^{-4}$ ft/min
11.6' – 21.6'	: 0.23 gpm at 10 psi	K = 4 " " "
0 – 21.6'	: 0.03 gpm at 0 psi	K = 3.2 " " "

DH-632, 34° Left

1.7 – 13.1'	: 2.55 gpm at 10 psi *	K = 7.5 " " "
13.1 – 23.1'	: 1.59 gpm at 10 psi	K = 4 " " "
0 – 23.1'	: 0.7 gpm at 0 psi	K = 3.5 " " "

* Leakage upstream

7-1654 (1-73)
Bureau of Reclamation

BY	DATE	PROJECT	SHEET
Monahan	1 Dec 76	Teton Dam	5 OF 5
CHKD BY	DATE	FEATURE	
		Right Abutment Keyway	

DETAILS: Pressure Test Data — Holes Requested by I.R.G

Station 13+35±
DH - 620, 45° Right

4.4 - 14.4'	: 0.1	gpm at 10 psi		k = 0.3 x10⁻⁴ ft/min
14.4' - 29.4'	: 0.1	" " 10 "		k = 0.25 " " "
29.4' - 39.2'	: 0.1	" " 10 "		k = 0.2 " " "
33' - 38'	: 0.2	" " 10 "		k = 0.7 " " "
38' - 43'	: 0.1	" " 10 "		k = 0.3 " " "
43' - 48'	: 0	" " 10 "		k = 0 " " "
48' - 53'	: 0.1	" " 10 "		k = 0.25 " " "
53' - 58'	: 0	" " 10 "		k = 0 " " "
58' - 63'	: 0	" " 10 "		k = 0 " " "
63' - 68'	: 0.1	" " 10 "		k = 0.22 " " "
68 - 73'	: 0	" " 10 "		k = 0 " " "
73' - 78'	: 0.1	" " 10 "		k = 0.2 " " "
78' - 83'	: 0.1	" " 10 "		k = 0.2 " " "
83' - 88'	: 0	" " 10 "		k = 0 " " "
88' - 93'	: 0.4	" " 10 "		k = 0.7 " " "
93' - 98'	: 0	" " 10 "		k = 0 " " "

Station 13+33.5, 14 feet U.S., 20° from Vertical downstream
DH - 633

5' - 10'	: 0.1	gpm at 10 psi	0.02	k = 0.5 x10⁻⁴ ft/min
10' - 15'	: 0.88	" " 10 "	0.18	k = 4.0 " " "
15' - 20'	: 0.41	" " 10 "	0.05	k = 1.7 " " "
20' - 25'	: 0.02	" " 10 "	"	k = 0.08 " " "
25' - 30'	: 0	" " 10 "	'	k = 0 " " "
30' - 35'	: 0	" " 10 "	,	k = 0 " " "
35' - 40'	: 0.04	" " 10 "		k = 0.1 " " "
40' - 45'	: 0.04	" " 10 "		k = 0.1 " " "
45' - 50'	: 0.04	" " 10 "	)	k = 0.09 " " "
50' - 55'	: 17.6	" " 10 "	3.50	k = 35 " " "
55' - 60'	: 17.3	" " 10 "	3.46	k = 35 " " "
60' - 65'	: 0.4	" " 10 "	0.13	k = 0.7 " " "
65' - 70'	: 0.1	" " 10 "	.2	k = 0.17 " " "
70' - 75'	: 0.13	" " 10 "	0.03	k = 0.2 " " "
75' - 80'	: 0.1	" " 10 "	0.12	k = 0.15 " " "

Unable to set packers below 80 feet
Total depth of hole 90 feet.

BY Manahan	DATE 6 Nov '76	PROJECT TETON DAM	SHEET 1 OF 2
CHKD BY	DATE	FEATURE	
DETAILS Pressure Test Data, Spillway			

DH-601

4'-40'	: 6.5 gpm at 10 psi	$K = 9 \times 10^{-4}$ ft/min
7'-40'	: 6.5 gpm at 10 psi	$K = 10 \times 10^{-4}$ ft/min
39.6'-74.6'	: 0.25 gpm at 10 psi	$K = 0.35 \times 10^{-4}$ ft/min
74.2'-109.2'	: 0.34 gpm at 10 psi	$K = 0.45 \times 10^{-4}$ ft/min

DH-602

4.7'-34.7'	: 2.92 gpm at 10 psi	$K = 4.7 \times 10^{-4}$ ft/min
5.7'-34.7'	: 1.07 gpm at 10 psi	$K = 1.7 \times 10^{-4}$ ft/min
34.7'-64.7'	: 2.1 gpm at 10 psi	$K = 3.5 \times 10^{-4}$ ft/min
64.7'-94.7'	: 0.38 gpm at 10 psi	$K = 0.6 \times 10^{-4}$ ft/min

DH-603

4.5'-39.5'	: 1.0 gpm at 10 psi	$K = 1.5 \times 10^{-4}$ ft/min
38.0'-73.7'	: 0.8 gpm at 10 psi	$K = 1.1 \times 10^{-4}$ ft/min
73.7'-108.7'	: 0.26 gpm at 10 psi	$K = 0.35 \times 10^{-4}$ ft/min

DH-604

5.5'-40.5'	: 0.77 gpm at 10 psi	$K = 1.1 \times 10^{-4}$ ft/min
40.5'-75.1'	: 0.5 gpm at 10 psi	$K = 0.7 \times 10^{-4}$ ft/min
74.7-109.7'	: 0.13 gpm at 10 psi	$K = 0.15 \times 10^{-4}$ ft/min

DH-605

6.0'-36.0'	: 0.2 gpm at 10 psi	$K = 0.35 \times 10^{-4}$ ft/min
36.0'-66.0'	: 0.185 gpm at 10 psi	$K = 0.3 \times 10^{-4}$ ft/min
66.0-96.0'	: 0.03 gpm at 10 psi	$K = 0.06 \times 10^{-4}$ ft/min

DH-606A

6.8'-41.8'	: 0.35 gpm at 10 psi	$K = 0.5 \times 10^{-4}$ ft/min
41.4'-76.4'	: 0.64 gpm at 10 psi	$K = 0.9 \times 10^{-4}$ ft/min
76.0'-111.0'	: 0.19 gpm at 10 psi	$K = 0.27 \times 10^{-4}$ ft/min

DH-607A

5.7'-40.3'	: 1.3 gpm at 10 psi	$K = 1.7 \times 10^{-4}$ ft/min
39.9'-74.9'	: 0.25 gpm at 10 psi	$K = 0.35 \times 10^{-4}$ ft/min
74.9-109.5'	: 0.04 gpm at 10 psi	$K = 0.07 \times 10^{-4}$ ft/min

DH-608

5.8'-35.8'	: 1.3 gpm at 10 psi	$K = 2 \times 10^{-4}$ ft/min
35.8'-65.8'	: 0.3 gpm at 10 psi	$K = 0.45 \times 10^{-4}$ ft/min
65.8'-95.8'	: 0.1 gpm at 10 psi	$K = 0.15 \times 10^{-4}$ ft/min
95.8'-125.8'	: 0.3 gpm at 10 psi	$K = 0.45 \times 10^{-4}$ ft/min.

BY	DATE	PROJECT	SHEET
Monahan	6 Nov 76	TETON DAM	SHEET 2 OF 2
CHKD BY	DATE	FEATURE	
DETAILS	Pressure Tests, Spillway		

DH - 609

5.7' - 40.7'	: 0.9 gpm at 10 psi	$K = 1.3 \times 10^{-4}$ ft/min
40.3' - 75.3'	: 1.7 gpm at 10 psi	$K = 2.5 \times 10^{-4}$ ft/min
74.9' - 109.9'	: 2.2 gpm at 10 psi	$K = 3.5 \times 10^{-4}$ ft/min
110.0' - 145.0'	: 0.3 gpm at 10 psi	$K = 0.4 \times 10^{-4}$ ft/min

No: Tests : $n = 31$
Mean : 0.55×10^{-4} ft/min
MAX : 10×10^{-4} ft/min
Min : 0.06×10^{-4} ft/min
Average : 1.63×10^{-4} ft/min

BY Moreland	DATE 12 Nov '76	PROJECT TETON DAM	SHEET ___ OF ___
CHKD BY	DATE	FEATURE Right Embankment	
DETAILS Pressure Test Data - post Failure Explorations.			

Pressure test data for DH-650 and DH-651 reflect holes were drilled from top of embankment. Locations, elevations and angles are from post drilling surveys and will differ from previously reported data. Static head included in Calculating "K".

DH-650, Station 3+00, 3' U/S Centerline grout Curtain, 31° from Vertical, left (up station), hole angled 2.2° upstream away from Centerline, ground elevation 5332.9, Elev. Top Csg. 5332.9 feet

*	90.0' - 97.7' :	32.1 gpm at 10 psi	K = 40 x 10^{-4} ft/min
	99.8' - 109.8' :	32.9 gpm at 10 psi	K = 60 x " " "
	103.1' - 127.4' :	28.3 gpm at 10 psi	K = 12 x " " "
	127.6' - 162.6' :	13.4 gpm at 10 psi	K = 3.5 x " " "
	160.6' - 197.2' :	8.4 gpm at 10 psi	K = 2 x " " "
	197.6' - 232.6' :	0 gpm at 10 psi	K = 0 x " " "
	232.6' - 267.5' :	20.3 gpm at 10 psi	K = 3.4 x " " "
	267.5' - 302.5' :	6.1 gpm at 10 psi	K = 1.0 x " " "
	301.7' - 331.7' :	2.1 gpm at 10 psi	K = 0.3 x " " "
	331.5' - 351.5' :	0 gpm at 10 psi	K = 0 x 10^{-4} ft/min

Lost drilling water at depth 91.6'

DH-652, Station 5+10, 5.5' U/S grout ℄, 33.6° from Vertical, Ground El. 5332.0, Csg El. 5332.7

	95.0' - 130.0' :	0.40 gpm at 10 psi	K = 0.14 x 10^{-4} ft/min
	130.0' - 165.0' :	0.40 gpm at 10 psi	K = 0.1 x " " "
	165.0' - 200.0' :	1.4 gpm at 10 psi	K = 0.35 x " "
	200.0' - 235.0' :	0.1 gpm at 10 psi	K = 0.02 x " " "
	235.0' - 270.0' :	0.8 gpm at 10 psi	K = 0.12 x " " "
	267.9' - 302.9' :	1.2 gpm at 10 psi	K = 0.25 x " " "
*	301.3' - 307.7' :	2.8 gpm at 10 psi	K = 1.5 x " " "
	307.7' - 347.7' :	0.8 gpm at 10 psi	K = 0.1 x " " "
	347.7' - 387.7' :	0.3 gpm at 10 psi	K = 0.4 x " " "
	387.4' - 427.4' :	0.1 gpm at 10 psi	K = 0.01 x " " "
	425.0' - 450.0' :	0.5 gpm at 10 psi	K = 0.09 x 10^{-4} ft/min

* Lost drilling water at depth 303'

BY	DATE	PROJECT	SHEET
[illegible]		TETON DAM	14 OF
CHKD BY	DATE	FEATURE	
		Right Abutment	

DETAILS
Pressure Test Data, Pre-Construction Drilling

Hole No	Surf Elev	∡ Vert	Depth W.T	Test depth – depth (Length)	Vert Depth W.T	Static Head Ft	Gage Head PSI	Total Head Ft	Rate gpm	K -10^{-4} ft/min
303	5319.5	30°	356.2	327.5–337.5 (10)	337.6	283.6	50	399	0	0
						"	100	515	0	0
				336.5–357.5 (11)		296.2	50	412	0	0
						"	100	527	1.2	0.2
				356.3–379.3 (23)		337.6	50	453	0	0
						"	100	569	0	0
				376.3–397.3 (21)		337.6	50	453	0	0
						"	100	569	0	0

...nahan	DATE 6 Nov '76	PROJECT TETON DAM	SHEET 1 OF 2
BY	DATE	FEATURE	
...S Pressure Test Data, Spillway			

DH-601

4' – 40'	: 6.5 gpm at 10 psi	$K = 9 \times 10^{-4}$ ft/min
7' – 40'	: 6.5 gpm at 10 psi	$K = 10 \times 10^{-4}$ ft/min
39.6' – 74.6'	: 0.25 gpm at 10 psi	$K = 0.35 \times 10^{-4}$ ft/min
74.2' – 109.2'	: 0.34 gpm at 10 psi	$K = 0.45 \times 10^{-4}$ ft/min

DH-602

4.7' – 34.7'	: 2.92 gpm at 10 psi	$K = 4.7 \times 10^{-4}$ ft/min
5.7' – 34.7'	: 1.07 gpm at 10 psi	$K = 1.7 \times 10^{-4}$ ft/min
34.7' – 64.7'	: 2.1 gpm at 10 psi	$K = 3.5 \times 10^{-4}$ ft/min
64.7' – 94.7'	: 0.38 gpm at 10 psi	$K = 0.6 \times 10^{-4}$ ft/min

DH-603

4.5' – 39.5'	: 1.0 gpm at 10 psi	$K = 1.5 \times 10^{-4}$ ft/min
38.0' – 73.7'	: 0.8 gpm at 10 psi	$K = 1.1 \times 10^{-4}$ ft/min
73.7' – 108.7'	: 0.26 gpm at 10 psi	$K = 0.35 \times 10^{-4}$ ft/min

DH-604

5.5' – 40.5'	: 0.77 gpm at 10 psi	$K = 1.1 \times 10^{-4}$ ft/min
40.5' – 75.1'	: 0.5 gpm at 10 psi	$K = 0.7 \times 10^{-4}$ ft/min
74.7 – 109.7'	: 0.13 gpm at 10 psi	$K = 0.15 \times 10^{-4}$ ft/min

DH-605

6.0' – 36.0'	: 0.2 gpm at 10 psi	$K = 0.35 \times 10^{-4}$ ft/min
36.0' – 66.0'	: 0.185 gpm at 10 psi	$K = 0.3 \times 10^{-4}$ ft/min
66.0 – 96.0'	: 0.03 gpm at 10 psi	$K = 0.06 \times 10^{-4}$ ft/min

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41.4' – 76.4'	: 0.64 gpm at 10 psi	$K = 0.9 \times 10^{-4}$ ft/min
76.0' – 111.0'	: 0.19 gpm at 10 psi	$K = 0.27 \times 10^{-4}$ ft/min

DH-607A

5.7' – 40.3'	: 1.3 gpm at 10 psi	$K = 1.7 \times 10^{-4}$ ft/min
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DH-608

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65.8' – 95.8'	: 0.1 gpm at 10 psi	$K = 0.15 \times 10^{-4}$ ft/min
95.8' – 125.8'	: 0.3 gpm at 10 psi	$K = 0.45 \times 10^{-4}$ ft/min

BY	DATE	PROJECT	SHEET 2 OF 2
Monahan	6 Nov 76	TETON DAM	
CHKD BY	DATE	FEATURE	

DETAILS Pressure Tests, Spillway

DH-609

5.7' - 40.7'	: 0.9	gpm at 10 psi	$K = 1.3 \times 10^{-4}$ ft/min
40.3' - 75.3'	: 1.7	gpm at 10 psi	$K = 2.5 \times 10^{-4}$ ft/min
74.9' - 109.9'	: 2.2	gpm at 10 psi	$K = 3.5 \times 10^{-4}$ ft/min
110.0' - 145.0'	: 0.3	gpm at 10 psi	$K = 0.4 \times 10^{-4}$ ft/min

No: Tests : $n = 31$
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MAX : 10×10^{-4} ft/min
Min : 0.06×10^{-4} ft/min
Average : 1.63×10^{-4} ft/min

BY Monahan	DATE 12 Nov '76	PROJECT TETON DAM	SHEET ___ OF ___
CHKD BY	DATE	FEATURE Right Embankment	
DETAILS Pressure Test Data - post Failure Explorations.			

Pressure test data for DH-650 and DH-651 reflect holes were drilled from top of embankment. Locations, elevations and angles are from post drilling surveys and will differ from previously reported data. Static head included in Calculating "K".

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*	90.0' - 97.7' :	32.1 gpm at 10 psi	K = 40 x 10^{-4} ft/min
	99.8' - 109.8' :	32.9 gpm at 10 psi	K = 60 x " " "
	103.1' - 127.4' :	28.3 gpm at 10 psi	K = 12 x " " "
	127.6' - 162.6' :	13.4 gpm at 10 psi	K = 3.5 x " " "
	160.6' - 197.2' :	8.4 gpm at 10 psi	K = 2 x " " "
	197.6' - 232.6' :	0 gpm at 10 psi	K = 0 x " " "
	232.6' - 267.5' :	20.3 gpm at 10 psi	K = 3.4 x " " "
	267.5' - 302.5' :	6.1 gpm at 10 psi	K = 1.0 x " " "
	301.7' - 331.7' :	2.1 gpm at 10 psi	K = 0.3 x " " "
	331.5' - 351.5' :	0 gpm at 10 psi	K = 0 x 10^{-4} ft/min

Lost drilling water at depth 91.6'

DH-652, Station 5+10, 5.5' U/s grout ℄, 33.6° from Vertical. Ground El. 5332.0, Csg El. 5332.7

q:

	95.0' - 130.0' :	0.40 gpm at 10 psi	K = 0.14 x 10^{-4} ft/min
	130.0' - 165.0' :	0.40 gpm at 10 psi	K = 0.1 x " " "
	165.0' - 200.0' :	1.4 gpm at 10 psi	K = 0.35 x " " "
	200.0' - 235.0' :	0.1 gpm at 10 psi	K = 0.02 x " " "
	235.0' - 270.0' :	0.8 gpm at 10 psi	K = 0.12 x " " "
	267.9' - 302.9' :	1.2 gpm at 10 psi	K = 0.25 x " " "
*	301.3' - 307.7' :	2.8 gpm at 10 psi	K = 1.5 x " " "
	307.7' - 347.7' :	0.8 gpm at 10 psi	K = 0.1 x " " "
	347.7' - 387.7' :	0.3 gpm at 10 psi	K = 0.4 x " " "
	387.4' - 427.4' :	0.1 gpm at 10 psi	K = 0.01 x " " "
	425.0' - 450.0' :	0.5 gpm at 10 psi	K = 0.09 x 10^{-4} ft/min

* Lost drilling water at depth 303'

Incl No. 2

BY	DATE	PROJECT	SHEET
[signature]		TETON DAM	14 OF
CHKD BY	DATE	FEATURE	
		Right Abutment	

DETAILS Pressure Test Data, pre-Construction Drilling

Hole No	Surf Elev	∡ Vert	Depth W.T	Test depth – depth (Length)	Vert Depth W.T	Static Head Ft	Gage Head PSI	Total Head Ft	Rate gpm	K -10^{-4} ft/min
303	5319.5	30°	356.2	327.5 – 337.5 (10)	337.6	283.6	50	399	0	0
						"	100	575	0	0
				336.5 – 357.5 (11)		296.2	50	412	0	0
						"	100	527	1.2	0.2
				356.3 – 379.3 (23)		337.6	50	453	0	0
						"	100	569	0	0
				376.3 – 397.3 (21)		337.6	50	453	0	0
						"	100	569	0	0

REVIEW REPORT

by the

Embankment Construction Task Group

for the

U.S. Department of the Interior
Teton Dam Failure Review Group

February 1977

ABSTRACT

This task group was originally charged to determine if the dam embankment was constructed in accordance with the contract plans and specifications. Added later was a second task to evaluate the embankment design concepts.

A generalization of the task group's critical conclusions would suggest the USBR (1) did not make proper assessments during design as to the potential for migration of zone 1 into the rock foundation, (2) did not choose and specify proper foundation preparation, and (3) did not make proper adjustments for critical embankment and foundation conditions encountered during construction. Assurance against the occurrence of the above events is provided by good organizational relationships and by having design and construction performed by experienced people with competent technical knowledge.

Detailed conclusions of the Embankment Construction Task Group are presented at the end of Appendix D.

INTRODUCTION

Purpose and Scope

This task group was originally charged to review the construction of Teton Dam embankment and foundation treatment exclusive of foundation pressure grouting for compliance with the plans and specifications. Added later was a second task to evaluate the embankment design concepts.

Participants

David C. Ralston, Soil Engineer, Soil Conservation Service, Washington, D.C., Chairman
Neil F. Parrett, Soil Engineer, Corps of Engineers, Washington, D.C.
Samuel D. Stone, Jr., Soil Engineer, Tennessee Valley Authority, Knoxville, Tennessee

Activities

The task group members first met at the damsite from August 3-6, 1976. The task group (1) interviewed USBR construction inspectors, (2) reviewed inspection records and reports, and (3) inspected the damsite and project laboratory. Following the site visit, members of the task group analyzed a variety of earthwork data. The members met again in Washington, D.C., on September 27-29, 1976, to coordinate report activities. The task group met in Knoxville, Tennessee, on January 19-21, 1977. The task group conclusions and associated discussions, charts, tables, and construction photographs are contained in this report.

Documentation

The documents reviewed include the following:

1. Specifications No. DC-6910, Volumes 1-4. The applicable specifications and drawings for Teton Dam.

2. Design Considerations for Teton Dam, October 1971, prepared by the USBR.

3. Construction Materials Test Data for Teton Dam, January 28, 1971, prepared by the USBR, E&R Center.

4. Teton Dam and Power and Pumping Plant.

   a. Soil Sample Index Sheets.

   b. Memorandums of Laboratory Test Results, Denver Laboratory.

   c. Undisturbed Embankment Samples Testing.

5. Earthwork Information extracted from Weekly Progress Reports.

6. Part C—Earthwork Construction Data from L-29 Reports.

7. Compilation of Earthwork Control Data, Summary of Field and Laboratory Tests of Compacted Fill, Zones 1, 2, 3, and 4.

8. Earthwork Control Statistics, Zones 1 and 3.

9. Design Engineer Notes, Design Considerations, March 1967 through October 1970.

10. Project Construction Office Photograph file.

11. As-Built Cross Sections of the Embankment Foundation.

12. Special Reports by Embankment Inspectors.

13. Daily Reports by Embankment Inspectors.

DESIGN REVIEW

Scope of Review

This task group's review of design was generally restricted to an overview of the design development, design philosophy, and major design criteria. No study was made of the detail design computations or analytical analyses.

Significant Design Assumptions

The following quotations have been extracted from USBR design notes to show the sequential development of significant decisions during the design of Teton Dam:

From Teton (Fremont) Dam Design Considerations (March 1967):

"1. General considerations: . . . cheapest and most abundant material is a silt . . . relatively good tan ϕ values and low permeability . . . low resistance to erosion, susceptibility to cracking, and liquefaction tendency . . . Third, the formation[1] at the damsite is thought to contain open joints. It is possible that some combination of open joints may extend from the reservoir to the downstream toe that will not be intercepted by grouting.

"2. Basic design criteria . . .

"(a) The upstream and downstream sides of the core should be blanketed by semipervious zones of sandy gravel . . .

"(b) A wide flat sloped cutoff trench should be provided across the valley to minimize cracking . . .

"(c) A relatively heavy upstream shell should be provided to compensate for possible strength loss in the saturated core due to seismic activity.

"(d) Under the core, open joints should be filled with grout or concrete so that piping through the formation will be impossible. Under the shells, piping should be controlled by removing overburden of the ML soil classification and replacing it with sand and gravel."

From Teton Dam Gradation Test Curves (June 1969). Zone 1 Material:

"Seepage velocities may be too low to move particles."

From Teton Dam (no date):

"Justification for sand filter blanket—heavy gravel rock zones.

"1. To prevent piping failure due to cracks in the core . . .

"2. To prevent piping failure due to seeps at rock contact . . .

"3. To prevent major spring from developing due to erosion of fines from open joints in the rock."

[1] The word "formation" is defined in specifications paragraph 66.c. as follows: "Formation—Any sedimentary, igneous or metamorphic material represented as a unit in geology, generally called rock . . ."

From USBR Design and Construction Book Teton Dam—Design Considerations, Crest Details (13 November 1969):

". . .

"1. . . .

"2. The silt core should be surrounded by sand and gravel . . ."

From Teton Dam—Design Considerations Abutment Section Above El 5300± (18 November 1969):

"1. Stripping . . .

"2. Riprap toe trench . . .

"3. Toe drains—pervious blanket d.s. side of cutoff (key) trench.

"a. All appreciable volume of seepage through the embankment could only result from cracks thru the core. Since the core is less than 50 feet high and is founded on formation in this elevation range, settlement cracks thru the core are unlikely.

"b. Considerable seepage may occur thru formation . . . and seepage will probably outcrop on the lower abutment.

"c. Conclusion: Toe drains or pervious blankets on the downstream side of the cutoff trench would serve no useful purpose.

"4. Extended grout curtain beyond limits of embankment

"a. . . . could be accomplished after the reservoir is placed in operation . . .

"5. Cutoff Trench

"a. Depth . . .

"b. Width . . .

"c. Cutoff trench side slopes.

"d. Special treatment of cracks in formation in the bottom of the cutoff trench.

"a. Alternate treatment methods include . . .

"b. Purpose of treatment

"1. Prevent movement of emb (embankment) fines into the cracks.

"2. Intercept seepage paths in surface formation layers where min (minimum) grout travel occurs."

From Teton Dam—Design Considerations, Abutment Section Below El 5300± (20 November 1969):

"1. Stripping . . .

"2. Foundation treatment in addition to stripping

"a. Shell area

"(1) . . .

"(2) Considerable seepage may issue from formation under the downstream shell . . .

"b. Core area

"(1) In the vicinity of the grout cap(s) excavation must be extended to groutable rock. The width of this area would correspond to the desired minimum width of the grout curtains and may be on the order of 30 feet. Within this area joints and cracks wide enough to permit the flow of grout should be cleaned out, caulked or sealed and grouted under pressure.

"(2) Formation under zone 1 but outside the limits of the grout curtain area. Required work includes (a) Excavation and dental treatment to remove loose rock; (b) Special compaction to prevent percolation along the formation embankment contact; (c) Large cracks and joints slushed or/and grouted and filled with grout by gravity.

"3. Upstream blanket under shell, on abutments, to extend the path of percolation . . .

"a. . . .

"b. . . .

"c. . . .

"d. . . .

"e. Conclusion: Reinforcing the grout curtain appears more economical . . ."

The following parameters were used in the embankment stability analysis:

Zone	Angle of internal friction ϕ	Cohesion intercept C (PSF)		Dry density (PCF)
		(wet)	(sat)	
1	31°	1,656	360	99.8 (Lab test)
2	35°	0	0	126.9 (Assumed)
3	31°	1,656	360	99.8 (Assumed from zone 1)
4	31°	1,656	360	99.8 (Assumed from zone 1)
5	35°	0	0	105.4 (Assumed)
Alluvium	35°	0	0	126.9 (Assumed)
Foundation	45°	0	0	137.5 (Assumed)

The following parameters were used in seepage analyses.

The permeability coefficient adopted for zones 1 and 3 was 10^{-6} cm/sec which was determined from laboratory tests. For these two zones, the permeability in the horizontal direction was assumed to be four times greater than in the vertical direction. Zones 2, 4, and 5 were assumed to have permeabilities significantly greater than zones 1 and 3.

Significant Findings Related to Design

The following have been identified as significant design items. These items relate to administrative and management procedures, site conditions, and possible improper technical evaluations by the designers.

The USBR designers did not consider the following items to be of major concern or they would have followed other courses of actions in the development of the design. The plans and specifications issued for construction reflect the USBR's judgment of what constituted a properly designed dam for the Teton site. Some of the following items may not have contributed to failure of the dam but reflect this task group's opinion that they were significant factors in the design of Teton Dam.

1. Design notes developed early in the design process identify and report a variety of potential design problems and possible design alternatives. There are no records, documents, or reports which show:

   a. The logical resolution of each of the identified design problems.

   b. Why a particular design alternative was considered satisfactory and selected in preference to others.

   c. Why an identified design problem was subsequently judged not to be serious and omitted from further consideration.

Obviously, the plans and specifications issued for construction received the concurrence of technical supervisory and management personnel. Because of the lack of documented rationale, it is not clear to what extent technical supervisory and management were involved during the design process.

2. Laboratory testing of embankment construction materials during design was minimal because the designers believed they possessed adequate knowledge of the materials due to past experiences with comparable materials at other damsites.

3. No laboratory permeability tests were performed on zone 2 material during design; however, the designers judged that this material would have sufficient water-carrying capacity to handle all normal seepage passing through zone 1 and also prevent zone 3 from becoming saturated. The designers did not envision zone 2 being required to convey large volumes of seepage. It was assumed that the zone 1 core would remain intact and not develop cracks due to differential settlement, hydraulic fracturing, or seismic activity. The designers also judged that adequate filter action could be provided by making zone 2 relatively wide (thick).

4. The designers judged that seepage exiting from the abutment rock downstream of the grout curtain would be minimal and could adequately enter the zone 2 blanket drain.

5. The designers judged that there was not sufficient precedence to use hydraulic gradient concepts to establish the minimum width of the abutment key trenches. The designers, therefore, judged that a theoretical maximum seepage gradient of 7.3 at the floor of the key trench at elevation 5100 was tolerable.

6. The designers judged that a treatment program for known defects in the surface rock beneath the dam could be best established after construction excavation exposed the rock.

7. The designers judged that the potential for migration of zone 1 fill into underlying or adjacent rock was insufficient to warrant the use of filter material between zone 1 and rock or to warrant sealing small openings in the rock with concrete or shotcrete.

8. The designers judged that their experiences at other dams were sufficient to adequately predict the performance of Teton Dam and that installation of instruments to measure foundation and embankment settlement, lateral movements, and to monitor piezometric pressures were unnecessary.

CONSTRUCTION REVIEW

Summary Of Information Obtained During Visit To Damsite

Inspection Organization

The embankment construction organization for inspection was excellent. An organization chart and staff names are shown in Appendix D1. The experience of the inspection staff was varied. Only a few of the embankment inspectors had prior experience in dam construction. Several had prior experience in earthwork construction, surveying, or laboratory construction control testing. Resumes of the project staff are included in Appendix D1.

Interviews of Embankment Inspectors

Inspectors were privately interviewed by the task group. The initial six interviews were conducted on August 3, 1976. The seventh interview and the second interview of Mr. Ringel and Mr. Hoyt, to confirm some of the task group findings while reviewing construction records, were conducted on August 5th. The persons interviewed, in order of first appearances, were Kenneth Hoyt, Jan Ringel, Glen Harris, Lyman Rogers, Richard McClung, Douglas Jarvie, and Stephan Johnson. Each person interviewed was very cooperative and judged to be responsive to the questions. The information obtained during the interviews is summarized by subject matter. The answers by different individuals were generally in agreement. Where conflicts in testimony were noted, all the different answers are reported.

Technical Qualifications of Personnel: The task group was impressed by the sincerity and conscientiousness of all the persons interviewed. Both principal inspectors (shift supervisors) were interviewed. One had no previous earth dam or earthwork experience. However, the most experienced supervisory embankment inspector was assigned to his shift. The other principal inspector had a career experience in inspection of earthwork construction. He had previous assignments at several earth dam projects. The three supervisory embankment inspectors interviewed had previous experience as earthwork inspectors. One had previous experience in earth dam construction. The second had previous earth dam experience as a laboratory technician. The third had no previous experience in earth dam construction. The earth placement inspectors interviewed had little previous earthwork experience and none had past experience in earth dam construction. Many of the earth placement inspectors were young persons learning construction inspection. At the time of these interviews, all were judged to be capable as first level inspectors in earth dam construction.

Controls to Obtain Quality Construction: All said that they believed they had inspected the construction of a good dam. The contractor had been cooperative with the inspector's requests. While some conflicts between contractor and inspectors had occurred, the contractor's performance and willingness to perform to USBR standards had been better than previously experienced on similar contracts. The inspectors complimented the Materials Engineering Branch (Project Laboratory) for its cooperation in obtaining field control tests when and where requested. The inspectors were authorized to work overtime so that inspection would be provided whenever the contractor was working. All inspectors interviewed confirmed that a personnel time overlap occurred between day and swing shifts for the purpose of verbal briefing concerning the status of work being performed. The earth placement inspectors, three per shift, spent all their work shift on the embankment. The supervisory embankment inspector, one per shift, would spend his total shift on earthwork features with approximately half his time in the borrow area and the other half on the embankment.

Excavation and Backfill of the Cut-Off Trench: The cutoff trench was specified to be excavated in the dry. Deep wells (to bedrock) were installed to dewater the excavation. The cutoff trench contained a deep silt stratum that complicated efforts to dewater ahead of excavation. The lower elevations of the cutoff trench were excavated from under water with a dragline. Once excavated, the area was dewatered by augmenting the established well system with ditches along the sides of the excavation and sump pumps. Gravel was placed at the toe of the excavated side slopes to provide continuous interception of seepage. The rock surface cleanup and backfilling was accomplished in the dry. At the base of the left abutment near the upstream side of the cutoff trench, a persistent spring was treated by providing gravel-covered drains and standpipes that were grouted when the backfilling reached sufficient height (elev. 5112). After operation of the dewatering system was discontinued, one seep penetrated to the surface of the backfill. Special dewatering and reconstruction of the area was provided. The cleanup of rock surfaces in the bottom of the cutoff trench removed all loose slabby and undesirable rock. In the basalt area at the base of the left abutment, washing was stopped as soon as the surface was clean because the 2-inch-diameter pressure water hose jet used would continue to loosen pieces of the closely jointed but otherwise sound basalt.

Excavation of the Key Trenches: The key trenches were excavated by alternate blasting and mucking activities. The blasting technique was to drill presplit holes along the boundary of the excavation on 2' to 3' centers. Production holes were drilled on patterns of 6' X 8' inside the presplit holes. The blasting used delay sequence. The sequence was to first shoot presplit holes, followed by center row holes, and then remaining production holes. The constructed depth of the key trench was approximately 68 feet. The excavation proceeded in lifts of 24 feet, 24 feet, and 20 feet. Due to the highly jointed nature of the rock, there was significant overbreak which resulted in an irregular rock surface particularly in the upper portions of the key trenches. No unusual weathering of the excavated rock surfaces occurred during the three-year exposure of the key trench rock. Some ravelling due to freeze-thaw action did occur.

Foundation Preparation and Treatment, Zone 1: The abutments under zone 1 were cleaned of all loose materials to firm sound rock. Two exceptions were noted. The first was an area of silt and rock on the right abutment approximately 200 feet upstream of the key trench and below approximate elevation 5050. This area extended under part of zone 1, zone 2, and zone 4. The decision not to remove this material was apparently made at a supervisory level above the embankment supervisor. The second area mentioned was on the right abutment downstream of the key trench somewhere between elevations 5200 to 5250 where a lot of loose rock was encountered. The procedure for general foundation preparation on the abutments was to remove as much material as possible with dozers working down the slope within the zone 1 contact area. Much of this work was performed during the 1974 construction season. When the canyon wall was too steep for dozers to operate, the contact area was cleaned with a backhoe working from the embankment surface as construction progressed. This cleaning removed all large loose rock and most of the overburden. Where firm intact rock formed overhangs, they were removed by machine excavation if possible or by blasting. Most of the overhangs that required removal by blasting were on the left abutment. One large right abutment overhang below a bench at elevation 5006 was removed from the dam centerline to 266 feet upstream. Prior to placing fill against the zone 1 abutment contact, the area was given final cleanup by air jets. Enough water to control dust was sometimes used in the air jet. The air-jet cleanup removed all unsound, undesirable materials that remained on the contact surface. Jackhammers were used to remove grout which had leaked to the surface of the key trench during curtain grouting activity.

The same contractor crew performed final cleanup work on both abutments. The cleanup was maintained one to five feet ahead of the fill placement elevation. After cleanup the inspector would mark the acceptable area of cleanup with a spray paint and mark the surface cracks and holes to be filled by gravity grouting. Most of the cracks that were grouted were oriented vertically. Drafts of cool air movement could be felt exiting from the rock at many of the holes marked for filling with gravity grout. There are some discrepancies concerning the extent of the gravity grouting performed. Gravity grout was placed on the right abutment under the zone 1 contact between elevation 5075± and 5205. The methods used to place gravity grout varied. Some gravity grout was placed directly from the chute on the truck. Other times gravity grout was placed through a short length of pipe inserted into the crack or hole to be filled. The openings selected to receive gravity were those too narrow or too deep to be backfilled with zone 1 material compacted with "pogo" sticks. Sometimes a small earth dike would be built around the opening to serve as a small retaining reservoir to facilitate the placement of gravity grout. Placement of specially compacted earthfill was not delayed if gravity grouting had not been completed; therefore, it was sometimes necessary to set a pipe so that the opening could be grouted from a higher elevation after a few lifts of fill were placed. No inspector recalled any opening marked for gravity grouting that was not grouted by one of these methods. No written criteria were available on which openings were to receive gravity grout or how gravity grout would be placed. The holes that received gravity grout were usually: (a) extensive in opening, (b) had air drainage or (c) had no limited depth when probed with a lath. Each hole was evaluated individually. Gravity grout placement was accomplished on the swing shift only; therefor, most cracks were seen by several inspectors. Generally any cracks too narrow to be backfilled with zone 1 material using "pogo" sticks and wider than 1/2 inch received gravity grout. The water-cement ratio by volume of the gravity grout was usually 0.7 to 0.8. Some inspectors, but not all, added sand and used a 1/1/1 mix if the gravity grout takes became large (usually greater than 4-1/2 to 10 cubic yards). Most gravity grouting under the zone 1 contact was done upstream or downstream of the key trench. The bottom of the key trench was described as sound rock with cracks adequately filled by the curtain grouting activity. The side slopes of the key trench excavation were described as blocky rock. Several inspectors said the cracks and holes in the side slopes of the key trench required little to no gravity grout. One inspector stated that the side slopes of the key trench required about the same amount of treatment as other areas under the zone 1 contact area. The filling of cracks and holes with gravity grout was not a separate contract pay item. The work was paid under the unit price for backfill concrete. The inspectors believed the price to be favorable for the contractor. The gravity grouting on the right abutment began at approximate elevation 5075. The inspectors generally agreed that there was no need for gravity grout treatment at lower elevations. According to Special Reports prepared by the inspectors, gravity grout treatment on the right abutment ended at elevation 5205. None of the inspectors interviewed remembered this elevation. Two did remember the month and year, August 1975. The decision to stop the gravity grout treatment was made by personnel above the level of principal inspectors. One inspector thought the decision was made by the Project Construction Engineer, Mr. Robison. No inspector knew of any USBR Denver office participation in the decisions to provide or to terminate the gravity grout treatment. The reasons the inspectors gave for the decision to stop the gravity grout treatment varied. Inspector's opinions varied as to whether the rock above elevation 5205 had more or fewer cracks and holes than the rock that received gravity grout at lower elevations.

All inspectors who discussed this subject gave a reduced reservoir head above this elevation as one of the reasons given to them for discontinuation of the gravity grout treatment. All inspectors agreed gravity grout placement was expensive work. One said the contractor complained about gravity grout treatment because it interfered with his schedule for topping out the embankment in 1975. Others said the contractor liked the gravity grout treatment because of his "dollars mark-up" in the item.

Foundation Preparation and Treatment for Zone 2: The foundation preparation for zone 2 removed all material with vegetation and all loose or soft material. Material that contained small roots was removed. The remaining overburden material was firm and impervious and hard to dig with a hand shovel. All inspectors who discussed this said that as best they could remember the downstream zone 2 contact surface on the right abutment was about 50% rock and 50% overburden. The Project Construction Engineer, Mr. Robison, indicated to the IRG and two members of this task group on February 8, 1977, that he disagreed with the above reported amount of impervious overburden remaining on the right abutment. He indicated that very little impervious overburden remained and that most of the overburden was pervious accumulations of rock.

Specially Compacted Earthfill: Specially compacted earthfill on the abutments was required only for zone 1 materials. Specially compacted earthfill was usually placed in the following manner. Zone 1 material would be dumped adjacent to the placement area. When necessary, the material would have water added and be mixed by means of a motor patrol. An effort was made to maintain the special earthfill material near optimum moisture content. After mixing the material and wetting the adjacent rock surface, the fill material was spread into 9-inch loose lifts. The lift height was controlled by using a lath to penetrate the lift and painting elevation marks on the rock surface. The lift would then be compacted with a sheepsfoot roller if possible. When a lift was the first to be placed on rock, it was spread to a 6-inch thickness and rolled with rubber-tired equipment. Where a sheepsfoot roller could not roll next to a steep rock slope, the compaction was accomplished with a loaded rubber-tired hauling truck. The boundary areas that could not be compacted by this equipment were compacted with a hydrohammer, wacker, or "pogo stick." Large holes and small rock overhangs were backfilled with zone 1 material compacted with "pogo sticks" or wacker power tampers. A hydrohammer plate mounted on a 3/4-ton chassis was said to be the best of the small size compactors. The surface area of a compacted lift would be scarified and the placement and compaction procedure repeated. A hand shovel was used to scarify the boundary areas that could not be scarified with disc, dozer mounted drag teeth, or dozer tracking. The inspectors selected the areas for the laboratory to perform control tests. Inspectors used the following criteria for selecting the locations for earthwork control testing: (1) areas where the compactive effort was suspected to be less than adequate, and (2) areas that would locate tests representatively over the material zone. Several inspectors said the construction sequence maintained the specially compacted earthfill at the rock surface boundary slightly above the adjacent fill and provided a gentle slope from the specially compacted earthfill areas downward toward the main embankment areas. One inspector agreed that this was the intent, but because of the slow nature of the work required at the abutments, the specially compacted earthfill was often 2 feet lower than the adjacent main embankment zone 1 fill. Zone 1 placement was temporarily halted when it was 3 feet higher than the specially compacted earthfill.

Specially compacted earthfill was placed around the four column legs of the material handling tower which were permitted to remain in the fill. The tower was located at the base of the right abutment immediately downstream of the key trench. (Station 16+75, 60± feet upstream of the dam centerline with footings at approximate elevation 5006.) This tower was used for filling of zone 1 hauling trucks throughout the 1974 construction season. Approximately one-half of the zone 1 material was transported to the embankment through the tower. At the end of the 1974 construction season, the exposed portion of the tower was disassembled. The column legs were cut off at the embankment fill height.

Placement Procedure, Zone 1: The routine placement procedure for zone 1 was to follow scarifying the dumping, spreading, sprinkling and mixing if necessary, scarifying, and compaction with a sheepsfoot roller. The compacted area was then scarified and the above procedure repeated. Scarifying was accomplished with a disc (12"± dia) or with drag teeth mounted on the back of a dozer. When mixing was required, a larger disc (14"± dia) that was weighted to increase the depth of penetration was employed. Mr. Aberle said, at a later date, that the diameters of the discs were 18 and 24 inches, respectively. The sheepsfoot rollers used were self-propelled four wheel drum Caterpillar 825-B, and single drum Ferguson SP-120-D. The inspectors concentrated their attention on moisture control and number of roller passes. The zone 1 material was of uniform appearance, but varied greatly in natural water content and optimum moisture content. The material arriving from the borrow area was extremely difficult to judge for water content. The inspectors requested many water content tests. The project laboratory used a microwave oven to rapidly determine water contents.

The contractor also made water content tests with a speedy moisture teller. Water content tests for final acceptance testing were made in standard drying ovens in the project laboratory. The inspectors selected the locations for field control tests (in-place density and water content). The same test location criteria described for specially compacted earthfill were used for determining the location for the control tests. The sand cone density test and USBR Rapid Compaction Control procedures were employed in the field control tests. When a control test indicated a fill area did not conform with the specification requirements, the material in the area was scarified and reworked or removed to zone 3. After recompaction the area was retested. When anticipating a rain storm or at the end of a construction season, the fill surface was sealed using rubber-tired equipment. Preparation for fill placement after a period of sealing included the removal to zone 3 of all excessively wet material and scarifying of the underlying surface until it became satisfactory for fill placement. The material removed to zone 3 would be spread and aerated and returned to zone 1 when it was satisfactory for placement into zone 1.

Placement Procedure, Zone 2: The routine placement procedure for zone 2 was to dump and spread the material in 12" thick loose lifts and to compact by dozer tracking or vibratory roller. The dozer tracking and vibratory roller were used intermittently. The vibratory rollers were often down for maintenance. Zone 2 material was extended up the downstream abutments to form a blanket drain under zone 3. The zone 2 material was placed one scraper dumping width and compacted with dozer tracking in a direction parallel to the abutment. A dozer was used for all compaction against the abutments because the tracks could get next to the surface of the prepared foundation. The inspectors concentrated on preventing contamination of zone 2 with zone 1 material and on controlling lift thickness. Construction traffic tracked adjacent to the zone 2 abutment blanket drain throughout most of the contract.

Inspection and Interviews of Project Laboratory

On August 4, 1976, an inspection was made of the project laboratory. Ralph Mulliner, Chief, Materials Engineering Branch, guided our inspection and discussed the laboratory's capabilities and methods of testing.

The laboratory has soil testing capability which includes the following:

Classification
Moisture content
Gradation (Sieve and Hydrometer analyses)
Specific gravity
Atterberg limits
Compaction (including mechanical tamper)
Relative Density (vibratory table)
Settlement-Permeability (constant head)
In-Place density (sand cone)

Testing of Zone 1 Fill Material: The laboratory monitored zone 1 fill placement by routine control testing which included: (a) in-place density tests by use of the sand cone method (USBR Designation E-24) and (b) rapid compaction control (USBR Designation E-25). During construction of the dam, these tests were performed at a frequency of approximately one set of tests per 1,900 cubic yards of compacted fill placed. At less frequent intervals, approximately once for every 34,000 cubic yards of compacted fill placed, the laboratory performed record tests which included: (a) in-place density, (b) rapid campaction control, and (c) combined laboratory permeability and settlement tests (USBR Designation E-13). Undisturbed block (cube) samples of compacted zone 1 fill were not obtained during construction of the dam.

Testing of Zone 2 Fill Material: Zone 2 fill compaction was monitored by in-place density tests (USBR Designation E-24). Laboratory relative density tests, by vibratory table procedures (USBR Designation E-12), were performed by the project laboratory to establish control criteria for acceptance testing. In addition, combined laboratory permeability and settlement tests (USBR Designation E-14) were made on laboratory compacted samples of zone 2 fill materials by the project laboratory. Approximately one combined permeability and settlement test was made for every 97,000 cubic yards of zone 2 fill material placed.

Testing of Zone 3 Fill Material: Zone 3 fill compaction was monitored by in-place density tests (USBR Designation E-24). Compaction control criteria were established by the rapid compaction control method or relative density, depending upon the nature of the material being placed.

The USBR *Earth Manual,* Second Edition, describes each of the above tests in significant detail. Therefore, the test procedures will not be repeated here.

The project laboratory appeared to be well equipped and staffed with qualified personnel. Project personnel associated with construction of the dam embankment were complimentary of the laboratory staff.

Stripping, Foundation Preparation, and Rock Surface Treatment

Within this section an attempt has been made to summarize all written information (exclusive of contract drawings and logs of subsurface investigations) that was available to the constructors in published documents and that would have direct influence on the stripping or foundation treatment to be performed. The documents that are quoted are DESIGN CONSIDERATIONS FOR TETON DAM, OCTOBER 1971 and SPECIFICATIONS NO. DC-6910, TETON DAM AND POWER AND PUMPING PLANT, VOLUME 1, as amended. Many of the details of actual foundation treatment not specified to be performed are not repeated from the section titled Interviews of Embankment Inspectors. Familiarity with that section will help in review of this section.

Design Considerations: The applicable quotes from the DESIGN CONSIDERATIONS FOR TETON DAM, OCTOBER 1971, are summarized at the end of this text in Exhibit A, pages 1 and 2. This exhibit is a tabular display that relates foundation design intent to the appropriate area of the foundation.

Specification Requirements: The applicable quotes from SPECIFICATIONS No. DC-6910, VOLUME 1, are summarized at the end of this text in Exhibit B, pages 1, 2, and 3. This exhibit is a tabular display that relates specification requirements for foundation work to the foundation area of application.

Construction Practices:

1. Stripping.—Because the results of stripping operations were not suspect or directly contributory to failure in any of the postulated causes for failure, the interview questions and review of construction data concerning stripping were rather general. Foundation stripping was accomplished using bulldozers. When abutment slopes were too steep for bulldozer operations, stripping was performed with a backhoe after the embankment construction progressed to an elevation that would provide a working platform. No available information indicates any areas where large or deep zones of soft or weak surface materials were permitted to remain. At the failure location, the zone 1 contact area was excavated to formation. The zone 2 and 5 contact areas were stripped and the overburden that remained was described as firm, impervious, and difficult to dig with a hand shovel. All stripping was measured by the cubic yards excavated for payment as "Excavation for dam embankment foundation . . . "

2. Foundation preparation and rock surface treatment.—Except where the zone 1 contact area was on formation, the foundation preparation for all areas was accomplished by stripping and initial fill placement. The stripping work provided some shaping and the initial fill placement backfill confined irregularities. Some zone 2 backfill materials were placed into confined areas by sluicing. Foundation preparation complied to specification requirements for zones 2, 4, and 5 (zone 3 did not contact bedrock). Specified foundation preparation work under the dam embankment was accomplished as a subsidiary obligation of embankment construction and was not measured for separate payment.

a. Zone 1, cutoff trench. The construction practice, except for excavation of the lower portion of the trench in the wet, was in accordance with specifications. After excavation, the dewatering system was able to control seepage. One persistent spring required special treatment. The rock surface treatment was performed on contact surfaces that were free of standing water. The zone 1 contact surface was cleaned using a 2-inch-diameter pressure water hose jet.

Less erosive surface cleaning was performed on the closely jointed basalt contact area. Holes, depressions, and irregularities were filled with specially compacted earthfill using hand-operated power tampers. No special sealing treatment of rock surfaces was specified or performed. All rock surface treatment was accomplished as a subsidiary obligation of embankment construction and was not measured for payment.

b. Zone 1, abutments. Trimming of contact area slopes to 1H on 2V or flatter was required only for zone 1. Photographs taken during construction show local areas where final formation slopes were steeper. There were on the right abutment two areas reported by inspectors where "shall be cleaned of all loose, soft, and . . . " [Specifications, paragraph 68.a.(2)] was not accomplished. These two areas were a silt and rock mixture approximately 200 feet upstream of the key trench below elevation 5100, and a jointed loose rock material downstream of the key trench between elevation 5200± to 5250±. Other than these exceptions, construction was in general accordance with specifications. The final cleanup of formation under the zone 1 contact area was performed with air jets. Holes, cracks, depressions, and

Stripping, Foundation Preparation and
Rock Surface Treatment

DESIGN CONSIDERATIONS FOR TETON DAM, OCTOBER 1971.

Page	Design Consideration	Stripping: Zone 1 Valley	Stripping: Zone 2 Valley Upst	Stripping: Zone 2 Valley Dnst	Stripping: Zone 2 Abut	Stripping: Zone 4 Valley	Stripping: Zone 4 Abut	Stripping: Zone 5 Valley Upst	Stripping: Zone 5 Valley Dnst	Stripping: Zone 5 Abut	Foundation Preparation & Rock Surface Treatment: Zone 1 Cutoff Valley	Foundation Preparation & Rock Surface Treatment: Zone 1 Cutoff Abut	Foundation Preparation & Rock Surface Treatment: Zone 1 Key Trnch	Foundation Preparation & Rock Surface Treatment: Zone 2	Foundation Preparation & Rock Surface Treatment: Zone 4	Foundation Preparation & Rock Surface Treatment: Zone 5
2	"III. Foundation Excavation"															
4	"B. Stripping															
	....stripped to uncover material equal in strength to the overlying embankment materials.	X	X		X	X	X	X		X						
	Additional considerations are that the valley floor downstream, underlying Zones 2 and 5, should be stripped to expose Zone 2 type material....			X					X							
	....and that the abutments should be graded to provide a reasonably smooth surface that will permit proper compaction of the embankment against the foundation with little or no special compaction....				X		X			X						
	....the foundation below elevation 5040...will have to be stripped to an average depth of approximately 3 feet."	X	X			X		X								
5	"....On the abutment...depth of excavation...was estimated to be 5 feet on the right side and 3½ feet on the left side.				X		X			X						
	....above the canyon rim...stripping...would average less than 2 feet.				X					X						
	....field forces should be alert to secure an acceptable foundation with the minimum amount of stripping...."	X	X		X	X	X	X		X						
6	"C. Cutoff Trench"															
7	"....On the abutments the cutoff trench is to be excavated through weathered and loosely open jointed rock to a firm...											X				
	...irregularities are to be removed and the surface sloped to 0.5:1 (1H on 2V)											X				
	...excavation could be accomplished by bulldozer and ripper...."											X				
8	"....(Field forces should be alert to secure an acceptable foundation with the minimum amount of stripping)...also apply to the depth the cutoff trench is to be extended into rock."										X	X				

NOTE: "X" denotes area of dam foundation to which Design Consideration information or Specifications requirements applies

Page	Stripping, Foundation Preparation and Rock Surface Treatment	Stripping: Zone 1 Valley	Stripping: Zone 2 Valley Upst	Stripping: Zone 2 Valley Dnst	Stripping: Zone 2 Abut	Stripping: Zone 4 Valley	Stripping: Zone 4 Abut	Stripping: Zone 5 Valley Upst	Stripping: Zone 5 Valley Dnst	Stripping: Zone 5 Abut	Foundation Preparation & Rock Surface Treatment: Zone 1 Cutoff Valley	Foundation Preparation & Rock Surface Treatment: Zone 1 Cutoff Abut	Foundation Preparation & Rock Surface Treatment: Zone 1 Key Trnch	Foundation Preparation & Rock Surface Treatment: Zone 2	Foundation Preparation & Rock Surface Treatment: Zone 4	Foundation Preparation & Rock Surface Treatment: Zone 5
8	"D. <u>Foundation Key Trench</u>"															
9	"....Excavation method shall be controlled to preserve the material in the side and bottom of the key trench in the soundest possible condition."												X			
	"IV. <u>Foundation Pressure Grouting</u>.															
	This topic has been the subject of the Grouting Subcommittee. Paragraphs H <u>Blanket Grouting</u> and <u>I. Open Joints, Cracks, and Springs</u> discuss techniques that could be applied for foundation preparation and rock surface treatment, but were restricted to uses in conjunction with construction of the grout curtain."															

Stripping, Foundation Preparation and Rock Surface Treatment

	Stripping									Foundation Preparation & Rock Surface Treatment					
	Zone 1	Zone 2			Zone 4		Zone 5			Zone 1			Zone 2	Zone 4	Zone 5
	Valley*	Valley		Abut	Valley	Abut	Valley		Abut	Cutoff		Key Trnch			
		Upst	Dnst				Upst	Dnst		Valley	Abut				
SPECIFICATIONS NO. DC 6910, VOLUME 1															
"67. OPEN-CUT EXCAVATION, GENERAL															
a. General...All necessary precautions, including control of blasting, shall be taken to preserve the material below and beyond the established lines of all excavation in the soundest possible condition...										X	X	X			
All excavation for embankment and structure foundations shall be performed in the dry....	X	X	X	X	X	X	X	X	X	NO	X	X	X	X	X
e. Measurement and payment - ....Measurement, for payment, of excavations... will be limited to the neatlines shown on the drawings or,...established by the contracting officer...	X	X	X	X	X	X	X	X	X						
....the unit prices bid in the schedule for excavation in open-cut shall include the cost of all labor, equipment, and materials...and all other work necessary to maintain the excavations in good order."	X	X	X	X	X	X	X	X	X	X	X	X			
"68. EXCAVATION FOR DAM EMBANKMENT FOUNDATION															
a. General - Excavation for dam embankment... includes all: (1) Stripping for foundation of dam embankment.	X	X	X	X	X	X	X	X	X						
(2) Excavation below stripping for foundation of dam embankment...			X					X		X	X	X			
Loose rock shall be removed from foundation contacts and rock cliffs, ledges, overhangs and sharp irregularities shall be reduced to provide satisfactory foundation contours.										X	X	X	X	X	X
The finished foundation contours against which the earthfill, Zone 1 portion of the dam embankment is to be placed shall be not steeper than 1/2:1, and shall be cleaned of all loose soft, and disintegrated materials, including removal of such materials from pockets, irregularities, fissures, and depressions in the foundation.											NO	X			
b. Stripping...The entire area to be occupied by the dam embankment,...shall be stripped to a sufficient depth, as determined by the contracting officer, to remove all unsuitable materials. The unsuitable materials to be removed by stripping shall include all surface boulders and loose rock; debris; topsoil, vegetable matter, including stumps and roots; and all other perishable and objectionable materials that are	X	X		X	X	X	X		X						

*Zone 1 contact area on the steep abutment was excavated to formation.

"NO" indicates a location where compliance to a specification requirement is known or believed not to have occurred. These particular specification requirements are underlined.

Stripping, Foundation Preparation and
Rock Surface Treatment

	Stripping: Zone 1 Valley	Stripping: Zone 2 Valley Upst	Stripping: Zone 2 Valley Dnst	Stripping: Zone 2 Abut	Stripping: Zone 4 Valley	Stripping: Zone 4 Abut	Stripping: Zone 5 Valley Upst	Stripping: Zone 5 Valley Dnst	Stripping: Zone 5 Abut	Foundation Preparation & Rock Surface Treatment: Zone 1 Cutoff Valley	Foundation Preparation & Rock Surface Treatment: Zone 1 Cutoff Abut	Foundation Preparation & Rock Surface Treatment: Zone 1 Key Trnch	Foundation Preparation & Rock Surface Treatment: Zone 2	Foundation Preparation & Rock Surface Treatment: Zone 4	Foundation Preparation & Rock Surface Treatment: Zone 5
unsuitable for use in permanent construction or that might interfere with the proper bonding of the embankment with the foundation, or....															
c. Excavation below stripping for cutoff trench and foundation key trench...The cutoff trench shall be excavated to firm formation, and the foundation key trench shall be excavated into the firm formation.										X	X	X			
d. Measurement and payment -															
Payment for excavation for dam embankment foundation will be made at the applicable unit price per cubic yard bid in the schedule for excavation for dam embankment foundation above elevation 5040, and for excavation for dam embankment foundation below elevation 5040, and shall include all costs as provided in Paragraph 67 and all costs of cleanup of foundation surfaces...."	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
EMBANKMENT															
"86. EMBANKMENT CONSTRUCTION, GENERAL															
b. Foundation preparation - No material shall be placed in any portion of the dam embankment until the foundation for each section has been unwatered, stripped, suitably prepared, and has been approved by the contracting officer...										X	X	X	X	X	X
All cavities, depressions, and irregularities, shall be filled with embankment materials and compacted as specified for the overlying embankment.										X	X	X	X	X	X
c. Placing embankment materials - The suitability of each part of the foundation...will be determined by the contracting officer. No embankment material shall be placed...when...the foundation...is frozen.										X	X	X	X	X	X
d. Measurement and payment -															
Payment under all items of embankment construction shall include the costs of preparing the embankment foundations...."										X	X	X	X	X	X

Stripping, Foundation Preparation and Rock Surface Treatment

	Stripping									Foundation Preparation & Rock Surface Treatment					
	Zone 1	Zone 2			Zone 4		Zone 5			Zone 1			Zone 2	Zone 4	Zone 5
	Valley	Valley		Abut	Valley	Abut	Valley		Abut	Cutoff		Key Trnch			
		Upst	Dnst				Upst	Dnst		Valley	Abut				
" 87. EARTHFILL IN DAM EMBANKMENT, ZONE 1 c. Preparation of foundations...Immediately prior to placing the first layer of earthfill, all surfaces upon or against which the earthfill portions of the dam embankment are to be placed shall be cleaned of all loose and objectionable materials in an approved manner by handwork, barring, picking, brooming, sluicing, or by other effective means. Such surfaces shall have all water removed from depressions and shall be properly moistened and sufficiently clean to obtain a suitable bond with the earthfill."										X	X	X			
" 89. SAND, GRAVEL, AND COBBLE FILL IN DAM EMBANKMENT, ZONE 2 c. Preparation of foundations - The foundation for the Zone 2 shall be prepared by leveling, moistening, and compacting so that the surface materials will be as compact as specified for subsequent layers of the Zone 2."													X		
"91. SILT, SAND, GRAVEL, AND COBBLE FILL IN DAM EMBANKMENT, ZONE 4 c. Preparation of foundations - The foundation for the Zone 4 shall be prepared by leveling, moistening, and compacting so that the surface materials will be as compact as specified for subsequent layers of the Zone 4."														X	
" 92. ROCKFILL IN DAM EMBANKMENT, ZONE 5 (No specified requirement for preparation of foundations)"															X

irregularities were filled with specially compacted earthfill, compacted with hand-operated power tampers. No special sealing treatment of the formation contact surface was specified. Openings or cracks that could not be backfilled using a "pogo stick" (power hand tamper) were filled with gravity grout. In general, cracks 1/2 inch or wider were filled with gravity grout. The gravity grout filling treatment of the formation surface on the right abutment began at elevation 5075± and discontinued at elevation 5188 downstream of the key trench and at elevation 5205 upstream of the key trench. The gravity grout work was measured and paid by the cubic yard placed under the contract unit price for backfill concrete. More details of the gravity grout work are described in the section of this report titled Interviews of Embankment Inspectors.

Embankment Materials And Construction

Zone 1: Zone 1 of Teton Dam was designed to serve as the principal water barrier within the embankment. The central core, the foundation cutoff trench, and the abutment key trenches were filled with compacted zone 1 material.

1. Source—Material for zone 1 fill was obtained from borrow area A. This borrow area was located on a plateau above the Teton River canyon on the northwest side of the river. This soil was deposited by nature as wind-blown sediments and is called loess. Borrow area B was an approved borrow area, but was not used as a source of zone 1 fill material.

2. Material Description and Engineering Properties—Zone 1 fill was predominantly a silt (ML) of low plasticity. Limited amounts of clay and silt mixtures classifying as CL-ML were used in zone 1. Figure D5-3 of Appendix D5 shows the low plasticity characteristics of the zone 1 fill material. Based on more than 125 laboratory tests, the average specific gravity of zone 1 soil from borrow area A was determined to be 2.62. The material was relatively uniform in texture and contained very few rock fragments or gravel. Typically, 83 percent or more of the soil passed through a No. 200 sieve (0.074-mm opening). Laboratory permeability tests indicated the compacted soil was practically impervious. One laboratory permeability test made on compacted soils from borrow area A prior to construction of the dam gave a permeability of 0.32 ft/yr ($3. \times 10^{-7}$ cm/sec). The average of 147 permeability tests made on laboratory compacted samples obtained from zone 1 of the dam during construction was 0.5 ft/yr ($5. \times 10^{-7}$ cm/sec). One triaxial compression test made during design on a compacted soil composited from borrow area A supplied the following drained (effective) shear strength parameters: Angle of internal friction equal to approximately 32 degrees, cohesion intercept equal to approximately 1,600 PSF.

3. Placement Procedures—Excavation of zone 1 fill material in borrow area A was accomplished with a Barber Greene Wheel excavator and Caterpillar scrapers. The material was hauled to the dam embankment in scrapers or belly-dump trucks. A conveyor system which connected to a materials handling tower located near the base of the dam's right abutment was used to transport approximately one-half of the zone 1 fill material placed prior to the end of the 1974 construction season.

a. Routine Compaction—After dumping, the fill material was normally spread with a Caterpillar D-8 dozer. The material was spread in loose horizontal layers to a thickness that would result in a layer thickness of approximately 6 inches after compaction. Each layer was compacted with 12 passes of a tamping (sheepsfoot) roller. The rollers used were self-propelled, 4-drum, Caterpillar 825-B and self-propelled single-drum Ferguson SP-120D. The upper surface of each compacted layer was scarified with an 18-inch-diameter disc or with teeth mounted on the back of a dozer prior to placement of the overlaying layer. Sprinkling and mixing of a layer with a 24-inch-diameter disc was done as necessary.

b. Specially Compacted Earthfill—Special compaction was performed on zone 1 fill that was adjacent to surfaces where compaction with large tamping rollers was impractical, e.g., against abutment key trench walls, against steep and irregular abutment areas, and around the column legs of the materials handling tower which were permitted to remain in the fill. This material was spread in 4- to 6-inch loose layers and compacted. Compaction was accomplished with pneumatic rams, hydrohammer (Arrow Model HD-1250), plate tampers or rubber tire wheel rolling. Rubber tire wheel rolling was done with heavily loaded end dump trucks or front-end loaders. Compacted layers were normally scarified with hand shovels or dozer cleats prior to placement of overlying layers. Additional details on specially compacted earthfill can be found in the section entitled Interviews of Embankment Inspectors.

4. Construction Control

a. Moisture Control—The specifications required that all acceptable zone 1 fill have a moisture content between 3.5 percent dry of and 1 percent wet of standard optimum moisture content. In addition, no more than 20 percent of the material could be drier than 3 percent dry of standard optimum moisture content. Likewise, no more than 20 percent of the material could be wetter than 0.5 percent wet of standard optimum moisture content. The average moisture content of all accepted fill was required to be between 0.5 and 1.5 percent dry of standard optimum moisture content.

b. Density Control—The specifications required that all acceptable zone 1 fill be compacted to at least 94 percent of standard maximum dry density. In addition, no more than 20 percent of the fill could have dry densities less than 95 percent of standard maximum dry density. The average dry density of all accepted fill was to be not less than 98 percent of standard maximum dry density.

c. Control Testing—Zone 1 fill placement was monitored by routine control testing which included: (a) in-place density tests by the sand cone method (USBR Designation E-24) and (b) rapid compaction control (USBR Designation E-25). In addition to the above two routine tests, the laboratory occasionally performed combined laboratory permeability and settlement tests (USBR Designation E-13) on record samples. Figures D5-4 and D5-5 of Appendix D5 are statistical plots of density control and moisture control for all acceptable zone 1 fill placed in the dam embankment. (These two statistical plots do *not* contain test results on unacceptably placed fill which was subsequently reworked to meet specification requirements.) The USBR *Earth Manual* requires at least one in-place density test for every 2,000 cubic yards of compacted earthfill placed. During construction of the dam, in-place density tests were performed at a frequency of approximately one test per 1,900 cubic yards of acceptable compacted zone 1 earthfill placed.

(1) Routine Compaction—Approximately 2,167 in-place density tests were made on routine zone 1 fill, which represents a testing frequency of one test per 2,370 cubic yards of acceptable material placed. The average degree of compaction obtained was approximately 98.6 percent of standard maximum dry density, and the average placement moisture content was approximately 1.2 percent dry of standard optimum moisture content.

(2) Specially Compacted Earthfill—Approximately 568 in-place density tests were made on special compacted zone 1 fill, which represents a testing frequency of one test per 88.5 cubic yards of acceptable material placed. The average degree of compaction obtained was approximately 97.3 percent of standard maximum dry density, and the average placement moisture content was approximately 0.5 percent dry of standard optimum moisture content.

Zone 2: The major portion of Teton Dam immediately upstream of the impervious core was designated as zone 2. In addition, zone 2 formed a blanket and chimney drain in the downstream portion of the dam.

1. Source—Material for zone 2 fill was obtained from borrow area C and from required excavation for the cutoff trench for the dam. Borrow area C was located upstream of the dam in the Teton River flood plain. Material in borrow area C was geologically young alluvial deposits. Since borrow area C was subject to flooding during periods of high flow in the river and during diversion through the river outlet works, the excavated material not placed directly into the embankment was stockpiled on the plateau above the left abutment.

2. Material Description and Engineering Properties—Zone 2 fill was a mixture of sand and gravel with little fines. The material classified as GW or GP. Based upon approximately 232 laboratory dry sieve analyses, the average percent passing through a No. 200 sieve (0.074-mm opening) was approximately 4 percent. Figure D5-21 of Appendix D5 shows the grain size distribution of zone 2 material in the right abutment area. The average specific gravity of zone 2 material passing a 3-inch sieve was determined by 231 laboratory tests to be 2.63. Figure D5-25 of Appendix D5 shows the results of 22 laboratory permeability tests on compacted zone 2 material. The majority of these laboratory permeability tests indicated compacted zone 2 material to be poor draining.

3. Placement Procedures—Zone 2 material was transported to the dam embankment in Caterpillar scrapers or by a conveyor system. The material was spread with Caterpillar D-9 dozers in loose horizontal layers to a thickness that resulted in a

maximum layer thickness of approximately 12 inches after compaction. Compaction was achieved with Caterpillar D-8 and D-9 crawler-type dozers, an Ingersoll-Rand L-60 vibratory roller, or a Rascal 700A self-propelled vibratory roller. Four complete passes of the treads of the crawler-type equipment was used for each layer. The number of vibratory roller passes used is unknown. Sluicing was used to place some zone 2 material in areas of limited access.

4. Construction Control

a. Moisture Control—There were no specific requirements for moisture content control for zone 2 fill. The specifications required that the moisture content be sufficient to attain the maximum relative density.

b. Density Control—The specifications required that all acceptable zone 2 fill be compacted to at least 65 percent relative density. In addition, no more than 20 percent of the fill could have relative densities less than 70 percent.

c. Control Testing—Zone 2 fill placement was monitored by in-place density tests by the sand cone method (USBR Designation E-24). Laboratory relative density tests, by vibratory table procedures (USBR Designation E-12), were performed by the project laboratory to establish control criteria for acceptance testing. In addition, combined laboratory permeability and settlement tests (USBR Designation E-14) were made on laboratory compacted samples. The USBR *Earth Manual* does not specify a frequency for in-place density tests for compacted pervious fill. Approximately 232 in-place density tests were made on zone 2 fill, which represents a testing frequency of approximately one test per 9,200 cubic yards of fill placed. The 232 tests indicated an average relative density of approximately 94 percent and an average moisture content of approximately 7 percent. The tests showed almost identical compaction results were achieved for the blanket drain and all other zone 2 areas above the foundation.

Zone 3: Zone 3 of Teton Dam was essentially a miscellaneous or random fill zone and was located in the downstream portion of the dam. The purpose of this zone was to provide a useful fill area for much of the material from required excavations and for material from the borrow areas unsuitable for other embankment zones.

1. Source—Material for zone 3 fill was obtained from a wide variety of sources: dam cutoff trench excavation, tailrace channel excavation, tunnel excavation, road excavation, dam abutment excavation, dam key trench excavation, zone 5 stockpile, and borrow area A. Material from borrow area A included soil meeting zone 1 criteria and other soil unsuitable for zone 1 due to the presence of caliche and calcareous materials.

2. Material Description—Zone 3 fill was highly variable due to the variety of sources. The material varied from silt, to sand and gravel, to caliche, to rock. Boulders and rock fragments larger than 12 inches were not permitted in zone 3. Fragments of caliche and hard calcareous material larger than 12 inches were permitted in zone 3 provided the fragments broke down to less than 12 inches during compaction. Most of the material sources from required excavations were depleted by September 1974, at which time zone 3 fill was at approximate elevation 5100. Therefore, almost all zone 3 fill above this elevation was composed of silty soils from borrow area A.

3. Placement Procedures—Zone 3 fill was normally hauled to the embankment in Euclid or Caterpillar end-dump trucks and spread with D-8 dozers. The material was spread in loose horizontal layers to a thickness that would result in a maximum layer thickness of approximately 12 inches after compaction. Depending upon the type of material being placed and availability of equipment, the following compaction equipment was used: 50-ton pneumatic-tired rollers pulled with Caterpillar D-8 or D-9 dozers, Caterpillar 825-B self-propelled tamping rollers, Euclid end-dump trucks, and dozers.

4. Construction Control

a. Moisture Control—There were no specific requirements for moisture content control for zone 3 fill. The specifications required that the fill material have the most practicable moisture content required for compaction purposes.

b. Density Control—There were no specific requirements on the percent of compaction for zone 3 fill. For a 50-ton pneumatic-tired roller in particular, the specifications contained a procedural requirement of 6 complete passes for each layer of fill. The specifications did not give procedural requirements for other compaction equipment.

c. Control Testing—Zone 3 fill compaction was monitored by one in-place density test (USBR Designation E-24) for approximately every 3,700 cubic yards of acceptable fill placed. The USBR *Earth Manual* does not specify a frequency for in-place density tests for compacted miscellaneous fills. Compaction control criteria were established by the rapid compaction control method or relative density, depending upon the nature of the material being placed. Seven in-place density tests on granular zone 3 fill indicated an average relative density of approximately 91 percent. Based upon 202 in-place density tests on materials controlled by the rapid method, the average degree of compaction was approximately 97.5 percent of standard maximum dry density.

Zone 4: Zone 4 of Teton Dam was an upstream diversion cofferdam which remained in place to become a permanent part of the dam embankment.

1. Source—Fill material for zone 4 was obtained from borrow area C, tailrace channel excavation, and excavations for the dam's cutoff and key trenches.

2. Material Description—Zone 4 fill materials were variable, but primarily granular in nature. This fill was composed of sand, gravel, small cobbles, some rock, and some silt.

3. Placement Procedures—Zone 4 material obtained from borrow area C was excavated with Caterpillar 641-B scrapers and D-9 dozers. Zone 4 material was hauled to the fill area in Caterpillar scrapers and Euclid end-dump trucks and spread with Caterpillar D-8 and D-9 dozers. The material was spread in loose horizontal layers to a thickness that would result in a maximum layer thickness of approximately 12 inches after compaction. Compaction was achieved with hauling equipment travel, tracking with Caterpillar D-9 dozers, and with 50-ton pneumatic-tired rollers pulled with Caterpillar D-8 and D-9 dozers.

4. Construction Control

a. Moisture Control—There were no specific requirements for moisture control for zone 4 fill. The specifications required the placement moisture content to be the optimum amount required to obtain the maximum dry unit weight of the material in place when compacted.

b. Density Control—There were no specific requirements on the percent of compaction for zone 4 fill. For a 50-ton pneumatic-tired roller in particular, the specifications contained a procedural requirement of 6 complete passes for each layer of fill. This specification did not give procedural requirements for other compaction equipment.

c. Control Testing—Zone 4 fill compaction was monitored by in-place density tests (USBR Designation E-24). One in-place density test was made for approximately every 6,800 cubic yards of zone 4 fill placed. Laboratory relative density tests, by vibratory table procedures (USBR Designation E-12), were made by the project laboratory to establish control criteria for acceptance testing. The results of 87 in-place density tests on the zone 4 cofferdam revealed an average relative density of approximately 93 percent.

Zone 5: The thin outer shells of rockfill of Teton Dam were designated as zone 5.

1. Source—Rock for zone 5 rockfill was obtained from required tunnel excavation, spillway excavation, and excavation for the dam's key trenches. In addition, oversize rock dumped in zone 3 was removed to zone 5.

2. Material Description—Rock for zone 5 consisted of rhyolite tuffs which were native to the damsite. These materials are described in the appendix of this report prepared by the geology task group.

3. Placement Procedures—Zone 5 material was hauled to the fill area in Euclid and Caterpillar end-dump trucks. After dumping, the rock was spread in 3-foot layers with a Caterpillar D-8 dozer. Compaction of this zone was accomplished by hauling equipment travel evenly distributed over the fill area.

4. Construction Control—No control tests were made on zone 5 fill.

Verification of Design

The design of a dam is based upon a number of assumed conditions during analysis of the embankment and its foundation. Verification of these assumptions during construction is of extreme importance. The designers of Teton Dam used the following reports and procedures to monitor construction and verify design assumptions.

a. Reviewed weekly progress reports prepared by the Project Construction Engineer.

b. Reviewed the applicable portions of monthly progress reports (L-29) prepared by the Project Construction Engineer. This report described the work progress and contained current and cumulative construction control test results on moisture and density control. The report also contained some photographs.[1]

c. Telephone conversations with construction personnel.

d. Visits to the construction site by the designated Construction Liaison representative. The visits were at the following times and purpose:[2]

Date	Purpose
June 12-23, 1972	General Construction Inspection
August 23-24, 1972	Inspection of major rock joints (left abutment)
November 14-17, 1972	General Construction Inspection
June 18-20, 1973	Foundation Grouting Inspection
October 8-11, 1973	General Construction Inspection
March 20-21, 1974	Inspection of right abutment key trench.

e. Visits to the construction site by designers. The visits were at the following times and purpose:[2]

Date	Purpose
October 16-17, 1973	Inspection of Foundation
March 20-21, 1974	Inspection of right abutment key trench.

CONCLUSIONS

1. Past experiences at other dam sites may have given the USBR designers an unwarranted sense of confidence to the extent that:

a. Their design testing of foundation and construction materials for gradation, permeability, and shear strength was insufficient to define the range of material properties that could be reasonably expected to be encountered during construction.

b. They thought that installation of instrumentation to monitor foundation and embankment settlements, lateral movements, and piezometric pressures to be unnecessary.

2. The designers were overly optimistic when they assumed that compacted low plasticity zone 1 silt would remain intact; therefore, they failed to provide adequate defensive measures to protect against the materials high potential for cracking and erosion.

Special provisions—such as concrete sealing of the rock openings and/or placement of filter material between zone 1 and the downstream rock—should have been provided to prevent the migration of zone 1 material into rock.

3. The design choices and subsequent construction in the badly jointed abutment area are believed to be the significant contribution to failure. The adverse physical aspects of foundation preparation were: (a) the rock geometry in the abutments that resulted in steep rock faces, deep tranches, and extremely high seepage gradients across the impervious barrier, and (b) rock surface treatment that permitted vertical faces, small overhangs, and cracks to be in contact with the erodible zone 1 material. The severe abutment geometry was conducive to the development of stress conditions that could encourage cracking due to differential settlement and hydraulic fracturing of zone 1.

This group may have selected design alternatives different than key trenches; however, if key trenches were specified, the following should have been provided: (a) abutment key trenches with minimum bottom (floor) widths of 0.25 of the maximum reservoir head that could act at that zone 1 rock contact elevation, (b) sealing of all openings in rock, with concrete or shotcrete, in the floor of the key trench, in the upstream wall of the key trench, and beneath zone 1 upstream of the key trench, (c) a continuous blanket of filter and drain material between zone 1 and the rock on the downstream wall of the key trench, beneath zone 1 downstream of the key trench, and that connected to the specified zone 2 blanket drain.

4. The designers did not adequately recognize: (a) the severity of the jointed rock foundation conditions upon the dam's performance and (b) the need for more extensive rock surface treatment. For the rock

[1] Some of the photographs are in Appendix D-2 of Appendix D.
[2] Trip reports are contained in Appendix G of the IRG report.

conditions at Teton, the specially compacted earthfill was inadequate to prevent migration of zone 1 fill into the joints. Designs to protect against migration of erodible material need to provide several complimentary treatments.

Contract work to provide extensive foundation cleanup and treatment should not be performed as subsidiary obligation to excavation or embankment placement.

5. Although the DESIGN CONSIDERATIONS FOR TETON DAM did not in all cases relay adequate information to the project construction staff, (e.g., surface sealing of rock joints) the designers are to be commended on their intent to supply to construction personnel information beyond that contained in the plans and specifications.

6. The gravity grouting of openings in the rock surface was inadequate for sealing the rock in that:

a. Construction personnel executed gravity grouting solely to create a firm surface against which zone 1 fill could be compacted or to fill voids that existed behind previously compacted zone 1 fill when rate of fill placement exceeded gravity grout placement.

b. Large openings that could be backfilled with silt using hand-operated power tampers and openings less than approximately one-half inch were normally not filled or sealed by gravity grouting.

c. Gravity grouting on the right abutment was terminated above elevation 5205.

7. The foundation preparation for the embankment was performed in accordance with the significant aspects of the contract plans and specifications.

8. The dam embankment was constructed in accordance with all applicable significant aspects of the contract plans and specifications. The fill placement methods used to construct the dam embankment were in accordance with the general practices and procedures followed in dam construction.

9. USBR personnel inspecting the earthwork construction were qualified to perform their responsibility. The embankment construction organization for inspection was excellent.

10. The project laboratory was properly equipped and staffed with qualified personnel to adequately perform embankment construction control testing.

11. Construction control testing was adequate and followed generally accepted procedures. During construction, the designers should have required some laboratory tests on undisturbed block samples of compacted zone 1 fill to confirm design shear strength parameters so that adjustments could have been made to the dam embankment if necessary.

12. The monitoring of construction by designers was less than adequate. Visits by designers to the site during construction were too infrequent for them to be knowledgeable of the construction conditions. Visual examination by designers is especially important in those areas, such as rock surface conditions, that cannot be adequately described in written reports. The effectiveness of the few site visits made by designers is questionable since they failed to adequately assess the foundation conditions and make suitable adjustments. In addition, the timeliness in which the designers received test results on materials used in the dam from the field is uncertain, e.g., permeability tests on zone 2 materials were still being made at the project laboratory nine months after the dam embankment was completed.

13. The ability of zone 2 to function as an effective drain is questionable. The majority of laboratory permeability tests conducted during construction indicate that the material would be unable to convey any seepage quantities beyond routine percolation through intact zone 1 fill.

14. There is no evidence that the failure was related to inadequate structural shear strength of the materials composing the dam and its foundation.

APPENDIX D-1

EARTHWORK CONSTRUCTION INSPECTION

1. Teton Dam Project Organization of Earthwork Construction Inspection

2. Resumés of Project Staff

Teton Dam Project Organization
of
Earthwork Construction Inspection

Position			
Field Engineer	D. Fillis, 1972-74 B. S. Beckley, 1974-76 P. P. Aberle, 1976		
Inspection Branch Chief	P. P. Aberle, 1974-76 B. S. Beckley, 1972-74		Materials Engineer Branch (Project Construction Control Testing Laboratory) *R. J. Mulliner, 1972-76
Principal Inspector (Shift Supervisor)	*L. G. Rogers, 1972-76	*J. R. Ringel, 1972-76	
Supervisory Embankment Construction Inspector	*K. C. Hoyt, 1975-76 *R. D. McClung, 1972-74	*D. C. Jarvie, 1973-76	
Earth Placement Inspector	C. Albertson, 1975 S. D. Elenberger, 1975 *S. Johnson, 1975 R. A. Jones, 1975 D. Miller, 1975 W. Stapleton, 1975 F. A. Selle, 1975	J. L. Smith, 1974-75 A. Stocker, 1973-75 R. Branson, 1973-75 C. Entwisle, 1974 *Glen Harris, 1974 K. Justesen, 1974 M. Miller, 1974 R. Mitchel, 1974	F. Emrich, 1973 T. Faull, 1973 B. Holbert, 1972-73 P. Aschenbrenner, 1972 A. Heintz, 1972 D. Peterson, 1972 J. Tucker, 1972 R. Wallace, 1972

*Interviewed by Embankment Construction Task Group

RESUMÉS OF PROJECT STAFF
(Those submitted by USBR)

Robert R. Robison
Project Construction Engineer
Teton Basin Project

Mr. Robison is a graduate of the University of Utah with a BS degree in mining engineering, is a member of AIME, and has 25 years' experience with the Bureau of Reclamation. Most of this experience has been in either materials control or earth dam construction and approximately 20 years has been in responsible supervisory positions associated with the construction of USBR dams and appurtenant works. His assignments include Field Engineer on Willard Bay and Stampede Dams and Resident Engineer on Causey Dam prior to his present assignment at Teton. These dams are all earth dams.

Peter R. Aberle
Field Engineer
Teton Basin Project

Mr. Aberle holds a degree in geological engineering from South Dakota School of Mines and has 15 years' experience in construction supervision with the Bureau of Reclamation. Most of his experience has been in grouting and he was responsible for supervising the control of grouting by contract on Heron and Mountain Park Dams. In 1971, he was detailed to the Navajo Indian Irrigation Project in Farmington to organize and supervise the foundation grouting program on Cutter Dam. Except for Mountain Park Dam, these are all earth dams. His experience in earth dams also includes an assignment as Chief Inspector on the James River diversion dam and his grouting experience includes work at Glen Canyon and Morrow Point Dams.

EXPERIENCE RESUMÉ

Bruce S. Beckley

I graduated from the University of Wyoming with a Bachelor of Science Civil Engineering degree in June, 1961, in the top one-fourth of the class.

I joined the Bureau of Reclamation upon graduation and was assigned as a rotational engineer to the Weber Basin Project, Ogden, Utah. After a short rotational program (6 weeks, 6 divisions) I was assigned to the construction division (Willard Canal Field Office) as a construction inspector. I inspected earth and concrete construction, pile driving, and paint and mechanical installation for Willard Canal, Willard Pumping Plants No. 1 and No. 2 and Layton Pumping Plant. For 2 one month details I was assigned to Willard Dam as construction inspector and surveyor. In January, 1963, I was assigned to Causey Dam Field Office as Chief of Surveys, supervising all survey work. In January, 1964, I was assigned as a Supervisory Civil Engineer on construction inspection for the construction of Causey Dam.

In April, 1965, I transferred to the Boulder City Development Office, Boulder City, Nevada. My basic duties were gathering design data and field work.

In January, 1968, I was reassigned to the Southern Nevada Water Project as Chief of Surveys, supervising all surveys for 34 miles of pipeline, 8 pumping plants, and 2 tunnels. In August, 1971, I was transferred to the Southern California Planning Office, San Bernadino, California supervising field work and design data gathering.

In April, 1972, I was transferred to the Teton Project as Chief Inspector.

In September, 1974, I was promoted to Chief, Field Engineering Division, Teton Dam.

In March, 1976, I was transferred to the Narrows Project as Acting Construction Engineer and in June, 1976, was assigned as Field Engineer, Narrows Project.

/s/ Bruce S. Beckley

Lyman Rogers
Supv. Construction Inspector

1/46 - 9/49—Survey Aid for U.S. Bureau of Reclamation

9/49 - 3/55—Inspected earth and concrete structures and was on concrete inspection on refacing Black Canyon Dam spillway.

3/55 - 8/58—Inspected earthwork and concrete structures on canals and laterals on the Minidoka Project, duty station. Rupert, Idaho.

8/58 - 11/59—Principal earthwork inspector on earthfill dam, Little Wood River Dam.

11/59 - 6/62—Principal inspector on all phases of construction of laterals, substations, wells, drainage, and buildings and installation of deep well pumps and testing of pumps, Rupert, Idaho

6/62 - 8/63—Principal inspector on all phases of pipeline rehabilitation of Dalton Gardens, Hayden Lake, and Avondale (Coeur d'Alene, Idaho) Districts. In charge of laboratory and materials.

8/63 - 5/67—Principal inspector on Spokane Valley Project—approximately 100 miles of transit pipeline (6" to 36" diameter, 34 pumping wells, water tanks and appurtenant structures.

5/67 - 8/71—Principal inspector to resident engineer in preconstruction activities—inspected contract drilling of exploration wells for geologic and hydrologic data. Established and maintained a water level observation program with area ground water geologists. Inspected drilling and testing of test wells for Teton Dam Project. Snake River Development Office, Boise.

8/71 - 6/76—Principal inspector on a shift on all phases of construction by contract of the Teton Dam, Power and Pumping Plant appurtenant structures. Worked under general supervision of Chief, Inspection Branch.

Jan Ringel,
Supv. Civil Engineer

Graduate Civil Engineer, BES, 1970

7/67 - 4/71—Engineer Trainee Rotational Program for the Utah Dept. of Highways which included training in the following divisions: materials and testing, road-way design, right-of-way design, hydraulics, transportation and planning, structure, traffic, construction and location.

USBR

5/71 - 8/72—Civil Engineer in Regional Office, USBR, Great Falls, Montana. Worked on reports and designed hydraulic structures, canals and drainage systems in the design branch. Wrote a set of specifications on a small drainage project. Worked on Canyon Ferry Dikes which includes setting alignments of dikes, designing dikes, and calculating quantities and cost estimates. Also worked on many smaller routine projects which required engineering skills.

9/62 - 4/75—Inspected excavation of the dam foundation and structure foundation excavation, excavation of tunnel portals; shift inspector on the driving and excavation of the 6-foot diameter tunnel and a 26-foot diameter shaft 300-feet deep. Also has been assigned to the earthwork operation in charge of different zones of earthwork on one shift. Worked with zones 1, 2, 3, 4, & 5 materials in excavation and placed zones 1, 2, 3, 4, & 5 in the dam embankment. Controlled material to obtain material suited for various zones. Worked in close conjunction with soils laboratory to obtain a number of density tests required in relation to yardage placed in the different zones and through test results varied the methods of placement to fit material conditions as to suitability on embankment.

Later assigned to the pumping plant, powerhouse, and canal inlet and outlet structures setting forms, miscellaneous metal work, piping, and the installation of reinforcing steel. Worked closely with surveys in preparing tack sheets for each placement in these areas and issued these tack sheets to the contractor. Discussed with supervisor the progress and problems as well as relations with the contractor. Also inspected concrete placements in the past and worked closely with the laboratory for concrete cylinders, air entrainment, and slump.

During the early winter months of 1974, supervised and participated in the planning and formulation of a comprehensive zoning report to determine the proper distribution of materials to be placed into Teton Dam. Responsible for coordinating several subordinates to complete this report on the given deadline. After completing this report, reassigned to set up and organize a field office in the power and pumping plant, and to confer with the contractor's representatives on the power and pumping plant design.

When the construction started again, assigned to the spillway as a shift inspector on the drainage trenches excavation; checked out concrete placement for the floor and walls to insure that the contractor had included all embedded materials in the proper location; conferred with the contractor's representatives on the interpretation of specifications or standards to the attention of the contractor and saw that these conditions were corrected; always advised supervisors of daily construction progress, daily problems, and contractor's relations; and attended weekly contractor safety meetings and worked with the contractor to keep a safe working condition in the work area.

During the winter months of 1975, again supervised the planning and formulation of a comprehensive zoning report to determine the materials distribution to be placed in the Teton Dam.

Richard McClung

4/25 - 12/55—Surveys, U.S.B.R.

1/58 - 5/58—Worked on canals at U.S.B.R., McCook, Nebraska. Duties consisted of field density tests, penetration resistance and moisture test of compacted embankment and structures backfill to determine the percent of compaction. Made Atterburg limits tests on soils samples and operated compression testing machine for testing the strength of concrete control cylinders.

5/58 - 4/60—Worked on an earth filled dam and pipeline at U.S.B.R., Anadarko, Oklahoma. Duties consisted of soil classification tests, mechanical analysis, specific gravity, permeability, compaction, and penetration of soil. Concrete work consisted of slump tests and air entrainment tests.

4/60 - 3/61—Construction Inspector at Weslow, Texas, rebuilding old canals. Checked contractors work on removing mud and fill of the old canals down to suitable subgrade. Compacted embankment checking the material used for moisture content, compaction, line, and grade. In placing concrete lining, checked slump, finish, curing, line, and grade.

3/61 - 2/62—Construction Inspector at St. Paul, Nebraska, working on structures. Checked subgrade, placing of concrete in structures, reinforcement steel for spacing, grade and quantity, stripping of forms, and finish of concrete and cure. Compacted backfill around structures, checking for moisture content, degree of compaction, line and grade. Inspected laying of precast concrete pipe for drainage and structures, checking line, grade, and compacted backfill. Worked in tunnel portion of the time.

2/62 - 4/63—Inspected work on a 230-kV transmission line at Lewiston, California. Consisted of placing concrete footings for reinforcement steel and compacting backfill.

4/63 - 5/65—Principal concrete inspector on Yellowtail Dam and Powerplant at Forth Smith, Montana. Inspected embedded items, contractor's concrete operations, concrete placing in spillway and spillway tunnel lining.

5/64 - 8/66—Inspector at U.S.B.R. Project at Yuma, Arizona, inspecting the contractor's operations or constructing pump discharge canal and pipelines.

8/66 - 4/68—Inspector on the Oraville-Tonasket Unit of the Chief Joseph Dam Project, Oraville, Washington. Work consisted of construction of reinforced concrete pipe cross-strings, concrete lined canals, elevated and bench flumes, and county road relocations.

4/68 - 4/70—Supervisory Inspector over the construction of the Sinlehiken Siphon and Toats Coulee Diversion Dam which are both a portion of the White Stone Coulee Unit of the Chief Joseph Dam Project. The Sinlehiken Siphon is a 6-mile pipeline ranging in size from 18-inches to 45-inches in diameter. Toats Coulee Diversion Dam is a reinforced concrete structure.

4/70 - 10/71—Supervisory Inspector on the pipeline laterals of the Southern Nevada Water Project. Principal responsibility was to supervise the contractor's operations in constructing the pipelines.

10/71 - Present—Shift Inspector at Teton Dam. Supervised inspection of the contractor's operations on excavation, foundation preparation and placement of embankment materials, and checking for compaction, lift thickness, and moisture. Inspected the contractor's operations in constructing the power and pumping plant and appurtenant structures for excavation, foundation preparation, forms, re-steel placing, and concrete placing.

Kenneth Hoyt

11/64 - 5/68—Surveys for U.S.B.R.

5/68 - 9/70—(Missouri Oahe Project, O'Niell, Nebraska) Construction Inspector on steel tower assembly and erection, and alinement and location of structures, including bolting, filling, and bracing of towers. Inspected forms, reinforcing steel, placement of concrete, clearing of right-of-way, and construction of gates.

9/70 - 3/72—(Missouri-Souris Project, Garrison, North Dakota) Performed inspection duties on installation of machinery, electrical equipment, metal work, and piping as required for installation and embedding in concrete.

3/72 - 7/72—(Missouri-Souris Project, Garrison, North Dakota) Performed inspection duties on earthwork including excavation, embankment, backfill, and compaction, earthlining, concrete construction, erection of forms, installation of reinforcing steel, and concrete placing.

7/72 - 3/75—(Navajo Indian Irrigation Project, Farmington, New Mexico) Inspected installation of precast concrete pipe for canal siphon and concrete pipe for canals, drains, siphons, and structures. Determined suitability of material from excavation for use in embankment, backfill, compacted embankment, and consolidated backfill.

3/75 - Present—(Teton Project, Idaho) Supervised three to five inspectors in the placing of zones 1, 2, 3, 4, and 5 material and riprap in main dam fill. See that material is suitable for use in respective zones.

Douglas Jarvie

10/56 - 10/59—Surveyor, U.S.B.R.

10/59 - 9/66—Supervisory Surveying Technician, U.S.B.R.

9/66 - 6/69—Construction Inspector at Meeks Cabin Dam, Mountain View, Wyoming. Inspected earth materials at both the borrow area and construction site. Inspected for suitability of earth materials of the foundations, dam embankments, road dykes. Checked for outlet works, spillways, and dykes. Checked embedded materials, placing procedures, curing and forming, and grouting operations.

6/69 - 4/71—Supervisory Construction Inspector (Same as above) Assigned to supervisory and inspecting the construction of the Lyman Project, which included Meeks Cabin Dam (earth filled), reservoir, and access road and other appurtenant structures.

4/71 - 6/72—(Upper Green River Project Office) Served as principal inspector of the contractor's operation on the Farson, Eden Irrigation Project. This consisted of reconstruction or modification of existing canals and laterals; construction of new canals, laterals, turnouts, measuring devices, and farm bridges; the remodeling and updating of the little Sandy Dam outlet work.

6/72 - 6/73—Principal inspector on a shift of the contractor's operations on Meeks Cabin Dam (earth filled). This included inspection on the final work on the dam, reservoir, access roads, and appurtenant structures.

6/73 - Present—Inspection shift of Teton Dam. Supervised and inspected the contractor's operations on excavation of keyway foundation, dewatering and foundation preparation, placement of embankment material, and checking for compaction, lift thickness and moisture. Inspected the contractor's operations in constructing the power and pumping plant and appurtenant structures for excavation, foundation preparation, and forms, re-steel placing, and concrete placing.

Glen M. Harris
Civil Engineering Technician

5/11/75 - Present—Responsible for administering construction contracts; coordinating construction operations; preparing construction cost estimates; collection of design data; and all related activities. Assigned specific duties which may include all functions in developing such progress payments to contractors, and is responsible for correct interpretation of the contract specifications in doing so. Based on knowledge of contract specifications, computes the computation of quantities including excavation and backfill for pipelines and pumping plants, concrete in structures, weights of metal parts, and weights of reinforcing steel for concrete structures. Assists in the assembling of data for calculation of earthwork quantities in borrow areas using aerial photogrammetric methods. Prepares various drawing or sketches needed to substantiate computed pay quantities.

Reviews contractor's drawings for bending and planning steel reinforcing bars and for forming and placing concrete in structures to assure conformance with specification requirements. These drawings and data are also used for monthly progress estimates and final payments. Collecting survey information in order to compute stake-out information for survey crews working on various boat ramps, recreation areas, access roads, and pipelines. Also checking computations before stake-out is performed.

5/10/75 - 7/2/73—Construction Inspector. Performs inspection duties on contractor's drilling and grouting operation which include: low pressure blanket holes, high pressure curtain holes, tunnel and shaft radial holes, and tunnel backfill grouting.

Carries out a defined grouting schedule in determining direction, depth and spacing of grout holes, grouting mixtures and pumping pressures. Keeps daily records of drilling and grouting operations, refers to supervisor conditions different from those indicated by previous instructions, and advises supervisor of foundation conditions encountered during drilling and grouting operations.

Working with contract administration during winter months. This included plotting cross sections, running end areas and compute columes for payment of access roads, service roads and earthfill dam.

Performs inspection duties and prepares daily inspection reports of earthfill dam. Daily progress and inspection of zones 1, 2, 3, 4, 5, and other related features of Teton Dam.

6/30/71 - 5/10/73—Supervisory Survey Technician. Horizontal and vertical surveys for preconstruction work. Responsible for keeping daily recordings of water gaging stations, water storage and release from reservoir to meet state and local water users allotment. Responsible for operations of high pressure gate in gate chambers and control gates in control house.

Due to the nature of Meeks Cabin Dam the various points were checked twice weekly. Alignment and settlement points on dam and stilling basin walls. Slope Indicator drill holes, piezometer wells, slip joints on 62 inch steel liner were checked three times weekly.

6/30/68 - 6/30/71—Working from general instructions. Bureau Survey Manuals, Survey Trade Publications, and Construction specifications and drawings. Organizes and assigns work to survey party to accomplish preconstruction and construction field surveys on a day to day basis as follows: Horizontal and vertical surveys, cross section surveys, final location surveys, and personally supervised work in progress to insure accuracy. Assembled and reviewed field noted and sketches, and supervised the preparation of maps, charts, graphs, etc.

Trained surveymen in general survey activities.

Detail Duties: Detail to Upper Green River Project Office, Rocks Springs, Wyoming. To perform construction surveys in connection with canal enlargements and rehabilitation on the Eden Project. 8-24-70 - 11-21-70

Detail to the Engineering Division to assist in Office Engineering functions. Reducing and checking field notes, plotting cross sections of dam area, borrow areas, and access roads, computing end areas and earthwork volumes from cross sections, miscellaneous computation and drafting as required. 1-3-69 to 4-18-69.

Detail to Office Engineering Division, same as above. 3-3-68 to 5-4-68

Detail to Stampede Dam, for the Lahontan Basin Project, Carson City, Nevada. To perform survey work on location of access road to Stampede Dam. Horizontal control, vertical control, original cross sections and controls for diversion tunnel. 11-7-66 to 12-17-66

6-6-66 - 6-30-68—Working from specific instructions, Bureau survey manuals, Survey Trade Publications, and construction specifications and drawings, incumbent performs the following day to day duties. Served as party chief to take and record cross sections, establish horizontal and vertical controls on borrow areas, river outlets, spillway, reservoir clearing lines, dam axis and other related features. Records and notes taken and used from cross sections, horizontal control, vertical control. Construction progress and monthly estimates used for payment.

Conducts and submits minutes of planned tool box safety meetings as required.

3-14-64 - 6-6-66—Accomplishes a varied program of surveys. Serves as journeyman instrumentman operating survey instruments.

Worked as Chief of a sub-party which is usually made of an instrumentman and one or two helpers. Served as party chief from 6-2-65 to 11-2-65 with 3 to 5 men.

3-18-62 - 3-14-64—Received general assignments in the execution of final location and construction surveys on the Flaming Gorge Dam. Powerplant, and appurtenant features, requiring the operation of all survey instruments.

Served as journeyman instrumentman operating survey instruments, including precision types such as the Wild T-2 theodilite and precise level, on construction surveys performing the following work:

Establising precise controls for the alignment of the dam, powerplant, and spillway, layout of penstocks, gate frames and guides, turbines, scrollcases, and other mechanical and electrical equipment.

Line and grade for excavation of structures. Checks and adjusts instruments for accuracy. Performs related duties as assigned.

12-11-60 - 3-18-62—Served as journeyman levelman, instrument man for final location and construction surveys for the Flaming Gorge Dam, power plant, diversion tunnel, spillway, and access roads.

6-14-59 - 12-11-60—Perform responsible duties of a transitman on pre-construction and preliminary surveys and duties as a levelman on construction surveys. Perform duties of rodman on precise controls necessary to establish and maintain the main control systems involving the carrying of controls through coordinates, triangulation, etc. into the working areas of the dam and related structures.

2-10-59 - 6-14-59—Engineering Aid. Holds rod for levelman when precise lines, grades and elevations of structures are being established for the setting of concrete forms for foundations of structures. The laying of water and sewer lines, the location and grading of roads, and construction features of diversion works. Sets slope stakes, cross sections, sketches and records target readings.

4-20-58 - 2-10-59—Engineering Aid. Take samples of concrete aggregate from field test areas being explored for potential sources of aggregate as used in construction. Take regular and special samples of earth materials, operates simple testing equipment such as rotap, screens compactors, and related equipment, make laboratory tests such as standard mechanical analysis, moisture and specific gravity tests. Field density, compaction, and penetration analysis. Make simple computations, compile records data resulting from tests. Perform related duties as assigned.

12-3-56 - 4-20-58—Laborer. Running gravel gradations and screening rotap samples for aggregate investigations. Screening analysis for concrete aggregate. In charge of weighing and computation for aggregate investigations for Flaming Gorge Dam and Diversion tunnel. Various laboratory experience. Slump test, making cylinders, etc.

10-19-56 - 12-3-56—Unemployed

3-56 - 10-18-56—Cement Finisher Apprentice. Construction of compressor station which consisted of cement finishing of colored cement floors, finishing sidewalls and foundations. Finishing and pre-pack on piers and foundations for high pressure tanks and large stationary motors. Plastering and stuccoing basement walls. Placing foundations for 6 permanent homes.

Work was performed with vibrators, hand finishing trowels, markers and edgers. Operation of large mixer and batching plants, preparing and caring of newly finished cement floors and driveways.

6-53 - 3-56—Second-class Carpenter. Building and repairing permanent type houses and garages. Laying out and constructing foundations for water tanks, houses, garages. Construction of built-in cabinets, closets, dressers, redecorating homes, plastering of walls, wall papering, laying of various types of linoleum. Operating power equipment such as cross-cut saw, rip saw, planers, jointers, shapers, band saw, datto blades, and sanders. Repairs on Railroad bridges, snow fences and stock yards.

3-17-53 - 6-9-53—Private—U.S. Army. Doing various detail jobs, entered service as a "Medical Hold" and stayed such until received an Honorable Medical Discharge.

10-52 - 10-17-53—Second-class Carpenter. Same experience as in 6-53 - 3-56.

5-50 - 10-52—Ranch hand. Operating farm machinery, tractors, mowers, rakes, push rakes, seeding drills, planting and harvesting of crops such as barley, oats, and alfalfa, preparing and caring of ground for crops. Handling and taking care of horses, cattle and sheep. Construction and repairs of fences. Operating trucks for marketing of crops and livestock.

Jerry L. Smith

12-1-75 - Present—Compute simple end areas and volumes. Reduce survey notes. Plotting "X" section of construction excavation, dam embankment and roads. Other office work as assigned.

6-1-73 - 12-1-75—Construction inspector for zone #1 dam embankment fill. Contractor time and equipment and inspection daily reports. Location of compaction tests. Insepction of contractors equipment.

1-27-69 - 5-31-73—Supervisory survey technician. 20% control, organizes, plans and applies best survey techniques and procedures in the field in order to establish horizontal and vertical control for the accurate layout of the various facilities. 40% field layout. 15% survey notes. 10% miscellaneous jobs and 15% estimates. Is responsible for taking cross-sections and other field data for monthly estimates.

9-8-68 - 1-27-69—Surveying Technician. Operates: Wild T-2 theodolite on triangulation control and operates precise leveling instrument. Make field notes of observations taken from triangulation points. I am responsible for instructing and informing subordinate personnel in arc operation and safety requirements. Have acquired knowledge in the use of life lines and safety belts.

3-24-68 - 9-8-68—Surveying Technician. As instrumentman established triangulation points, bench marks, reference points, and other necessary control points for the excavation and construction of multi-arch dam. Also operate all instruments for obtaining "X" section, establishing level circuits, producing lines for construction and taking topog.

5-7-67 - 3-24-68—Surveying Technician Instrumentman. Helped determine fills zones in dam, around intake structure, stilling basin and spillway. At the pumping and relift plants, pipe drop and siphon staked the layout, checked forms, station, grade and distances, slope stake approximately five miles of canal.

7-17-65 - 5-7-67—Surveying Technician Instrumentman; establish clearing lines around Phillips Reservoir. "X" section spillway, dam site, borrow area and roads. Determined cut lines; dam site, grout trench, spillway, stilling basin and Black Mountain road. Spillway, stilling basin and outlet works. Established and checked joint lines, grades and stations.

10-11-64 - 2-27-65—Surveying Technician. "X" section construction sites, canals, wasteway, dikes along Sun River for riprap. On Tiber Dam downstream from stilling basin for riprap along banks and bridge at Gibson Dam. Location of 1000' siphon and siphons.

10-11-64 - 2-27-65—Surveying Technician. As transit man, helped locate and make section ties to transmission lines, turn and check accuracy angles, establish structure sites, "X" section structure legs, stake steel tower, concrete footing for excavation, check concrete footing for vertical and horizontal errors, inspect wood pole transmission lines, excavation of anchors and hardware on the poles.

10-22-63 - 10-11-64—Surveying Technician. As instrument man, directed head chainman to center line, turn and check angles for accuracy on Oahe to Eagle Butte transmission line and Mission, S.D. to Martin, South Dakota. Profile: transmission lines from Sioux Falls, S.D. to Ft. Thompson.

5-14-61 - 10-22-63—Engineering aid. Aided in locating transmission lines, topog. on possible substation sites, head chainman from Hinton Substation to Spencer, Iowa and from Denison, Iowa to Creston, Iowa.

9-60 - 5-14-61—Surveying Aid. Rodman doing "X" sections on creeks; mapping section to be used for possible irrigation study. Second and third order levels.

2-25-57 - 9-60—Survey aid. Rodman doing topog. 200' to the inch, on several stretches of creek bottoms. As chainman I chained out blocks, established control points approximately every half mile to be used later for mapping.

2-4-55 - 2-1-57—Fireman

5-54 - 12-54—Station attendant

8-53 - 2-54—Rodman and rear chainman, mostly cross section roads. Helped run levels.

Stephen D. Elenberger

6-9-74 - Present—Construction Inspector. Drilling, shooting, excavation, re-bar installation and concrete forming and placement of the spillway stilling basin. Also included in this is a sub-terrainian water pressure relief system. Inspected some of the main fill and key trench excavation and clean-up for the Dam. When that was complete, inspected the Dam fill placement and compaction which consisted of five different types of fill material which were put in by differing methods and lifts. Mechanical Installation and painting of stoplogs, radial bulkhead, radial gates, river outlet gates, outlet works, shafts metal work, etc. Also inspected coal-tar enamel application to a steel-lined tunnel and all painting in power house, pumping plant, and warehouse.

Performed inspection on the following: Power house super structure, roofing plumbing & piping, sewage system, emergency water and fire system, Penstock piezometer systems, crane installations, discharge pipeline, duct bank and canal outlet portion. After the failure of Teton Dam assisted in some investigation research and inspection of emergency canal reconstruction. Also 3 emergency pumping systems were built and put into service, and had to serve as water master making sure pumps were kept running to suit irrigation demands.

3-72 - 6-9-74—Surveying Technician. Preliminary surveying of 5 different borrow areas, the various structures, access roads, recreation areas, and Dam proper. Served as Instrumentman and helped organize work in concurrence with specifications and contractors needs for survey party chief. Used several different surveying transites, levels, distance meters, compasses, etc.

6-68 - 3-72—Member of survey crew, progressed to Instrumentman. Preliminary and construction surveying were done on the wide base canal with its many bridges, siphons, box culverts, freeway relocation, public and private road relocations, railroad detour and relocation, and a canal inlet headworks structure. Work with the survey crew was running "P" lines for access roads and determining cut and fill balances for location for construction.

5-67 - 6-68—Surveying Aid. Assigned to a survey party which was involved in construction, reconstruction and maintenance of Department of Transportation Roads and Mountain passes. Primary duties were that of any other rodman-chainman on a survey crew bit was also assigned as inspector at times. Inspector of paving operation on the Kingsbury Grade between Lake Tahoe and Carson Valley, Nevada. Ran Sieve analysis to make sure the bituminus Plant Mix was within the limits of mix design and at times served as weigh-master.

Frank Emrich

9/55 - 3/60—Surveying, U.S.B.R.

3/60 - 8/61—(Willard and Ogden, Utah) Performed standard field and lab tests on construction materials as required, which included test for investigation and control for earth and concrete materials.

8/61 - 6/63—(Willard, Utah) Construction Inspector. Inspected construction work performed by contractor's forces on Willard Canal, Willard pump plant, and related works (1) earthwork—canal embankments and backfills (2) concrete construction of pumping plants and C.H.O.'s (3) driving of piling below both pumping plants (4) laying of pipe—discharge and C.H.O.'s.

6/63 - 10/63—(Yuma, Arizona) Construction Inspector. Inspected the drilling of 33 drainage wells located in Wellton Mohawk and South Gila areas.

10/63 - 3/64—(Yuma, Arizona) Construction Inspector. Inspected 8,000 acres of tile drains in Wellton Mohawk area and 6 miles of tile drains in Yuma area.

3/64 - 6/64—(Yuma, Arizona) Construction Inspector. Inspected and acted as part-time geologist on drilling of three deep supply or production wells in South Gila Valley.

6/64 - 4/65—(Yuma, Arizona) Construction Inspector. Inspected group cap curtain trench excavation, installation of grout pipe in trench, placing grout cap concrete and all steps in drilling and grouting of grout holes. Inspected hauling and placing of materials used for reservoir blanket, hauling, weighing, and placing of riprap material, checking slopes while excavating rock for service roads. Inspected structure subgrade, placing of structural steel anchor bars while being grouted into formation, placing of large and small concrete placements and structural welding on penstock installation.

4/65 - 8/65—(Yuma, Arizona) Construction Inspector. Inspected quarrying of rock from Pilot Knob rock quarry.

8/65 - 11/65—(Yuma, Arizona) Construction Inspector. Inspected installation and erection of 2563.95 foot of 12-foot diameter flume.

11/65 - 5/69—(Corning, California) Construction Inspector. Inspected construction of Tehama-Colusa Canal, which included inspection of excavation and construction of prism and embankments, concrete placements, and laying of irrigation pipelines.

5/69 - 1/70—(Carson City, Nevada) Civil Engineering Technician. Performed required tests on earth materials, concrete, and aggregates. Performed mechanical inspection duties on swing shift.

1/70 - 5/70—(Carson City, Nevada) Surveying, U.S.B.R. Detailed to Soils Laboratory (performed lab tests).

5/70 - 6/72—(McClusky, North Dakota) Construction Inspector. Inspected construction of McClusky Canal, Reach 3A. Excavated and constructed canal prism and embankments; laid CMP road crossing drains. Inspected fabrication of trashracks and inspected and kept records on drilling test wells.

6/72 - 3/75—(Teton Dam, Idaho) Construction Inspector. Inspected construction of all office facilities.

Worked on grouting dam's grout curtain, lining outlet works tunnel, and gate chamber. Worked as checkout and placing inspector as concrete was placed in constructing powerhouse and pumping plant. Occasionally did mechanical inspection. Classified zone 1 material. Inspected contractor's zone 2 excavation and stockpiling operation. Inspected rock excavation of dam's cutoff trenches, spillway, and riprap rock quarry.

Inspected installation of mechanical equipment in power and pumping plant.

3/75 - Present—(Teton Dam, Idaho) Mechanical Engineer. Inspected contractor's mechanical installation operations. Inspected materials and supplies delivered to jobsite. Tested mechanical equipment.

Roger P. Michel
Civil Engineer

6/73 - 9/73
6/72 - 9/72
6/71 - 9/71—Summer help at the Dakota Utilities Co., Montana. Main responsibility was reading gas meters, however, also worked a small amount of gas pipeline construction.

6/74 - 2/76—Civil engineer at Teton Dam Project. Inspection activities include rock excavation, grouting, drilling, concrete, earthwork, and surveying.

2/76 - Present—Civil Engineer at Teton Project. Analyze contractors' claims for work changed from the specifications. This involves reviewing of the specifications, drawings, and records of labor, equipment and materials. Such information is used to establish authenticity of data provided by the contractor in his change orders. Either a denial of such claims or a monetary figure of adjustment is derived from the analysis. Review subcontracts for compliance to the specifications of the prime contract. Compose and send letters to the prime contractor indicating whether or not all requirements have been met and what additions are necessary for approval.

David Miller
Construction Inspector

3/68 - 5/69—Engineering Aid II/Inspector for the State Highway Commission of Indiana. Worked in bridge construction, inspecting embankment placement and testing of earthwork on bridge approaches and off ramps of the Interstate Highway system. On bridge construction, acted as rodman and chainman, checking layout and grade. Checked reinforcing steel according to drawings and specifications, set and checked screed elevations. Performed soil compaction and moisture tests, concrete Air Entrainment, Yield and Slump test, Flexural strength tests. Attended the Indiana State Highway Concrete and Earth School at Purdue University in West Lafayette, Indiana. Kept records and made weekly and monthly reports to send to the regional office.

5/69 - 10/69—Self-employed selling and installing chain link fences, privacy fences and split rail fences. Sub-contracted fence installation for the Central Hardware Chain of St. Louis, Mo. and for the Sears Corporation.

10/69 - 4/70—Kept records of parts and parts shortages for the Peerless Pump Division of the FMC Corporation. Scheduled production and delivery of parts to assembly and shipping. Coordinated between management and union labor representatives.

5/70 - 6/70—Clerk—Elliott Feed and Farm Supply.

6/70 - 5/71—Assigned to Yuma Projects Office. Established location lines, elevation controls, cross sections of proposed and built structures, topographic maps and cadastral surveys that pertained to construction of irrigation and drainage systems and appurtenant structures. Acted as a rodman and head and rear chainman. Carried marked and set stakes. Recorded and reduced field survey notes.

5/71 - 3/72—Rodman and chainman for Mountain Park Dam Project. Instrumentman on highway relocation and railroad relocation, established reference points for construction of complex concrete structures. Promoted to position of Surveying Technician GS-4.

3/72 - 3/73—Assigned to the Teton Dam Project as head and rear chainman, rodman and sometimes instrumentman. In May of 1972, assigned as full time instrumentman operating electronic measuring devices, transits, levels precise level, theodolite and alidade. Performed duties as a surveying technician.

3/73 - 5/74—Assigned to the Lab as a materials engineering technician. Took samples of earth materials and performed tests of earth materials. Tested concrete as it was being manufactured. Performed tests for slump water-cement ratio, etc.

6/74 - Present—Assigned to the Inspection Branch of the Teton Dam. Inspection of contractors drilling and grouting operations. In June 1974 assigned duties of inspecting the placement and compaction of the Dam Embankment consisting of Zone 1, Zone 2, Zone 3, and Zone 5 materials and continued in this until November 1974 when again assigned duties of grouting inspector.

Stephen C. Johnson

9/69 - 12/74—Highway Engineer I & II for Washington State Highway Department. Analyzed roadway design to determine operational level of service and required improvement; designed signals, highway signing, and illumination; surveying (with two months pipe inspection).

5/75 - —Teton Project, Idaho. Inspected placement of dam embankment zones 1 through 5, and riprap & installation of mechanical equipment in the powerhouse; aided in writing final construction report for Teton Dam.

Richard Jones
Civil Engineer

I. EDUCATION
  A. College
    1. B.S. Civil Engineering from California State University @ Fresno, California.

  B. 1. Planning and estimating school
    2. Builder school—Building techniques
    3. Instructor's school

II. EXPERIENCE
  A. U.S. Navy (SEABEES 4 years)
    1. Planning and estimating
      a. Manual Network Analysis
        1) Originate and complete C.P.M.'s
      b. Estimate materials
      c. Manpower estimates and updates
    2. Quality Assurance
      a. Assisted in drafting Q.A. Program
      b. Jobsite Inspector
        1) Road construction; including bituminous surfacing
        2) Conc. and conc. masonry building construction
        3) Soils and conc. lab
        4) Built-up roofing
        5) Painting (including texcote)
        6) Conc. pre-cast yard
        7) Re-bar pre-fab yard
        8) Conc. masonry unit fabrication plant
        9) Mechanical in buildings
        10) Electrical in buildings
    3. Minor Design Work

  B. Bureau of Reclamation
    1. Teton Dam, Newdale, Idaho
      a. Jobsite Inspector
        1) Left and right abutment fill
        2) Left and right key trench fill
        3) Placement of rip-rap
        4) Placement zones 1, 2, 3, and 5 fill
        5) Conc. repair ROW intake structure
        6) Painting ROW tunnel
      b. Snow Removal
    2. Riverside Park, Palisades, Idaho
      a. Jobsite Inspector (90% of Project)
        1) Road construction; including gravel surfacing
        2) Conc. and conc. block buildings
        3) Mechanical—chrysler crapper units, sprinkler system, and water supply (culinary)
        4) Electrical—service and bldg. elec.
        5) Responsible for field changes, interpreting bureau specs. etc.

Alvin Heintz

8/55 - 5/60—Surveys for various projects, U.S.B.R.

5/60 - 12/60—Served as concrete batch plant inspector, Prosser Creek Construction Field Office

12/60 - 5/61—Surveys, U.S.B.R.

5/61 - 12/61—Performed inspector's duties on Prosser Creek Dam in borrow pits observing excavation operations at two separation plants located in zone 1 and zone 2 areas. Worked in coordination with the different zone inspectors in regard to moisture, mixture, and compaction of the different materials in the fill of the embankment area.

12/61 - 5/62—Surveys, U.S.B.R.

5/62 - 11/62—Performed inspection duties on Prosser Creek Dam on the zone 1 and zone 2 embankments coordinating with the Chief Inspector in regard to suitable material to be placed and compacted to specifications. Inspected upstream and downstream zone 2 embankment, boulder blankets, and the upstream riprap blanket; excavation on both abutments; placing of reinforcing steel, embedded materials, formwork, and cleanup of stilling basin, control building shaft, installation of drain pipes, fitters and construction of open drains; Government and contractor furnished materials to assure compliance with specification requirements.

11/62 - 9/63—Surveys, U.S.B.R.

9/63 - 9/67—As Construction Inspector, assigned to San Luis Project, worked on the following phases of construction: spillway, piezometer, terminal wells, San Luis Pumping and Generating Plant, including service yard and switchyard and intake structure.

9/67 - 12/67—At Stampede Dam, Truckee, California, when was principal inspector over full inspection operations of the concrete spillway and outlet works tunnel. Consisted of 1) excavation for spillway and outlet works stilling basins, wheer footings, and trenches for perforated drain pipe; 2) drilling and grouting of anchor bars; installation of perforated drain pipe; 3) placing of reinforcement steel, concrete, and finish work; 4) formwork, including embedded materials.

12/67 - 5/68—Surveys, U.S.B.R.

5/68 - 12/68—Resumed construction work at Stampede Dam.

12/68 - 5/69—Surveys, U.S.B.R.

5/69 - 4/70—Principal inspector on above plus riprap: placing riprap material adjacent to spillway; chainlink fence; backfill; second stage concrete intake (pressure grouting); gate chamber, and stage (pressure grouting).

4/70 - 10/71—Principal inspector at South Nevada Water Project with responsibilities over all inspection operations of concrete structures, including electrical, mechanical, and tunnel operations.

10/71 - Present—Principal inspector covering excavation, installation of materials and equipment, backfill and embankment. This includes placement of earth, concrete, metalwork, and technical equipment.

Charles Entwisle

5/62 - 11/63—Construction Inspector on U.S.B.R. Flaming Gorge Unit. Inspected pressure grouting of contraction joints and inspected and directed grouting of rock foundation. Included inspection of concrete cooling, testing, assembling, and embedding stress, strain, and joint meters.

11/63 - 11/64—Construction Inspector on U.S.B.R. project at Sanford, Texas. Checked forms for line, grade, bracing in compliance with the specifications. Checked reinforcing for size, spacing, position, clearance, quantities, laps, and shapes. During placement of concrete, checked mix, vibration, finishes, protection.

11/64 - 5/70—Inspected tunnel excavation as it progressed on the U.S.B.R. San Juan-Chama Project, Chama, New Mexico. Informed the tunnel chief of any changes that occurred in excavated material and assisted in determining the protection needed on shale surfaces. Took gas tests and kept complete and accurate records of gas. Inspected concrete placement for consolidation, finishing, repair, and curing. Inspected gunite protective coating installation. From 1/66 to 4/67 of this time also inspected conventional tunnel operations through glacial till and reported changes of material, installation of steel tunnel liner plates and backfill behind liner plates.

5/70 - 12/70—Inspected drilling and grouting operation on U.S.B.R. San Juan-Chama Project. Inspected placement of dirt fill in dam for moisture content, required compaction, unsuitable materials in fill, proper slopes, placement of proper zone materials in right places, and that fill materials did not freeze.

12/70 - 10/72—Inspected contractor operations at the Grand Coulee Third Powerplant in Washington. Inspected various operations of concrete, dirt, and blasting.

10/72 - —Teton Dam. Chief shift inspector for grouting work directly under the Chief of Grouting. Drilling and grouting was done on three curtain grouting on left and right abutments in welded tuff and basalt area of cutoff trench, backfill grouting of diversion tunnel and one curtain grouting on left abutment and cutoff trench.

Karl S. Justesen

9/62 - 6/65—Surveys, U.S.B.R.

College

3/66 - 9/66—Surveys, U.S.B.R.

College

6/67 - 11/67—Began work on Starvation Dam, U.S.B.R. Project at Duchesne, Utah, as Construction Inspector. Inspected dam foundation excavation, cutoff trenches, stripping and pre-watering of borrow, placement of embankment material, proper thickness of lifts, moisture control, and rolling of embankment. Inspected final cleanup of foundations and abutments, and special compaction along abutment.

11/67 - 1/68—In charge of shift on grouting operations on Starvation Dam. Inspected and instructed the layout of holes to be drilled and grouted. Inspected water testing of holes, depth of stage drilled, pressure to be used. Inspected the grouting of holes, various mixes to be used, and when to use.

1/68 - 5/68—Assigned to Starvation Dam outlet works tunnel. Inspected final cleanup, setting reinforcement steel, checking the cleanup oiling, setting of steel forms, ordering concrete and actual placement and finishing of concrete.

5/68 - 10/68—Shift inspector on Starvation Dam (earthfill dam). Inspected excavation for dam foundation, cutoff trenches, borrow areas, and embankment operations. Inspected final cleanup of abutments and specially compacted zone 1 embankment, and placing of riprap around inlet and outlet of Starvation Tunnel.

10/68 - 3/70—Shift inspector on Strawberry Aqueduct, Water Hollow Tunnel and Channel No. 2 Tunnel. Inspected tunnel excavation, setting of support steel, centers to be set on, and use of logging.

3/70 - 5/71—Shift inspector on concrete operations of Water Hollow Tunnel.

5/71 - 11/71—Shift inspector on the placing of concrete in both the upper and lower tunnels, both upper and lower gate chambers, the chute from the upper tunnel to the outlet works and the outlet works for both tunnels of the Soldier Creek Dam.

11/71 - 6/73—Construction inspector at U.S.B.R. office in Provo, Utah. Served as shift inspector on the Jordan Aqueduct. Inspected trench excavation, the slope of banks, and subbase that pipe was to be laid on. Inspected the laying of pipe, checking of joints after pipe was laid and inspected the placing and compacting of pipe bedding after bedding was placed and compacted. Inspected tunnel excavation on three small tunnels under D&RG Railroad, and the State Highway. Inspected installing of liner plate, line and grade after completion of tunnel, and inspected the grouting of tunnel. Inspected the installation of reinforcement steel, preparing and actual placing of concrete for turnout and manholes along aqueduct; after placement inspected the finishing and repairs of concrete. Inspected the final cleanup of right-of-way, restoring of forms ditches, replacing of top rail, and the reseeding of right-of-way through nonagricultural land.

11/71 - —Teton Dam. Inspection on all phases of foundation grouting, blanket grouting, backfill grouting and tunnel radial hole grouting, grouting of trench drain under zone 1 embankment and in the powerhouse. Instructed inspector as to holes to be drilled, depth of hole, stage pressure, and mix to use.

APPENDIX D-2

CONSTRUCTION PHOTOGRAPHS

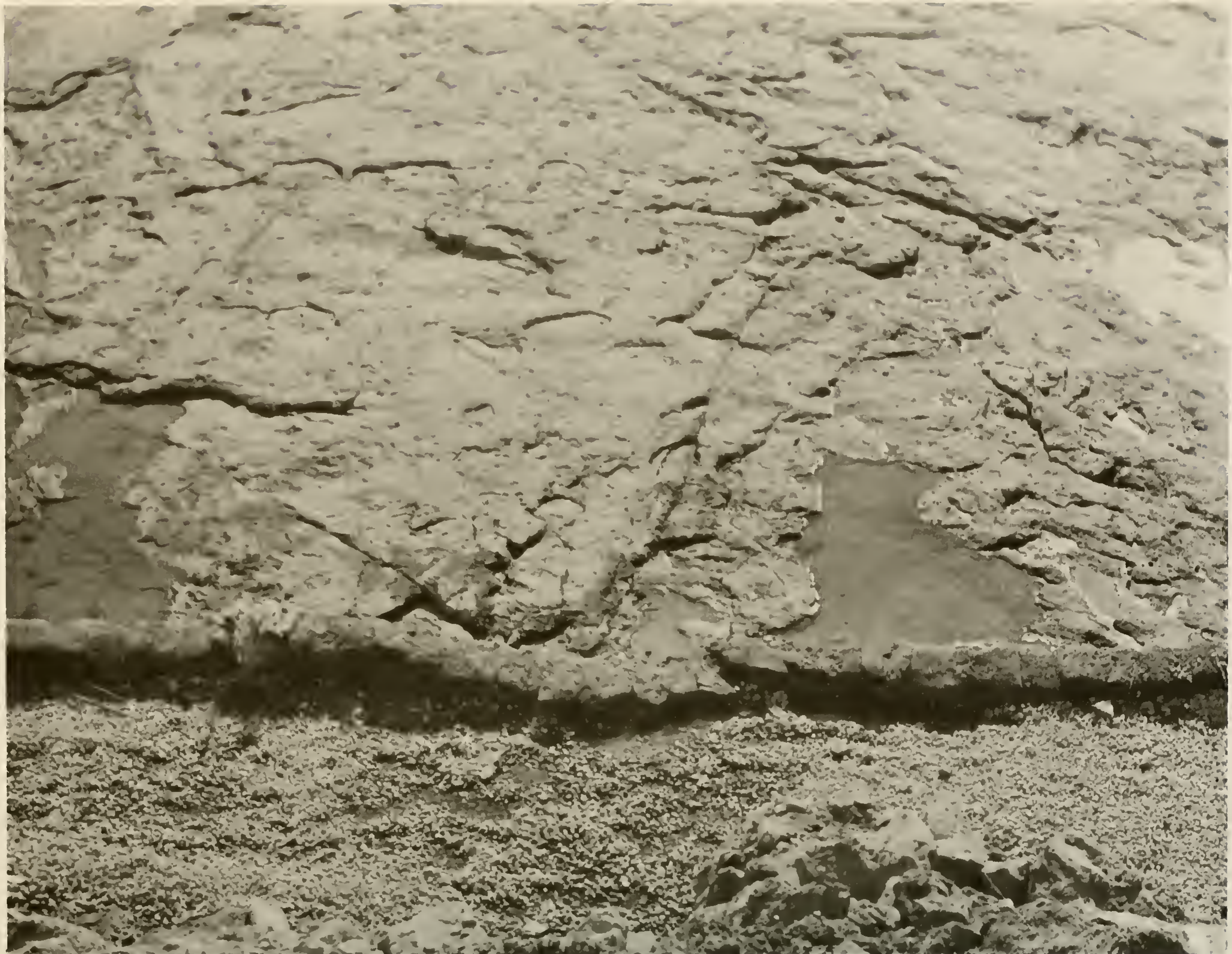
CONTENTS

CONTENTS—Continued

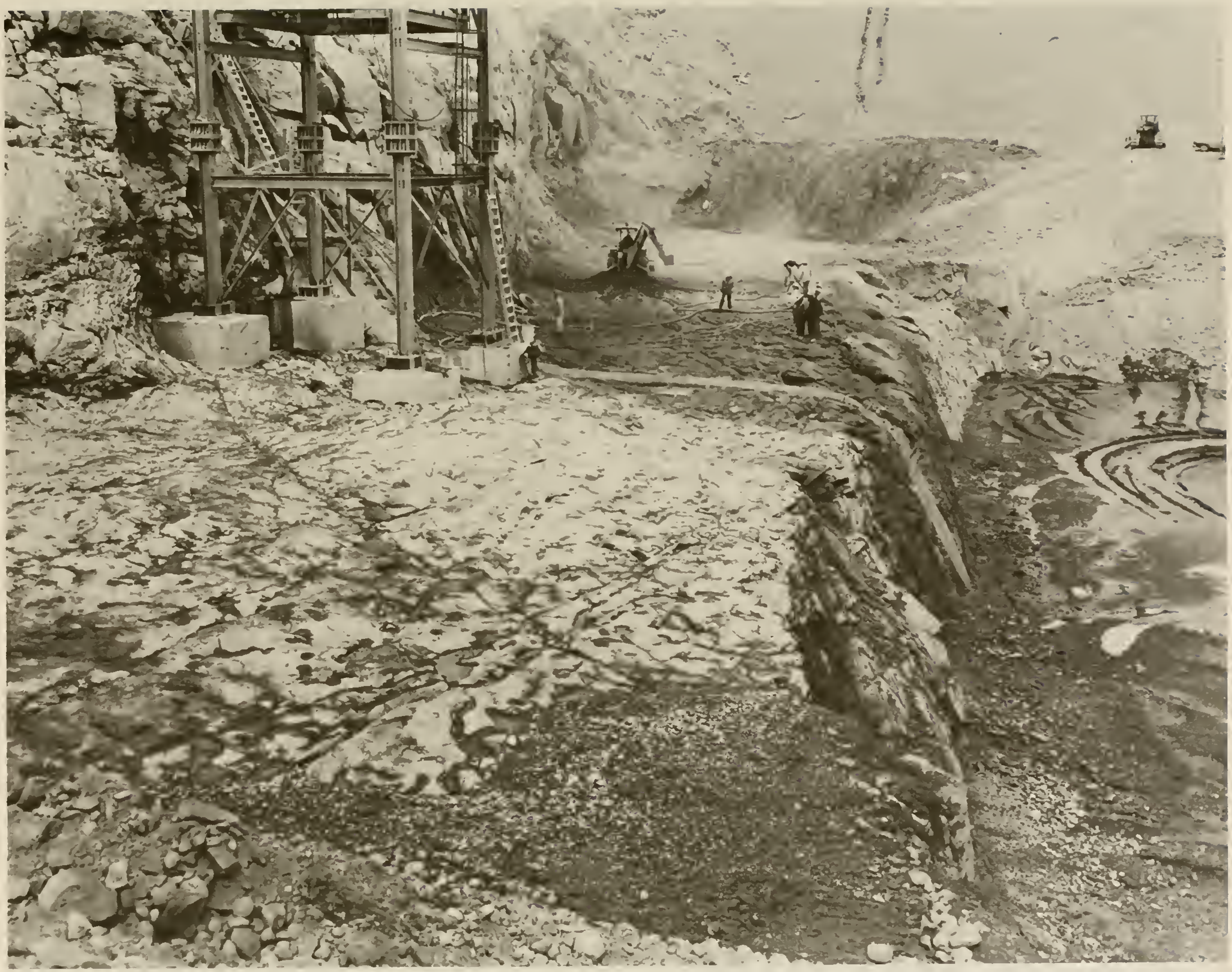
1.—View of right foundation keyway trench and zone 5 stockpile, also portion of spillway area, 7/19/73; P549-147-2912 NA. Note number of benches with adjacent steep faces.

2.–View of grout cap looking toward right abutment in cutoff trench. Photo taken at station 19+00, 100 feet upstream from centerline of dam. 10/9/73; P549-147-3297 NA.

3.—View of workmen cleaning rhyolite in bottom of cutoff trench. Photo taken at station 18+10, 90 feet upstream from centerline of dam, 10/15/73; P549-147-3356 NA.



4.—Bottom of cutoff trench, stations 18+75 to 18+25. Photo taken at station 18+50, 220 feet upstream from centerline of dam, 10/16/73; CN549-147-703 NA.



5.–View of cleanup on bench below conveyor-tower, stations 16+50 to 17+50. Photo taken at station 18+25, 60 feet downstream from centerline of dam, 4/3/74; P549-147-3816 NA.

6.—View of cleanup on bench below conveyor-tower, stations 16+50 to 17+50. Photo taken at station 18+25, 60 feet downstream from centerline of dam, 4/3/74; P549-147-3815 NA.

7.—View of cutoff trench. Photo taken at station 22+30, 180 feet upstream from centerline of dam, 4/12/74; P549-147-3994 NA.

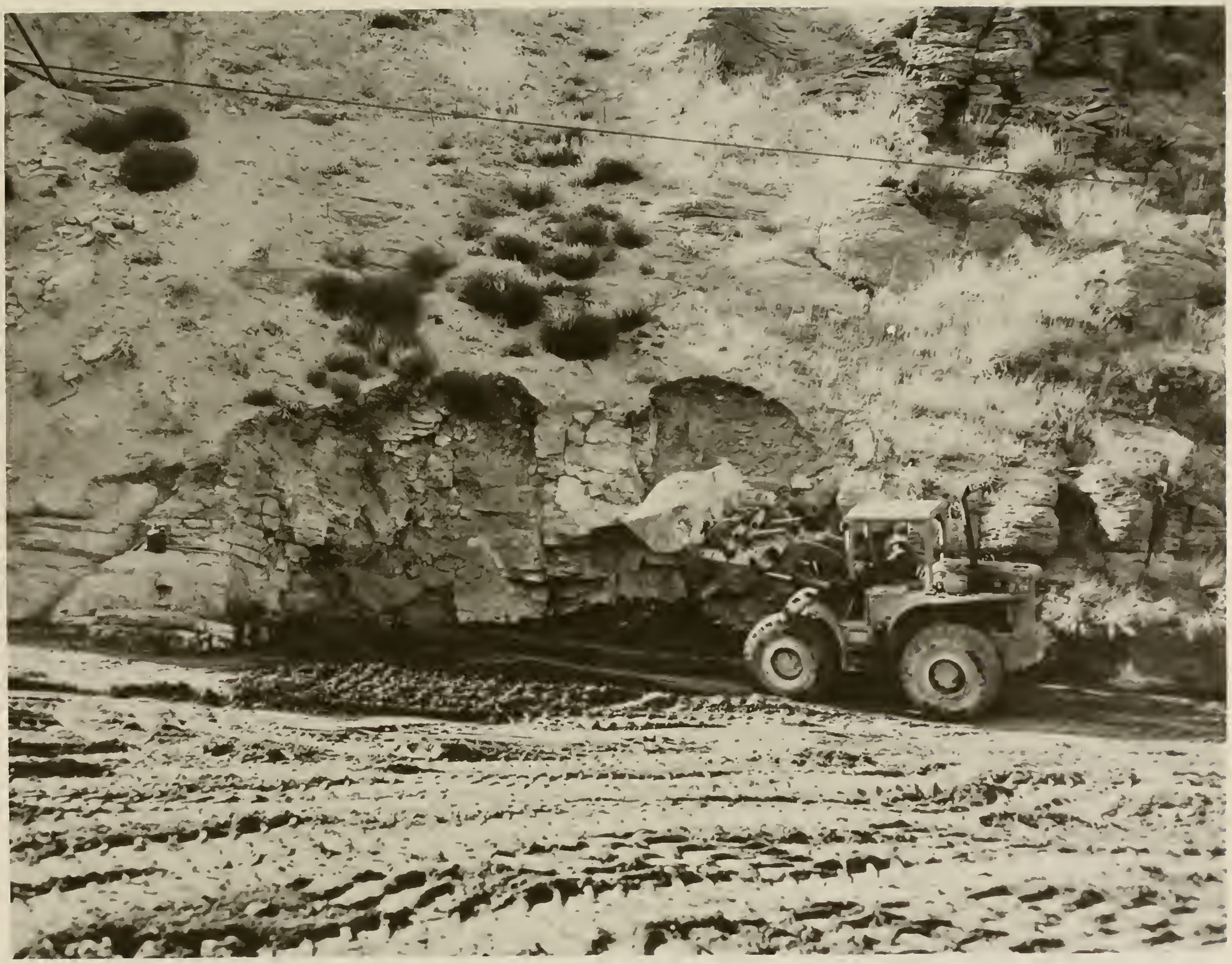
8.—Dewatering operations zone 1 cutoff trench to take care of seepage due to high water on the upstream side of the dam, 6/18/74; P549-147-4539 NA.

9.—View of sluicing operation on zone 2 blanket area. Material is sluiced into cracks between rocks to insure all voids are filled. 7/19/72; CN549-147-140 NA.

10.—View of grout leaking out of cracks in rocks, right abutment from 110 feet downstream to centerline of dam, 7/22/74; P549-147-4692 NA.

11.—Viewing zone 1 fill and preparation work. Taken from station 16+00, 550 feet upstream, 7/2/74; P549-147-4589 NA.

12.–Hydraulic hammer used for special compactor along abutment and around tower supports, 7/2/74; P549-147-4590 NA.



13.—Right abutment showing rock 260 to 300 feet upstream of centerline. Zone 1 embankment elevation is 5106, 10/11/74; P549-147-5044 NA.

14.—View of open joint on the right abutment at elevation 5142, station 14+52, 90 feet upstream ready to be grouted, 11/14/74; P549-147-5216 NA.

15.—View of laborer compacting zone 1 material along right abutment with plate tamper. Photo taken at station 17+60, 100 feet upstream from centerline of dam, 11/1/73; P549-147-3423 NA.

16.—Euclid end dump loaded with rock and earth being used for special compaction of zone 1 material next to rock face of the right abutment, 5/29/75; P549-147-5732 NA.

17.—Left abutment zone 1. Laborer using a hand held air driven compactor to compact zone 1 material up to rock face of downstream left abutment, 5/27/75; P549-147-5731 NA.

18.—Right key trench showing workmen cleaning up rock face, station 9+25, prior to the placement of zone 1 in the key trench, 6/26/75; P549-147-6241 NA.

APPENDIX D-3

CONSTRUCTION SPECIFICATIONS

FOR

EMBANKMENT EARTHWORK

AND

RELATED FOUNDATION TREATMENT

VOLUME 1

UNITED STATES

DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

– – –

Schedule, General Provisions, and Specifications

– – –

TETON DAM

AND

POWER AND PUMPING PLANT

– – –

Lower Teton Division

Teton Basin Project, Idaho

Bids will be received by the Bureau of Reclamation, at the Council Chambers in City Hall, Idaho Falls, Idaho, until 10 a.m. (local time at the place of bid opening), September 9, 1971. Bids submitted by mail should be addressed to the Bureau of Reclamation, c/o U.S. Geological Survey, Post Office Box 697, Idaho Falls, Idaho 83401.

(PRICE—VOLUMES 1, 2, 3, and 4—$17.00)

REPRINT

CONTENTS

SPECIFICATIONS

TETON DAM
AND
POWER AND PUMPING PLANT

LOWER TETON DIVISION
TETON BASIN PROJECT, IDAHO

SCHEDULE

Item	Work or material	Quantity and unit	Unit price	Amount
3	Excavation for dam embankment foundation above elevation 5040			
	a. First 300,000 cu yd	300,000 cu yd	$	$
	b. Over 300,000 cu yd	150,000 cu yd		
4	Excavation for dam embankment foundation below elevation 5040:			
	a. First 420,000 cu yd	420,000 cu yd		
	b. Over 420,000 cu yd	230,000 cu yd		
5	Excavation, in open cut, for structures:			
	a. First 310,000 cu yd	310,000 cu yd		
	b. Over 310,000 cu yd	160,000 cu yd		
6	Excavation for grout cap to 5 feet in depth	1,700 cu yd		
7	Excavation for grout cap between depths of 5 and 8 feet	170 cu yd		
	Excavation, stripping borrow pits:			
8	In Borrow Areas A and B	230,000 cu yd		
9	In Borrow Area C and C Extension	120,000 cu yd		
10	Excavation in Borrow Area C and C Extension and stockpiling	2,700,000 cu yd		
11	Excavation in Borrow Areas A and B and transportation to dam embankment:			
	a. First 4,600,000 cu yd	4,600,000 cu yd		
	b. Over 4,600,000 cu yd	2,400,000 cu yd		

SCHEDULE—Continued

Item	Work or material	Quantity and unit	Unit price	Amount
27	Earthfill in dam embankment, Zone 1	4,800,000 cu yd	$	$
28	Specially compacted earthfill, Zone 1	41,000 cu yd		
29	Placing sand, gravel, and cobble fill in dam embankment, Zone 2	500,000 cu yd		
30	Procuring and placing sand, gravel, and cobble fill in dam embankment, Zone 2	2,000,000 cu yd		
31	Miscellaneous fill in dam embankment, Zone 3	860,000 cu yd		
32	Placing silt, sand, gravel, and cobble fill in dam embankment, Zone 4	220,000 cu yd		
33	Procuring and placing silt, and gravel, and cobble fill in dam embankment, Zone 4	330,000 cu yd		
34	Rockfill in dam embankment, Zone 5	790,000 cu yd		

DIVERSION AND CARE OF RIVER DURING CONSTRUCTION AND REMOVAL OF WATER FROM FOUNDATIONS

63. DIVERSION AND CARE OF RIVER DURING CONSTRUCTION AND REMOVAL OF WATER FROM FOUNDATIONS

a. General.—The contractor shall construct and maintain all necessary cofferdams, channels, flumes, drains, sumps, and/or other temporary diversion and protective works and temporary stream crossings; shall furnish all materials required therefor; shall furnish, install, maintain, and operate all necessary pumping and other equipment for removal of water from the various parts of the work and for maintaining the foundations and other parts of the work free from water as required for constructing each part of the work. The downstream cofferdams shall be located so as to minimize silting of the tailrace channel from underwater excavation of cofferdams and wet placement of bedding for riprap and riprap.

All cofferdams or other temporary diversion and protective works constructed upstream from the dam and not a part of the permanent dam embankment shall be removed or leveled and graded to the extent required to prevent obstruction in any degree whatever of the flow of water to the spillway or outlet works.

The contractor shall be responsible for and shall repair at his expense any damage to the foundations, structures, or any other part of the work caused by floods, water, or failure of any part of the diversion or protective works.

All contractor operations shall be such as to prevent pollution of the river in accordance with Paragraph 60.

b. Plan.—Prior to beginning any work on diversion and care of river and removal of water from foundations, the contractor shall submit to the Construction Engineer for approval, a water control plan showing his proposed method for the diversion and care of the river during construction and removal of water from foundations. For payment purposes, the plan shall be in not more than 12 major divisions as provided in Subparagraph f. The plan may be placed in operation upon approval, but nothing in this paragraph shall relieve the contractor from full responsibility for the adequacy of the diversion and protective works.

c. Diversion and care of river.—Because of fish requirements, and because of downstream irrigation requirements between May 1 and September 30, the contractor shall not interrupt the natural flow of the Teton River through the damsite except that the contractor will be permitted to reduce such flow in the amount of water available for construction purposes as provided in Paragraph 52 and during the period of closure of the river outlet works the contractor will be permitted to reduce the flow past the dam as provided in Subparagraph (6) below.

For the purpose of diverting the flow past the damsite the contractor will be permitted to:

(1) Pass the flow over the dam embankment completed to the top of the cutoff trench: Provided, That flow through the damsite shall not be raised higher than the elevation of the present channel.

(2) Use temporary pipe, flume, or other approved facilities for conveying flow through the damsite. Drop structures shall be constructed as required to minimize erosion. No pipe, flume, or other facility will be permitted to be embedded in the dam embankment.

(3) Leave a temporary gap in the dam embankment in accordance with Paragraph 86: Provided, That flow through the damsite shall not be raised higher than the elevation of the present channel.

(4) Use the river outlet works for diversion purposes after completion of the following construction, both temporary and permanent:

(a) River outlet works diversion inlet channel.

(b) Open-cut excavation above intake structure and gate chamber.

(c) Open-cut excavation for the canal outlet works structure, and for the river outlet works structures and outlet channel upstream from Station 34+70.

(d) Open-cut excavation for the power and pumping plant, tailrace retaining walls, and 80-foot-wide tailrace channel immediately downstream from the powerplant.

(e) All other required open-cut excavation which requires blasting within 50 feet of completed portions of the structures.

(f) All first-stage concrete in the river outlet works structures, except concrete in intake structure above elevation 5141.00 and in shaft house and shaft above the construction joint at elevation 5295.

(g) Bypass in diversion inlet consisting of 36-inch slide gate, trashrack, lift, and pipe.

(h) Diversion inlet stoplog seats and blockout concrete at upstream end of diversion inlet structure.

(i) Pressure grouting of foundation surrounding river outlet works intake structure, tunnel, gate chamber, and shaft.

(j) Embedded 36-inch pipe in channel lining for canal outlet branch.

(k) Reservoir level gage piping which is embedded in concrete.

(l) Embedded 13.5-foot inside-diameter steel penstock between gate chamber and downstream portal.

(m) Upstream embedded penstock transition leading to the 10-foot 6-inch by 13-foot 6-inch wheel-mounted gate chamber.

(n) Approved temporary protection of first-stage concrete, reinforcing, waterstops, recesses for second-stage concrete, grout outlets and vents, covers for wall openings in channel lining, recess for canal outlet branch, and upstream edge of stilling basin dividing wall.

(o) Pervious backfill and riprap for stilling basin as directed.

(p) Minimum size outlet channel downstream from stilling basin.

The upstream Zone 4 portion of the dam embankment may be incorporated into the cofferdam during the period of diversion through the outlet works.

(5) Use the auxiliary outlet works for diversion purposes in conjunction with use of the river outlet works for diversion purposes in accordance with (4) above, after completion of the following:

(a) Auxiliary outlet works complete below elevation 5295.00, including a reliable temporary power source for gate operation and temporary controls for gate operation, furnished by and at the expense of the contractor and as approved by the contracting officer.

(b) Spillway complete downstream from the contraction joint at Station 33+84, including completion of riprap in the outlet channel.

(6) The auxiliary outlet works shall be used as the only method of diversion during the period of closure of the river outlet works as provided below. The period of closure of the river outlet works shall begin on October 1 and shall be completed by the following April 30. Prior to use of the auxiliary outlet works as the only method of diversion and prior to initiating closure of the river outlet works the following construction shall be completed:

(a) Dam embankment complete to elevation 5332.0.

(b) All first-stage concrete in river outlet works intake structure and installation of intake bulkhead seat and gate and trashracks.

(c) All first-stage concrete in river outlet works shaft house and shaft.

(d) Complete spillway, including completion of riprap in the outlet channel.

(e) Complete auxiliary outlet works, including a reliable temporary or permanent power source for gate operation. This power shall be available for the remainder of the contract period.

(f) Canal outlet works and control structure and Enterprise-East Teton feeder pipeline complete to access road Station 19+00.

Prior to diversion through the auxiliary outlet works, all sediment, rocks, and debris shall be removed from the auxiliary outlet works and the spillway stilling basin.

During the period of diversion through the axuiliary outlet works and prior to April 30, the contractor shall: close the river outlet works and place the second-stage concrete in the intake structure and gate chamber; install the outlet works gates, penstock manifold, outlet pipe and other appurtenant metalwork and equipment; and complete the tailrace channel.

The contractor shall complete the closure and perform all construction required to complete the river outlet works and the powerplant tailrace, during the specified closure period to the extent that the full release of available water may commence through the river outlet works by May 1. Prior to May 1, the contractor shall provide reliable temporary or permanent power source for testing and operating all gates in the spillway and river outlet works. The power source shall be available for the remainder of the contract period.

d. Removal of water from foundations.—The contractor's method of removal of water from foundation excavations shall be subject to the approval of the contracting officer. The use of a sufficient number of properly screened wells or other equivalent methods will be approved for dewatering.

Where excavation for the cutoff trench and for the foundation key trenches in embankment foundations, and excavation for the spillway, outlet works, power and pumping plant structures, and for the pipelines, extends below the water table in common material, the portion below the water table shall be dewatered in advance of excavation. The dewatering shall be accomplished in a manner that will prevent loss of fines from the foundation, will maintain stability of the excavated slopes and bottom of the excavation, and will result in all construction operations being performed in the dry. The contractor shall be responsible for and shall repair at his expense any unstable slopes and structural damage that may occur during the dewatering operations.

The contractor will also be required to control seepage along the bottom of the dam embankment cutoff trench and the foundation key trenches, which may require supplementing the approved dewatering systems by pipe drains leading to sumps from which the water shall be pumped. Such pipe drains shall be of uniform diameter for each run, shall be provided with grout connections and returns at 50-foot intervals, and shall be embedded in reasonably well-graded gravel or like material.

During the placing and compacting of the embankment material in the dam embankment cutoff trench, the water level at every point in the cutoff trench shall be maintained below the bottom of the embankment until the compacted embankment in the cutoff trench at that point has reached a depth of 10 feet, after which the water level shall be maintained at least 5 feet below the top of the compacted embankment. When the embankment has been constructed to an elevation which will permit the dewatering systems to maintain the water level at or below the designated elevations, as determined by the contracting officer, the pipe drains and sumps including surrounding gravel, shall be filled with grout composed of water and cement or clay.

EARTHWORK

66. DEFINITIONS OF MATERIALS

Materials excavated will not be classified for payment. For purposes of these specifications, other than for payment, materials of earthwork and embankment construction are defined in detail as follows:

a. Rock.—A sound and solid mass, layer, or ledge of mineral matter in place and of such hardness and texture that it:

(1) Cannot be effectively loosened or broken down by ripping in a single pass with a late model tractor-mounted hydraulic ripper equipped with one digging point of standard manufacturer's design adequately sized for use with and propelled by a crawler-type tractor rated between 210- and 240-net flywheel horsepower, operating in low gear, or

(2) In areas where the use of the ripper described above is impracticable, rock is defined as sound material of such hardness and texture that it cannot be loosened or broken down by a 6-pound drifting pick. The drifting pick shall be Class D, Federal Specification GGG-H-506d, with handle not less than 34 inches in length.

b. Common material.—All earth materials which do not meet the requirements of rock as defined in a. above.

c. Formation.—Any sedimentary, igneous, or metamorphic material represented as a unit in geology, generally called rock but not necessarily meeting the classification requirements for rock in a. above.

d. Cobbles.—Rounded pieces of rock which are not greater than 12 inches, but are larger than 3 inches in maximum dimension.

e. Boulders.—Detached pieces of rock, generally rounded but may be subrounded to angular, which are larger than 12 inches in maximum dimension.

f. Rock fragments.—Pieces of rock which generally are not rounded.

g. Soil components.—NOTE: Soils in nature usually consist of a number of soil components. They are identified by the predominance of one of the components and other criteria given in the Unified Soil Classification System. See notes on drawings of logs of explorations.

(1) Clay.—Plastic soil which passes a United States Standard No. 200 sieve.

(2) Silt.—Nonplastic soil which passes a United States Standard No. 200 sieve.

(3) Sand.—Mineral grains which pass a United States Standard No. 4 sieve and are retained on a United States Standard No. 200 sieve.

(4) Gravel.—Pieces of rock which are not greater than 3 inches in maximum dimension, and are retained on a United States Standard No. 4 sieve.

h. Other materials.—

(1) Slopewash.—Pebble- to boulder-size angular fragments of rhyolite or tuff in a silty soil matrix.

(2) Talus.—Pebble- to boulder-size angular fragments of rhyolite or tuff.

(3) Alluvium.—Silt, sand, gravel, and some cobbles.

(4) Sandstone.—Sand that has been cemented to some degree.

(5) Siltstone.—Silt that has been cemented to some degree.

(6) Claystone.—Clay that has been cemented to some degree.

(7) Rhyolite.—Fine-grained mass of natural minerals, hard and coherent except along fractures and where locally weathered.

(8) Tuff.—A material of relatively light weight formed by consolidation of volcanic ash and dust.

(9) Lapilli tuff.—Tuff with numerous volcanic fragments that are larger (generally 4 to 32 millimeter) than fragments comprising matrix.

(10) Welded tuff.—Used synonymously with rhyolite in this case.

(11) Basalt.—Very fine-grained mass of natural minerals, hard and coherent except along fractures and where locally weathered.

67. OPEN-CUT EXCAVATION, GENERAL

a. General.—All open-cut excavation required for the dam, power and pumping plant, tailrace channel, and appurtenant works shall be performed in accordance with this paragraph and Paragraphs 68 to 70, inclusive. Open-cut excavation in borrow areas shall be in accordance with Paragraphs 71 and 72. Excavation for pipe trenches and structures for the Enterprise-East Teton Feeder Pipeline between Stations 0+00 and 58+50, and for the pump discharge line between Stations 3+60.93 and 12+03.50, shall be in accordance with Paragraphs 291 and 294. Excavation for roadway shall be in accordance with Paragraph 312.

Excavation shall be made to the lines, grades, and dimensions shown on the drawings or established by the contracting officer.

During the progress of the work, it may be found necessary or desirable to vary the slopes, grades, or the dimensions of the excavations from those specified herein. Any increase or decrease of quantities excavated as a result of such variations will be included in the estimates. However, if the contracting officer determines that the contractor's costs of performing the work will be increased by reason of such variations, an equitable adjustment will be made to cover such increased costs. Otherwise, the work will be paid for at the unit prices bid therefor in the schedule regardless of such variations.

Where ripping is used for excavation the ripper tooth shall not be operated closer than 4 feet to the final excavation line and the excavation shall be completed by light blasting, wedging, barring, channeling, line drilling and broaching, or other suitable methods approved by the contracting officer.

All blasting for excavation shall be subject to the provisions of this paragraph and to the provisions of Paragraph 53. All necessary precautions, including control of blasting, shall be taken to preserve the material below and beyond the established lines of all excavation in the soundest possible condition. Whenever, in the opinion of the contracting officer, further blasting might injure the material upon or against which concrete is to be placed, the use of explosives shall be limited to light loads, and the excavation shall be completed by light blasting, wedging, barring, channeling, line drilling and broaching, or other suitable methods approved by the contracting officer. Any damage to the work due to the contractor's operations, including shattering of the material beyond the required excavation lines, shall be repaired at the expense of and by the contractor. Slopes shattered or loosened by blasting shall be taken down at the expense of and by the contractor.

Blasting shall be done using lift (or bench) heights not greater than 24 feet and using blast holes having diameters not greater than 3 and 1/2 inches: Provided, That as the excavation for structure foundations approaches the final excavation lines, the depths of the blast holes shall not exceed one-half of the depth of the rock remaining above the final grade: Provided further, That the final 5 feet of rock to be excavated for structure foundations shall be blasted and excavated as a separate operation.

Any and all excess excavation for the convenience of the contractor or overexcavation performed by the contractor for any purpose or reason, except as may be ordered in writing by the contracting officer, and whether or not due to the fault of the contractor, shall be at the expense of the contractor. Where required to complete the work, all such excess excavation and overexcavation shall be refilled with materials furnished and placed at the expense of and by the contractor: Provided, That payment will be made for cement used in concrete placed to refill such excess excavation or overexcavation unless such excess excavation or overexcavation is caused by careless excavation or is intentionally performed for the convenience of the contractor to facilitate his operations, as determined by the contracting officer.

All excavation for embankment and structure foundations shall be performed in the dry. No excavation shall be made in frozen materials without written approval. No additional allowance above the unit prices per cubic yard bid in the schedule for excavation will be made on account of any of the materials being wet or frozen.

Excavations shall be made to the full dimensions required and shall be finished to the prescribed lines and grades except that individual sharp points of undisturbed formation material will be permitted to extend within the prescribed lines not more than 6 inches where the excavation surfaces are not to be covered with concrete or are not required as a foundation for pipe.

d. Excavated materials.—So far as practicable, as determined by the contracting officer, all suitable materials from excavations for specified permanent construction shall be used in the permanent construction required under these specifications.

Materials shall be selected as follows:

(1) Mixtures of clay, silt, sand, gravel, and cobbles less than 5 inches in maximum dimensions shall be used in Zone 1 of the dam embankment.

(2) Reasonably well-graded pervious mixtures of sand, gravel, and cobbles less than 12 inches in maximum dimension shall be used in Zone 2 of the dam embankment.

(3) Miscellaneous mixtures of clay, silt, sand, gravel, cobbles, and rock fragments less than 12 inches in maximum dimension shall be used in the Zone 3 of the dam embankment.

(4) Mixtures of silt, sand, gravel, and cobbles less than 12 inches in maximum dimensions shall be used in the Zone 4 of the dam embankment and for Zone 4 to the left of the tailrace channel.

(5) Cobbles, boulders, and rock fragments less than 1 cubic yard in volume shall be used in the Zone 5 of the dam embankment.

Any roots larger than one-fourth inch in diameter excavated with otherwise suitable material shall be removed prior to compaction on the embankment.

Excavated materials which are unsuitable for or are in excess of dam embankment or other earthwork requirements, as determined by the contracting officer, shall be wasted as provided in Paragraph 73.

The contractor's blasting and other operations in excavations shall be such that the excavations will yield as much suitable material for such construction as practicable, and shall be subject to the approval of the contracting officer. Where practicable, as determined by the contracting officer, suitable materials shall be excavated separately from the materials to be wasted and the suitable materials shall be segregated by loads during the excavation operations. The suitable materials shall be placed in the designated final locations directly from the excavation, or shall be placed in temporary stockpiles and later placed in the designated locations as directed by the contracting officer. In excavating materials which are suitable for use in embankment, the contracting officer will designate the depths of cut which will result in the best gradation of materials, and the cuts shall be made to such designated depths.

If, after excavation, sand, gravel, and cobble fill, Zone 2 material, or silt, sand, gravel, and cobble fill, Zone 4 material has a moisture content greater than that required for placement and compaction in embankment, the material shall not be placed on the embankment, but shall be placed temporarily in stockpiles and allowed to drain or dry until the moisture content is reduced sufficiently to permit it to be placed in the embankment.

e. Measurement and payment.—Excavated material will be measured, for payment, in excavation to the lines shown on the drawings or described in these specifications, or when lines are not shown on the drawings or described in these specifications, then as prescribed by the contracting officer, and will include only material that is actually removed within the prescribed pay lines.

Where concrete is to be placed directly upon or against the excavations, such excavations shall be sufficient at all points to provide for the minimum dimensions of concrete. Where dimensions of a concrete structure are shown on the drawings or if the elevation of the foundation is indicated, such dimensions shall be considered as the minimum dimensions and such elevation shall be considered as the elevation determining the minimum dimensions of the structure. Where a dimension or an elevation is not indicated on the drawings, minimum dimensions will be established by the contracting officer.

Where concrete is to be placed directly upon or against the excavations and the character of the material cut into is such that the material can be trimmed efficiently to accurate dimensions by oridinary excavation finishing methods to the required lines of the concrete structure, as determined by the contracting officer, measurement for payment will be made only of the excavation within the neatlines of the concrete structure.

Where concrete is to be placed directly upon or against the excavations and the character of the material cut into is such that the material cannot be trimmed efficiently to accurate dimensions by ordinary excavation finishing methods, as determined by the contracting officer, measurement for payment thereof will be made to the prescribed average dimension lines. The prescribed average dimension lines shall be considered as 3 inches outside the neatlines of the concrete for the purposes of measurement, for payment, of excavation.

Measurement, for payment, of excavations upon or against which concrete is not required to be placed will be limited to the neatlines shown on the drawings or, where not shown on the drawings, to the most practicable lines, grades, and dimensions as established by the contracting officer.

Except as otherwise provided in Paragraph 63 for diversion and care of river during construction and removal of water from foundations, the unit prices bid in the schedule for excavation in open-cut shall include the cost of all labor, equipment, and materials for cofferdams and other temporary construction and of all pumping, bailing, draining, and all other work necessary to maintain the excavations in good order during construction and of removing such temporary construction where required.

The unit prices bid in the schedule for excavation in open-cut shall also include the entire cost of:

(1) Excavating to designated depths.

(2) Segregating materials by loads.

(3) Draining or drying otherwise suitable materials.

(4) Transportation of materials from the excavation to points of final use, to disposal areas, to temporary stockpiles, and from temporary stockpiles to points of final use.

(5) Rehandling excavated materials which have been deposited temporarily in stockpiles.

(6) Disposal of excavated waste materials.

(7) Preparation of concrete structure foundations.

All excavated materials actually placed in complete embankment construction will again be included for payment under appropriate items of the schedule covering such construction.

Except as provided in Subparagraph b. above, no payment will be made for excavation performed in previously placed embankment, refill, or backfill.

68. EXCAVATION FOR DAM EMBANKMENT FOUNDATION

a. General.—Excavation for dam embankment foundation shall be in accordance with this paragraph and Paragraph 67, and includes all:

(1) Stripping for foundation of dam embankment.

(2) Excavation below stripping for foundation of dam embankment, including cutoff trench, foundation key trench, and other areas if required, but not including excavation for grout cap.

(3) Excavation for the river outlet works shaft and shaft house above elevation 5295.00.

(4) Excavation for the open drain shown on Drawing No. 4 (549-D-8).

The alinements and excavation lines shown on the drawings are subject to such changes as may be found necessary by the contracting officer to adapt the dam foundation excavation to the conditions disclosed by the excavation.

Accurate trimming of the slopes of the excavation will not be required, but the excavation shall conform as closely as practicable to the established lines and grades. Loose rock shall be removed from foundation contacts and rock cliffs, ledges, overhangs and sharp irregularities shall be reduced to provide satisfactory foundation contours. The finished foundation controus against which the earthfill, Zone 1 portion of the dam embankment is to be placed shall be not steeper than 1/2:1, and shall be cleaned of all loose, soft, and disintegrated materials, including removal of such materials from pockets, irregularities, fissures, and depressions in the foundation.

b. Stripping.—The entire area to be occupied by the dam embankment, except the area beyond the toe of the downstream Zone 2. 1 and 1/2:1 slope, which is to be covered by Zone 4, shall be stripped to a sufficient depth, as determined by the contracting officer, to remove all unsuitable materials. The unsuitable materials to be removed by stripping shall include all surface boulders and loose rock; debris; topsoil; vegetable matter, including stumps and roots; and all other perishable and objectionable materials that are unsuitable for use in permanent construction required under these specifications or that might interfere with the proper bonding of the embankment with the foundation, or the proper compaction of the materials in the embankment, or that may be otherwise objectionable as determined by the contracting officer. Materials from stripping operations shall be wasted in areas upstream from the dam embankment and as provided in Paragraph 73.

c. Excavation below stripping for cutoff trench and foundation key trench.—The cutoff trench shall be excavated to firm formation, and the foundation key trench shall be excavated into the firm formation, as shown on Drawings No. 4 (549-D-8) and 5 (549-D-9).

d. Measurement and payment.—Measurement, for payment, of excavation for dam embankment foundation will be made as provided in Paragraph 67.

The dividing surfaces for measurement for payment between excavation for dam embankment foundation and:

(1) Excavation for roadway will be a vertical plane normal to the centerline of the roadway at roadway Station 1+50.00.

(2) Excavation, in open-cut, for structures will be those surfaces described in Paragraph 69.

(3) Excavation for grout cap will be the bottom of the excavation for dam embankment foundation as shown on the concrete grout cap detail on Drawing No. 5 (549-D-9).

Payment for excavation for dam embankment foundation will be made at the applicable unit price per cubic yard bid in the schedule for excavation for dam embankment foundation above elevation 5040, and for excavation for dam embankment foundation below elevation 5040, and shall include all costs as provided in Paragraph 67 and all costs of cleanup of foundation surfaces as provided in Subparagraph a. above.

70. EXCAVATION FOR GROUT CAP

Excavation for grout cap shall be in accordance with this paragraph and Paragraph 67.

Excavation for grout cap shall be performed by the use of handtools and approved mechanical equipment, in such a manner as to prevent shattering of the sides and bottom of the excavation. At the option of the contractor and with the approval of the contracting officer, line drilling and light blasting, or other similar methods may be employed. If line drilling and light blasting are employed, the diameter, spacing, and depth of the holes shall be subject to the approval of the contracting officer, and the spacing shall be such as to insure that the material will break along the desired lines. The light blasting shall be limited to approved methods which provide for successive fracturing of the worked face as the work is advanced by use of power tools and handwork. Whenever, in the opinion of the contracting officer, further blasting might injure the surfaces upon or against which concrete is to be placed, the use of explosives shall be discontinued.

When an excavation for grout cap crosses a fault or seam, the excavation shall be carried to depths shown on the drawings or as may be directed and shall be keyed into the formation on the sides of the fault or seam as directed: Provided, That if excavation is required to a greater depth than 8 feet measured normal to the finished surface of excavation for dam embankment foundation, such excavation will be ordered in writing in accordance with Clause No. 3 of the General Provisions.

The contractor shall furnish all materials to support the sides of the excavation where necessary, and all supports shall be removed before or during the placing of concrete.

Measurement, for payment, of excavation for grout cap will be made to the prescribed average dimension in width, and to the designated depth measured normal to the finished surfaces of excavation.

The measurement will not include excavation for the trench under the spillway inlet structure between vertical planes 50.5 feet left and right of the spillway centerline. Excavation for the trench in this reach will be included in the measurement for payment for excavation, in open cut, for structures.

Payment for excavation for grout cap will be made at the applicable unit price per cubic yard bid in the schedule for excavation for grout cap to 5 feet in depth, and for excavation for grout cap between depths of 5 and 8 feet, which unit prices shall include all costs as provided in Paragraph 67, and the cost of line drilling and light blasting if employed, the cost of furnishing, installing, and removing supports, and the cost of all other work described by this paragraph.

The requirement for excavation for grout cap between depths of 5 and 8 feet is uncertain and will be governed by conditions encountered as the work progresses, as determined by the contracting officer. The contractor will be entitled to no additional allowance above the unit price bid therefor in the schedule by reason of any amount or none of the work for this item being required.

71. BORROW AREAS A AND B

a. General.—All materials required for construction of dam embankment Zones 1 and 3, and of specially compacted earthfill, Zone 1, which are not available from excavations required for permanent construction under these specifications, shall be obtained from Borrow Areas A and B. The locations of the borrow areas are shown on Drawings No. 442 (549-D-165) and 443 (549-D-166).

Explorations in the borrow areas indicate that the materials are variable in nature and texture and contain variable amounts of moisture and oversize materials, and of hard calcareous layers and caliche areas. Approximate percentages of oversize materials and the location of hard strata are indicated on the logs. The absence of percentages of oversize or hard strata on any log of explorations within the area does not, however, imply that oversize materials or hard layers will not be encountered in the vicinity of such explorations.

Ground-water level encountered in the explorations, as shown on the logs, is for the indicated dates. The absence of a ground-water level or moisture content on any log of explorations within the areas does not, however, imply that ground water or variable moisture content will not be encountered in the vicinity of such explorations.

Bidders are cautioned that wide variation from the nature, texture, moisture content, and the percentage of oversize material as indicated by the explorations, is to be anticipated. Bidders and the contractor must assume all responsibility for deductions and conclusions concerning the nature, moisture content, and texture of materials, the percentages of oversize materials and hard calcareous layers, the total yield of suitable materials, the difficulties of making excavations, of breaking down or removing the oversize materials, of obtaining a satisfactory moisture content, and of obtaining a uniform mixture of materials.

Some exploratory test pits and test embankments in the borrow areas will be open for inspection and bidders should inspect the borrow areas and examine the test pits, and bidders are urged to sample and test materials from borrow areas prior to submitting bids.

The type of equipment used and the contractor's operations in the excavation of materials in borrow pits shall be such as will produce the required uniformity of mixture of each of the types of materials at the borrow pits.

The location and extent of all borrow pits within borrow areas shall be as directed, and the Government reserves the right to change the limits or location of borrow pits within the limits of the borrow areas in order to obtain the most suitable material, to minimize stripping, or for other reasons.

Borrow Areas A and B shall not excavated below elevation 5325 on the reservoir side of the borrow area, and a berm tying into the crest of the dam, with a minimum width of 200 feet shall not be excavated below elevation 5332.

To avoid the formation of pools in borrow pits during the excavation operations, and after the excavation operations are completed, drainage ditches from borrow pits to the nearest outlets shall be excavated by the contractor where, in the opinion of the contracting officer, such drainage ditches are necessary. The borrow areas shall be excavated so that the drainage pattern is away from the dam structures.

Final excavated surfaces of borrow pits shall be graded to slopes not steeper than 2:1. Other than as specified above, the contractor will not be required to excavate surfaces of borrow pits to any specified lines and grades, but such surfaces shall be left in a reasonably smooth and even condition and may require trimming, as directed by the contracting officer to provide a neat appearance or to provide suitable surfaces for seeding. Borrow pits shall be operated and left in a condition so as not to impair the usefulness nor mar the appearance of any part of the work or any other property of the Government, and shall be left in a condition as required in Paragraph 58. The surfaces of wasted material shall be left in a reasonably smooth and even condition.

b. Moisture and drainage.—The moisture content of the earthfill, Zone 1 material prior to and during compaction shall be in accordance with Paragraph 87. As far as practicable, the material shall be conditioned in the borrow pits before excavation. If required, moisture shall be introduced into the borrow pits for the earthfill material by irrigation, at least 45 days in advance of excavation operations. When moisture is introduced into the borrow pits for earthfill material prior to excavation, care shall be exercised to moisten the material uniformly to produce the required moisture content during compaction, avoiding both excessive runoff and accumulation of water in depressions. The contractor is cautioned to control carefully the application of water and check on the depth and amount of water penetration during application so as to avoid overirrigation.

Ponding and sprinkling tests were conducted in Borrow Areas A and B. The results of these tests are shown on Drawing No. 445 (549-D-323).

If at any location in the borrow pits for earthfill material, before or during excavation operations, there is excessive moisture, as determined by the contracting officer, steps shall be taken to reduce the moisture by selective excavation to secure the drier materials; by mixing the wet material with dry material in the excavation; by excavating and placing in temporary stockpiles material containing excessive moisture; by excavating drainage ditches; by allowing adequate additional time for curing or drying; or by any other approved means.

In any event, the contractor will be required to excavate sufficient suitable mterial in portions of Borrow Areas A and B to complete the work under these specifications, regardless of whether overly wet conditions encountered are due to ground water, precipitation, difficulty of draining, or for any other reason.

The contractor shall be entitled to no additional allowance above the unit prices bid in the schedule on account of the requirement for excavating drainage ditches; for allowing additional time for curing or drying; for stockpiling and rehandling excavated materials which have been deposited temporarily in stockpiles; delays or increased costs due to stockpiling; poor trafficability in the borrow areas on the haul roads, or the embankment; reduced efficiency of the equipment the contractor elects to use; or on account of any other operations or difficulties caused by overly wet materials. No additional allowance above the unit prices bid in the schedule will be made because of variation in the proportion between wet and dry materials which are required to be excavated in order to obtain adequate suitable material.

c. Stripping and waste.—Borrow pit sites shall be cleared as provided in Paragraph 65. Borrow pits will be designated by the contracting officer as the work progresses, and stripping operations shall be limited only to designated borrow pits. The contractor shall carefully strip the sites of designated borrow pits of boulders, topsoils, sod, loam, and other matter which is unsuited for the purposes for which the borrow pit is to be excavated. The contractor shall maintain the stripped surfaces free of vegetation until excavation operations in the borrow pit are completed and the contractor shall be entitled to no additional allowance above the unit prices bid in the schedule because of this requirement. Materials from stripping from Borrow Areas A and B which are suitable for topsoil shall be selected during stripping operations, temporarily stockpiled adjacent to borrow pits if necessary, and spread over exhausted portions of the borrow pits as directed by the contracting officer. Materials from stripping which are not suited for topsoil shall be disposed of in exhausted borrow pits, or in approved areas adjacent to borrow pits, or as provided in Paragraph 73.

If materials unsuitable, or not required, for permanent construction purposes are found in any borrow pit, such materials shall be left in place or excavated and wasted, as directed. Where excavation of such materials is directed, payment for such excavation and disposal of unsuitable or excess materials will be made at the unit price per cubic yard bid in the schedule for excavation, stripping borrow pits in Borrow Areas A and B.

d. Excavation and transportation.—The contracting officer will designate the depths of cut in all parts of the borrow pits, and the cuts shall be made to such designated depths. The earthfill materials delivered on the dam embankment shall be equivalent to a mixture of materials obtained from an approximately uniform cutting from the full height of the designated face of the borrow pit excavation.

Shallow cuts will be permitted in the borrow areas if unstratified materials with uniform moisture content are encountered.

The contractor shall transport the materials to the dam embankment location designated by the contracting officer.

The contractor shall be entitled to no additional allowance above the unit prices bid in the schedule on account of the designation by the contracting officer of the various portions of the borrow areas from which materials are to be obtained, on account of the depths of cut which are required to be made, or on account of the zone or location on embankment where materials are hauled.

Haul roads on the sides of the canyon, from Borrow Areas A and B to the dam embankment shall be located upstream in the reservoir area, in locations approved by the contracting officer.

e. Measurement and payment.—Measurement, for payment, of excavation, stripping borrow pits will be made in excavation and will include only the stripping in locations and to the depths as directed by the contracting officer. Payment for excavation, stripping borrow pits in Borrow Areas A and B will be made at the unit price per cubic yard bid therefor in the schedule, which unit price shall include the costs of selecting, stockpiling, and spreading the topsoil over exhausted portions of the borrow pits, or otherwise disposing of materials from stripping.

Measurement, for payment, of excavation in Borrow Areas A and B will be made in excavation only and to the excavation lines prescribed by the contracting officer. Payment for excavation in Borrow Areas A and B and transportation to dam embankment will be made at the unit price per cubic yard bid therefor in the schedule, which unit price shall include all costs of irrigation and unwatering of borrow pits, of conditioning the material properly, and all work other than stripping required by this paragraph. All materials from borrow pits in Borrow Areas A and B placed in dam embankment, Zones 1 and 3, and in specially compacted earthfull, Zone 1 will again be included for payment under the applicable items of the schedule for placing such earthwork.

If the contractor elects to obtain other material from Borrow Area A or B for which the cost of furnishing is included in other items of work, no payment will be for stripping or excavation of such materials obtained from borrow areas. The contractor shall keep his operations for the production of these materials separate and distinct from his other borrow area operations.

72. BORROW AREA C AND C EXTENSION

a. General.—All materials required for construction of dam embankment Zones 2 and 4, which are not available from excavations required for permanent construction under these specifications, shall be obtained from materials excavated from Borrow Area C and C extension. All materials required for bedding for riprap and for previous backfill shall be selected from materials excavated from Borrow Area C and C extension and stockpiled as provided below. Materials required for gravel surfacing in accordance with Paragraph 318 may be obtained from materials excavated from Borrow Area C and C extension and stockpiled as provided below. The location of the borrow area is shown on Drawings No. 442 (549-D-165), 443 (549-D-166), and 443A (549-D-503).

Borrow Area C and C extension will be subject to flooding during periods of high flow in the river and during diversion through the outlet works. The required materials from Borrow Area C and C extension shall be excavated and stockpiled in the location shown on Drawing No. 442 (549-D-165).

Prior to initiating excavation in Borrow Area C or C extension, the contractor shall submit to the Construction Engineer, for approval, an excavation and disposal plan for the borrow area, which shall include plans for control of turbidity as required by Paragraph 60. The plan shall reflect the requirements of this paragraph and shall relate these requirements to the construction program, and to the anticipated riverflows and the diversion discharge curves. Drawings included with the plan shall indicate haul road locations, river crossings, temporary dike locations and dimensions, methods of diverting the river through the operations, and turbidity control structures.

The flow of the river shall be diverted away from excavation operations whenever possible, so that underwater excavation will be performed in still water.

The type of equipment used and the contractor's operations in the excavation of materials in borrow pits shall be such as to permit selection of the proper mixture of each of the types of the materials at the borrow pits, shall be capable of underwater excavation, and shall be subject to the approval of the contracting officer.

The location and extent of all borrow pits within the borrow area shall be as directed, and the contracting officer reserves the right to change the limits or location of borrow pits within the limits of the borrow area in order to obtain the most suitable material, to minimize stripping, or for other reasons.

b. Explorations.—Logs of additional explorations in the extended portion of Borrow Area C will be furnished by additions to the "Records of Subsurface Investigations," referred to in Paragraph 54.

Explorations in the borrow area indicate that the materials are variable in nature and texture and contain variable amounts of moisture and oversize materials. Approximate percentages of oversize material encountered in the explorations within the borrow area are shown on the logs. The absence of percentages of oversize on any log of explorations within the area does not, however, imply that oversize materials will not be encountered in the vicinity of such explorations.

Ground-water level encountered in the explorations, as shown on the logs, is for the indicated dates. The absence of a ground-water level or moisture content on any log of explorations within the area does not, however, imply the ground-water or variable moisture content will not be encountered in the vicinity of such explorations.

Bidders are cautioned that wide variation from the nature, texture, moisture content, and the percentage of oversize material as indicated by the explorations is to be anticipated. Bidders and the contractor must assume all responsibility for deductions and conclusions concerning the nature, moisture content and texture of material, the percentages of oversize materials, the total yield of suitable materials, the difficulties of making excavations, of removing the oversize materials, and of selecting the proper mixture of materials for the various items of work.

Some exploratory test pits in the borrow area will be open for inspection and bidders should inspect the borrow area and examine the test pits, and bidders are urged to sample and test material from the borrow area prior to submitting bids.

c. Stripping and waste.—Borrow pit sites shall be cleared as provided in Paragraph 65. Borrow pits will be designated by the contracting officer as the work progresses, and stripping operations shall be limited only to designated borrow pits. The contractor shall carefully strip the sites of designated borrow pits of boulders, topsoil, sod, loam, and other matter which is unsuited for the purposes for which the borrow pit is to be excavated, as directed by the contracting officer. The stripping depth shall not exceed 12 inches for the removal of roots. Any roots larger than one-fourth inch in diameter excavated with otherwise suitable material shall be removed prior to compaction on the embankment.

The contractor shall maintain the stripped surfaces free of vegetation until excavation operations in the borrow pit are completed and the contractor shall be entitled to no additional allowance above the unit prices bid in the schedule because of this requirement. Materials from stripping shall be disposed of in dikes as provided in Paragraph 60, in exhausted borrow pits, or in approved areas adjacent to borrow pits, as required in Paragraph 73.

If materials unsuitable or not required for permanent construction purposes are found in any borrow pit, such materials shall be left in place or excavated and wasted, as directed. Where excavation of such materials is directed, payment for such excavation and disposal of unsuitable or excess materials will be made at the unit price per cubic yard bid in the schedule for excavation, stripping borrow pits in Borrow Area C and C extension.

d. Excavation, transportation, and stockpiling.—The contracting officer will designate the depths of cut in all parts of the borrow pits, and the cuts shall be made to such designated depths.

A berm not less than 150 feet wide shall be left between the upstream toe of the dam embankment and the edge of the borrow pit, with a slope not steeper than 4:1 to the bottom of the borrow pit.

In other areas the contractor will not be required to excavate surfaces of borrow pits to any specified lines and grades, but such surfaces and surfaces of wasted material shall be left in a reasonably smooth and even condition.

Excavation in Borrow Area C and C extension shall be scheduled taking into account the flow of the Teton River and the diversion capacities of the outlet works so that work is not delayed.

Where suitable material is available, excavation in Borrow Area C and C extension shall be at least 25 feet deep in order to minimize stripping.

It is anticipated that the most of the material excavated from Borrow Area C and C extension will be from below the water table. The contractor shall be entitled to no additional allowance above the prices bid in the schedule on account of poor trafficability in the borrow area or on the haul roads, reduced efficiency of the equipment the contractor elects to use, or on account of any other operations or difficulties caused by overly wet materials.

The contractor shall transport the materials to the stockpile.

Haul roads from Borrow Area C and C extension to the stockpile shall be in locations approved by the contracting officer. Haul roads which will not be inundated by the reservoir shall be obliterated and left in a condition which will facilitate natural revegetation prior to completion of the contract.

Materials for the various embankment items shall be selected and placed in separate areas of the stockpile as directed. The material shall be placed in the stockpile to dimensions as directed by the contracting officer.

73. STOCKPILING AND DISPOSING OF EXCAVATED MATERIAL

So far as practicable, as determined by the contracting officer, all suitable materials from excavation required under these specifications shall be used in the permanent construction. Excavated materials that are unsuitable for, or are in excess of permanent construction requirements shall be wasted.

Materials excavated from Borrow Area C and C extension shall be stockpiled in accordance with Paragraph 72.

Suitable materials excavated for permanent construction and which cannot be placed in the designated final locations directly from the excavation shall be placed in stockpiles and shall later be placed in the embankment. The locations of these stockpiles shall be subject to approval. Care shall be taken not to cover or contaminate materials which are required for use in the permanent construction. Materials in these stockpiles not used in construction shall be disposed of in approved disposal areas prior to completion of work under these specifications.

The disposal of all excavated materials that are to be wasted shall be subject to the approval of the contracting officer, but the contractor will not be required to haul materials to be wasted more than 3,000 feet along the most practicable routes to the designated disposal areas. Areas designated for disposal of waste material from excavation are shown on Drawing No. 4 (599-D-8). Additional disposal areas will be designated by the contracting officer as needed.

Waste piles shall be located where they will not interfere harmfully with the natural flow of the stream and drainage channels, with construction operations in the borrow areas, with the operation of the reservoir, or with the flow of water to or from the spillway, outlet works, or powerplant, and where they will neither detract from the appearance of the completed project nor interfere with the accessibility of the structures for operation. Waste piles shall be leveled and trimmed to reasonably regular lines.

Except as otherwise provided in Paragraph 72, no separate payment will be made for stockpiling and disposing of excavated materials, and all costs of transporting excavated materials from excavations to disposal areas or to points of final use, including stockpiling and rehandling, if required, and of disposing of all excavated materials that are wasted, as provided in this paragraph, shall be as provided in Paragraphs 67, 71, and 72.

EMBANKMENT

86. EMBANKMENT CONSTRUCTION, GENERAL

a. General.—For the purpose of these specifications, the term "dam embankment" includes all portions of the dam embankment as follows:

(1) The earthfill, Zone 1 portions designated on the drawings by the figure 1 encircled, including specially compacted earthfill.

(2) The sand, gravel, and cobble fill, Zone 2 portions designated on the drawings by the figure 2 encircled.

(3) The miscellaneous fill, Zone 3 portions designated on the drawings by the figure 3 encircled.

(4) The silt, sand, gravel, and cobble fill, Zone 4 portions designated on the drawings by the figure 4 encircled.

(5) The rockfill, Zone 5 portions designated on the Drawings by the figure 5 encircled.

(6) The riprap on the upstream slope of Zone 2 dam embankment.

The surfacing on the crest of the dam shall be gravel surfacing in accordance with Paragraph 318.

Other items of embankment, which are not necessarily a part of the dam embankment, include bedding for riprap, riprap, pervious backfill and that portion of Zone 4 to the left of the tailrace channel.

Earthwork for the Enterprise-East Teton Feeder Pipeline, the pump discharge line, and appurtenant structures shall be in accordance with Paragraphs 286 through 297.

Roadway embankment shall be in accordance with Paragraph 314.

The embankment shall be constructed in accordance with this paragraph and Paragraphs 87 through 95. The completed embankment shall be to the lines and grades shown on the drawings: Provided, That the thickness of the downstream Zone 5 embankment and of the adjoining Zone 3 embankment shall vary as directed by the contracting officer so as to accommodate the volume of rockfill available; Provided further, That the upper portion of the Zone 1 and of the adjoining Zone 2 embankment shall vary as directed by the contracting officer so as to accommodate the volume of Zone 2 material in the stockpile.

b. Foundation preparation.—No material shall be placed in any portion of the dam embankment until the foundation for each section has been unwatered, stripped, suitably prepared, and has been approved by the contracting officer. Stripping shall be in accordance with Paragraph 68.

All cavities, depressions, and irregularities, either existing or resulting from removal of rock fragments found within the area to be covered by embankment, and which extend below or beyond the established lines of excavation for dam embankment foundations, shall be filled with embankment materials and compacted as specified for the overlying embankment.

c. Placing embankment materials.—The suitability of each part of the foundation for placing embankment materials thereon, and of all materials for use in embankment construction will be determined by the contracting officer. No embankment material shall be placed in the embankment when either the material or the foundation or embankment on which it would be placed is frozen.

No brush, roots over one-fourth inch in diameter, sod, or other perishable or unsuitable materials shall be placed in the completed embankment.

Each load of the material placed in the embankment, whether from excavation for other parts of the work or from borrow pits, shall be placed in the location designated by the contracting officer and the contractor shall be entitled to no additional allowance above the unit prices bid in the schedule on account of this requirement.

In any separate portion of dam embankment being constructed, each layer of each zone shall be constructed continuously and approximately horizontal for the width and length of such portion at the elevation of the layer.

The contractor shall maintain the embankment in an approved manner, including maintaining surfaces free of weeds or other vegetation, until final completion and acceptance of all the work under the contract.

The contractor will be permitted to construct separate portions of the embankment below original ground surface, subject to the approval of the contracting officer.

Above original ground surface, construction of the dam embankment shall be subject to the following conditions:

(1) Longitudinal bonding surfaces (surfaces parallel to the centerline crest of embankment) will not be permitted in Zone 1. Within other zones of dam embankment, longitudinal surfaces between previously constructed embankment and embankment to be constructed shall be subject to approval and shall be not steeper than 1 and 1/2:1.

(2) A temporary gap through the dam embankment, for diversion purposes as described in Paragraph 63 will be permitted: Provided, That the slopes of transverse bonding surfaces (surfaces normal to the centerline crest of embankment) in Zone 1 shall be not steeper than 4:1, and in other zones of dam embankment, transverse bonding surfaces between previously completed portions of embankment and embankment to be constructed shall be not steeper than 2:1. No other transverse bonding surfaces will be permitted in the dam embankment.

During construction of earthfill, Zone 1, embankment in the openings at the gap, the contractor shall construct a keyway trench in each 4:1 transverse bonding surface in the previously placed earthfill, Zone 1. The keyway trenches shall be excavated in the bonding surfaces to a minimum depth of 5 feet, shall have 4:1 side slopes, and shall have a minimum bottom width of 20 feet. The centerline of the trenches shall be located approximately midway between the upstream and downstream slopes of the Zone 1. The treanches shall be refilled with earthfill, Zone 1, material subject to the provisions of Paragraph 87.

(3) At any cross section, the elevation of the Zone 1 portion of the dam embankment shall not exceed the elevation of the immediately adjacent Zone 2 by more than 5 feet.

Placing shall be performed in a manner to prevent damage to structures.

d. Measurement and payment.—Measurement, for payment, of the various items of embankment construction will be made of the material in place in the completed embankment to the lines, grades, slopes, and thicknesses shown on the drawings including crest camber, or established by the contracting officer. The cross sections obtained by surveys made after completion of excavation for dam embankment foundation and the lines and dimensions shown on the drawings and as directed will be used in computing the quantity of embankment placed. No allowance will be made in measurement for payment for settlement, shrinkage, and consolidation of the foundation or of the material in the embankment. In measuring embankment for payment, the volume of structures, of specially compacted earthfill, and of other work for which items for payment are provided in the schedule, will be deducted.

The dividing surfaces for measurement, for payment, between embankment items and roadway embankment for the left abutment county road connection, will be a vertical plane normal to the centerline of the roadway, at roadway Station 1+50.00, and between Zone 4 embankment and roadway embankment for the access road will be a vertical plane normal to the centerline of the road at roadway Station 19+00.

Payment under all items of embankment construction shall include the costs of preparing the embankment foundations; of placing; of supplementary wetting on the fill, if necessary, and any additional work required on the embankment to accomplish uniform moisture application; of compacting where compaction is required, except for compacting pervious backfill; of preparing bonding surfaces; and all other operations required to secure adequate bond between embankment in place and embankment to be placed. Payment for compacted pervious backfill will be made in accordance with Paragraph 95.

Payment for the embankment items will be in addition to payment made for excavation and transportation of the materials: Provided, That materials obtained from the stockpile of materials from Borrow Area C and C extension will not be measured in excavation, and the costs of excavating from stockpile and transportation to points of final use of these materials shall be included in the appropriate items of embankment, as provided in the embankment paragraphs: Provided further, That materials obtained directly from Borrow Area C and C extension, without stockpiling, will be measured only in place in the embankment and the costs of excavating and transporting to points of final use of these materials shall be included in the appropriate items of the schedule for which payment includes procuring.

It may be feasible to transport some of the materials which are excavated for other parts of the work and which are suitable for embankment construction directly to the embankments at the time of making the excavations, but the contractor shall be entitled to no additional compensation above the unit prices bid in the schedule by reason of it being necessary, or required by the contracting officer, that such excavated materials be deposited temporarily in stockpiles and rehandled prior to being placed in the embankment.

No measurement or payment will be made for excavating keyway trenches in embankment or for refilling the trenches and the cost of preparing bonding surfaces, including excavating keyway trenches and refilling such trenches in transverse bonding slopes and all other operations required to secure adequate bond between embankment in place and embankment to be placed shall be included in unit prices bid for items of constructing embankments.

87. EARTHFILL IN DAM EMBANKMENT, ZONE 1

a. General.—The earthfill, Zone 1 portion of the dam embankment shall be constructed in accordance with this paragraph and Paragraph 86.

b. Materials.—Zone 1 of the earthfill portion of the dam embankment shall consist of a mixture of clay, silt, sand, gravel, and cobbles, available from excavations required for permanent construction under these specifications and from borrow pits in Borrow Areas A and B.

The contractor's operations in the excavation of the materials for the earthfill shall be in accordance with Paragraphs 67 and 71.

Cobbles and rock fragments having maximum dimensions of more than 5 inches shall not be placed in the earthfill. Should cobbles and rock fragments of such size be found in otherwise approved earthfill materials, they shall be removed by the contractor either at the site of excavation or after being transported to the earthfill but before the materials in the earthfill are rolled and compacted. Such cobbles and rock fragments shall be placed in the Zone 3 portion of the dam embankment or wasted as approved by the contracting officer.

Fragments of caliche and hard calcareous materials larger than 5 inches in maximum dimensions may be placed on the embankment, provided that the fragments are broken down to less than 5 inches during compaction, or the fragments may be removed and placed in Zone 3.

c. Preparation of foundations.—The foundation for the earthfill, Zone 1, except surfaces of formation materials, shall be prepared by leveling, moistening, and rolling so that the surface materials of the foundation will be as compact and will provide as satisfactory a bonding surface with the first layer of the earthfill as specified for the subsequent layers of the earthfill.

Immediately prior to placing the first layer of earthfill, all surfaces upon or against which the earthfill portions of the dam embankment are to be placed shall be cleaned of all loose and objectionable materials in an approved manner by handwork, barring, picking, brooming, sluicing, or by other effective means. Such surfaces shall have all water removed from depressions and shall be properly moistened and sufficiently clean to obtain a suitable bond with the earthfill.

d. Moisture and density control.—

(1) General.—Each layer of the material on the embankment shall be compacted by 12 passes of the tamping roller as provided in Subparagraph g. below, which shall be the minimum compacting effort to be performed by the contractor. During compaction, the placement moisture content and dry density of the earthfill shall be maintained within the control limits specified below.

To determine that the moisture content and dry density requirements of the compacted earthfill are being met, field and laboratory tests will be made at frequent intervals on samples taken at embankment locations determined by the contracting officer. Field and laboratory tests will be made by the contracting officer in accordance with Designations E-11, E-24, and E-25 of the Bureau of Reclamation Earth Manual. The results of all completed earthwork tests will be available to the contractor at the Government laboratory.

Materials not meeting the specified moisture content and dry density requirements, as determined by the tests, shall be reworked until approved results are obtained. Reworking may include removal, rehandling, reconditioning, rerolling or combinations of these procedures. The contractor shall be entitled to no additional allowance above the prices bid in the schedule by reason of any work required to achieve the placement moisture content and density specified in this paragraph.

(2) Moisture control.—The standard optimum moisture content is defined as, "That moisture content which will result in a maximum dry unit weight of the soil when subjected to the Bureau of Reclamation Proctor Compaction Test." The maximum dry weight, in pounds per cubic foot, obtained by the above procedure is the Proctor maximum dry density. The Bureau of Reclamation Proctor Compaction Test (Designation E-11 of Earth Manual) is the same as ASTM Designation: D 698, Method A, except that a 1/20-cubic foot compaction mold is used and the rammer is dropped from a height of 18 inches.

The moisture content of the earthfill material prior to and during compaction shall be distributed uniformly throughout each layer of the material. The allowable ranges of placement moisture content are based on design considerations. The moisture control shall be such that the moisture content of compacted earthfill, as determined by tests performed by the contracting officer, shall be within the following limits:

(a) Material represented by the samples tested having a placement moisture content more than 3.5 percent dry of the standard optimum condition, or more than 1.0 percent wet of the standard optimum condition will be rejected and shall be removed or reworked until the moisture content is between these limits.

(b) Within the above limits, and based on a continuous record of tests made by the Government on previously placed and accepted material, the uniformity of placement moisture content shall be such that:

(aa) No more than 20 percent of the samples of accepted embankment material will be drier than 3.0 percent dry of the standard optimum moisture content, and no more than 20 percent will be wetter than 0.5 percent wet of the standard optimum moisture content.

(bb) The average moisture content of all accepted embankment material shall be between 0.5 and 1.5 percent dry of the standard optimum moisture content.

The Government will inform the contractor and when the placement moisture content is near or exceeds the limits of uniformity specified above, and the contractor shall immediately make adjustments in procedures as necessary to maintain the placement moisture content within the specified limits.

As far as practicable, the material shall be brought to the proper moisture content in the borrow pit before excavation, as provided in Paragraph 71. Supplementary water, if required, shall be added to the material by sprinkling on the earthfill and each layer of earthfill shall be conditioned so that the moisture is uniform throughout the layer.

(3) Density control.—Density control of compacted earthfill shall be such that the dry density of the compacted material, as determined by tests performed by the contracting officer, shall conform to the following limits:

(a) Material represented by samples having a dry density less than 94 percent of its Proctor maximum dry density will be rejected. Such rejected material shall be rerolled until a dry density equal to or greater than 94 percent of its Proctor maximum dry density is obtained.

(b) Within the above limit and based on a continuous record of tests made by the Government on previously placed and accepted embankment, the uniformity of dry density shall be such that:

(aa) No more than 20 percent of the material represented by the samples tested shall be at dry densities less than 95 percent of Proctor maximum dry density.

(bb) The average dry density of all accepted embankment material shall be not less than 98 percent of the average Proctor maximum dry density.

The Government will inform the contractor when the dry density is near or exceeds the limits of uniformity specified above, and the contractor shall immediately make adjustments in procedures as necessary to maintain the dry density within the specified limits.

e. Placing.—The distribution and gradation of the materials throughout the earthfill shall be such that the fills will be free from lenses, pockets, streaks, or layers of material differing substantially in texture, gradation, or moisture from the surrounding material. The combined excavation and placing operations shall be such that the materials when compacted in the earthfill will be blended sufficiently to secure the best practicable degree of compaction and stability. Successive loads of material shall be dumped on the earthfill so as to produce the best practicable distribution of the material, subject to the approval of the contracting officer, and for this purpose the contracting officer may designate the locations in the earthfill where the individual loads shall be deposited, to the end that the most impervious materials shall be placed in the central portion of the earthfill and the more pervious materials shall be placed so that the permeability of the fill will be gradually increased toward the upstream and downstream slopes of the earthfill.

The material shall be placed in the earthfill in continuous, approximately horizontal layers not more than 6 inches in thickness after being compacted. If, in the opinion of the contracting officer, the surface of the prepared foundation or the compacted surface of any layer of earthfill is too dry or smooth to bond properly with the layer of material to be placed thereon, it shall be moistened and/or worked with harrow, scarifier, or other suitable equipment, in an approved manner to a sufficient depth to provide a satisfactory bonding surface before the next succeeding layer of earthfill material is placed. If, in the opinion of the contracting officer, the compacted surface of any layer of the earthfill in place is too wet for proper compaction of the layer of earthfill material to be placed thereon, it shall be removed; allowed to dry; or be worked with harrow, scarifier, or other suitable equipment to reduce the moisture content to the required amount; and then it shall be recompacted before the next succeeding layer of earthfill material is placed.

The earthfill on each side of the structures and at the abutments shall be kept at approximately the same level as the placing of the earthfill progresses, and the structures shall be protected against displacement or other damage.

f. Rollers.—Tamping rollers shall be used for compacting the earthfill. The rollers shall meet the following requirements:

(1) Roller drums.—Tamping rollers shall consist of two or more roller drums mounted side by side in a suitable frame. Each drum of a roller shall have an outside diameter of not less than 5 feet and shall be not less than 5 feet nor more than 6 feet in length. The space between two adjacent drums, when on a level surface, shall be not less than 12 inches nor more than 15 inches. Each drum shall be free to pivot about an axis parallel to the direction of travel. Each drum ballasted with fluid shall be equipped with at least one pressure-relief valve and with at least one safety head as shown on Drawing No. 375 (40-D-6001) or with approved equivalent types. The safety head shall be equal to union type safety heads as manufactured by Black, Sivals, and Bryson, Inc., Kansas City, Missouri, with rupture disks suitable for between 50- and 75-psi rupturing pressure.

The pressure-relief valve shown is a manually operated valve and shall be opened periodically. Personnel responsible for opening pressure-relief valves shall be instructed to ascertain that valve openings are free from plugging to assure that any pressure developed in roller drums is released at each inspection.

(2) Tamping feet.—At least one tamping foot shall be provided for each 100 square inches of drum surface. The space measured on the surface of the drum, between the centers of any two adjacent tamping feet, shall be not less than 9 inches. The length of each tamping foot from the outside surface of the drum shall be not more than 11 inches and shall be maintained at not less than 9 inches. The cross-sectional area of each tamping foot shall be not more than 10 square inches at a plane normal to the axis of the shank 6 inches from the drum surface, and shall be maintained at not less than 7 square inches nor more than 10 square inches at a plane normal to the axis of the shank 8 inches from the drum surface.

(3) Roller weight.—The weight of a roller when fully loaded shall be not less than 4,000 pounds per foot of length of drum.

Drawings No. 374 (40-D-6000) and 375 (40-D-6001) are detailed drawings of a tamping roller meeting the above requirements.

The loading used in the roller drums and operation of the rollers shall be as required to obtain the specified compaction. If more than one roller is used on any one layer of fill, all rollers so used shall be of the same type and essentially of the same dimensions. Rollers operated in tandem sets shall be towed in a manner such that the prints of the tamping feet produced by the tandem units do not overlap. The design and operation of the tamping roller shall be subject to the approval of the contracting officer who shall have the right at any time during the prosecution of the work to direct such repairs to the tamping feet, minor alterations in the roller, and variations in the weight as may be found necessary to secure optimum compaction of the earthfill materials. Rollers shall be drawn by crawler-type or rubber-tired tractors. The use of rubber-tired tractors shall be discontinued if the tires leave ruts that prevent uniform compaction by the tamping roller. Tractors used for pulling rollers shall have sufficient power to pull the rollers satisfactorily when drums are fully loaded with sand and water.

At the option of the contractor, self-propelled tamping rollers conforming with the above requirements may be used in lieu of tractor-drawn tamping rollers. For self-propelled rollers, in which steering is accomplished through the use of rubber-tired wheels, the tire pressure shall not exceed 40 psi. During the operation of rolling, the spaces between the tamping feet shall be maintained clear of materials which would impair the effectiveness of the tamping rollers.

g. Compaction.—When each layer of material has been conditioned to have the specified moisture, as provided in Subparagraph d., it shall be compacted by passing the tamping roller over it 12 times, and when compacted the density shall be essentially uniform throughout the layer. If the uncompacted earthfill material is too wet for proper compaction, the earthfill material shall be worked with harrow, scarifier, or other suitable equipment to reduce the moisture content to the amount specified; shall be allowed to dry until such time as the material has dried so that it contains only the specified moisture content; or the material shall be removed from the embankment. Compacted earth material having a moisture content or dry density that do not meet the criteria specified in Subparagraph d. above shall be reworked and rerolled as directed by the contracting officer, to obtain the specified moisture content and dry density of embankment in place.

h. Measurement and payment.—Measurement, for payment, of earthfill in dam embankment, Zone 1, will be made of all earthfill compacted in place by rollers specified in Subparagraph f. and as provided in Paragraph 86.

Payment for earthfill in dam embankment, Zone 1, will be made at the unit price per cubic yard bid therefor in the schedule, which price shall include all costs of work required under this paragraph and as provided in Paragraph 86.

Where portions of the earthfill in dam embankment, Zone 1, require special compaction, payment therefor will be made as provided in Paragraph 88.

88. SPECIALLY COMPACTED EARTHFILL, ZONE 1

Where compaction of earthfill, material by means of roller specified for use on the dam embankment is impracticable or undesirable, the earthfill shall be specially compacted as specified herein at the following locations:

(1) Portions of the earthfill in dam embankment, Zone 1, where designated by the contracting officer, at steep and irregular abutments and on rough and irregular embankment foundations.

(2) Earthfill material placed to refill additional excavation, ordered in writing by the contracting officer, in common excavation for structure foundations.

(3) Portions of the earthfill, in dam embankment adjacent to structures and structure foundations shown on the drawings as specially compacted earthfill.

(4) Earthfill material at locations outside the limits of the dam embankment as shown on the drawings or where designated by the contracting officer.

Material used in specially compacted earthfill shall conform to materials required for earthfill in dam embankment: Provided, that gravel, cobbles, and rock fragments having maximum dimensions of more than 1 inch shall not be placed in the specially compacted earthfill. The material shall be obtained from excavation required for permanent construction under those specifications and from excavation in Borrow Areas A and B.

All specially compacted earthfill material shall be placed in accordance with the applicable provisions of Subparagraph 88e: Provided, That earthfill material to be specially compacted may require placement in layers thinner than those specified for roller compaction of earthfill material to obtain the desired compaction with the equipment used.

Where the foundation or compacted surface of any layer is too smooth to bond properly with the succeeding layer, it shall be scarified or otherwise roughened to provide a satisfactory bonding surface before the next layer of earthfill material is placed.

When each layer or material has been conditioned to have the required moisture content, it shall be compacted by special rollers, mechanical tampers, or by other approved methods. All equipment and methods used shall be subject to approval. The moisture and density shall be equivalent to that obtained in the earthfill placed in the dam embankment in accordance with Subparagraph 88d.

Measurement, for payment, of specially compacted earthfill, Zone 1, will be made of the material specially compacted, as provided in this paragraph, and as provided in Paragraph 86. Under (1) above, measurement, for payment, of specially compacted earthfill at steep and irregular dam abutments will be limited to a thickness of 2 feet measured horizontally from the average contacts where practicable, or as otherwise determined by the contracting officer, and measurement, for payment, of specially compacted earthfill on rough and irregular embankment foundations will be made in the most practicable manner as determined by the contracting officer.

Payment for specially compacted earthfill, Zone 1, will be made at the unit price per cubic yard bid therefor in the schedule, which unit price shall include all costs as provided in Paragraph 86.

89. SAND, GRAVEL, AND COBBLE FILL IN DAM EMBANKMENT ZONE 2

a. General.—The sand, gravel, and cobble fill in dam embankment, Zone 2, shall be constructed in accordance with this paragraph and Paragraph 86.

b. Materials.—The materials shall consist of a reasonably well-graded pervious mixture of sand, gravel, and cobbles less than 12 inches in maximum dimension selected from excavations required for permanent construction under these specifications and from the stockpile of materials from borrow pits in Borrow Area C and C extension in accordance with Paragraph 72.

Boulders and rock fragments larger than 12 inches in maximum dimensions, delivered to the embankment with otherwise approved Zone 2 materials, shall be removed before the Zone 2 material is compacted. Such oversize boulders and rock fragments shall be placed in Zone 5, or on the outer slopes of the upstream embankment, or shall be wasted as directed.

c. Preparation of foundations.—The foundation for the Zone 2 shall be prepared by leveling, moistening, and compacting so that the surface materials will be as compact as specified for subsequent layers of the Zone 2.

d. Moisture control.—The moisture content of the sand, gravel, and cobble fill material, prior to and during compaction, shall be distributed uniformly throughout each layer of the material. The moisture content shall be sufficient to attain the maximum relative density of the material in place, when compacted by the specified compaction procedure as provided in Subparagraph e. In general, the material shall be thoroughly wetted to obtain the maximum practicable compaction but shall not contain moisture to the extent which will interfere with trafficability of the contractor's hauling, placing, or compacting equipment.

Moisture, as required, may be applied by sprinkling on the fill, or by other approved methods.

e. Placing and compacting.—The contractor's operations shall be such and he shall handle and place the material in such a manner as to prevent segregation.

The materials obtained from excavations for permanent construction shall be placed separately from the materials obtained from Borrow Area C and C extension, to facilitate measurement for payment.

The sand, gravel, and cobble material shall be placed in the dam embankment in continuous, approximately horizontal layers not more than 12 inches in compacted thickness. When each layer of material has been conditioned to have the required moisture, as provided in Subparagraph d., it shall be compacted by four passes of the treads of the crawler-type tractor weighing approximately 40,000 pounds, or as provided below. One pass of the treads is defined as the required number of successive tractor trips which, by means of sufficient overlap, will insure complete coverage of the entire surface of the layer by the tractor treads. Second and subsequent passes of the treads shall not be made until each pass, as defined above, is completed.

If the contractor elects to use methods of compaction other than the one specified above, the weight of the compactor, the number of passes, inflation pressures of tires (if rubber-tired compactors are used), and thickness of lift not to exceed 12 inches compacted, shall be such as to result in sand, gravel, and cobble fill in dam embankment compacted within the following limits:

(1) Material represented by samples having a relative density less than 65 percent will be rejected. Such rejected material shall be recompacted until a relative density equal to or greater than 65 percent is obtained.

(2) Within the above limit and based on a continuous record of tests made by the contracting officer on previously placed and accepted embankment, the uniformity of relative density shall be such that no more than 20 percent of the material represented by the samples tested shall be at relative densities less than 70 percent.

The contracting officer will inform the contractor when the relative density of the compacted material is close to or outside the limits specified above and the contractor shall immediately make adjustments in procedures as necessary to maintain the relative density of the compacted embankment within the specified limits.

The relative density of the compacted sand, gravel, and cobble fill material will be determined by the contracting officer for the full depth of each compacted layer in accordance with Part B of Designation E-12 of the Bureau of Reclamation's Earth Manual. Zone 2 material surrounding the shaft of the river outlet works and the auxiliary outlet works near the surface shall be consolidated by other approved methods which give equivalent relative densities.

f. Measurement and payment.—Measurement and payment, of sand, gravel, and cobble fill in dam embankment, Zone 2, will be made as provided in this paragraph and in Paragraph 86.

Embankment constructed of materials from excavations for permanent construction required under these specifications and placed in sand, gravel, and cobble fill in dam embankment, Zone 2, will be measured separately from embankment constructed of materials obtained from the stockpile of materials from Borrow Area C and C extension and placed in the dam embankment, Zone 2.

Measurement, for payment, of placing sand, gravel, and cobble fill in dam embankment, Zone 2, will be made of the embankment constructed of materials from excavations for permanent construction required under these specifications. Payment for placing sand, gravel, and cobble fill in dam embankment, Zone 2, will be made at the unit price per cubic yard bid therefor in the schedule which unit price shall include all costs as provided in Paragraph 86.

Measurement, for payment, of procuring and placing sand, gravel, and cobble fill in dam embankment, Zone 2, will be made of the embankment constructed of materials obtained from the stockpile of materials from Borrow Area C and C extension and of materials obtained directly from Borrow Area C and C extension. Payment for procuring and placing sand, gravel, and cobble fill in dam embankment, Zone 2, will be made at the unit price per cubic yard bid therefor in the schedule, which unit price shall include all costs as provided in Paragraph 86.

90. MISCELLANEOUS FILL IN DAM EMBANKMENT, ZONE 3

a. General.—The miscellaneous fill, Zone 3 portion of the dam embankment shall be in accordance with this paragraph and Paragraph 86.

b. Materials.—The miscellaneous fill, Zone 3 portion of the dam embankment shall consist of miscellaneous mixtures of clay, silt, sand, gravel, cobbles, rock fragments, or fragments of caliche and calcareous materials to 12 inches in maximum dimensions. The materials shall be obtained from excavations required for permanent construction under these specifications and from Borrow Areas A and B.

Boulders and rock fragments larger than 12 inches in maximum dimensions shall be removed from otherwise approved miscellaneous fill material, either at the site of excavation or after the material has been placed on the embankment, but before the Zone 3 material is compacted. Such oversize boulders and rock fragements shall be placed in Zone 5 or wasted as directed.

Fragments of caliche and hard calcareous material larger than 12 inches in maximum dimensions may be placed on the embankment, provided that the fragments are broken down to less than 12 inches during compaction, or the fragments may be removed and wasted in approved disposal areas.

c. Moisture control.—Prior to and during compaction, the material in each layer of the Zone 3 fill material shall have the most practicable moisture content required for compaction purposes as determined by the contracting offficer. Additional moisture as required may be applied by sprinkling on the dam embankment.

d. Placing and compacting.—The material shall be placed in the miscellaneous fill in continuous and approximately horizontal layers, not more than 12 inches in thickness after being compacted as herein specified. The combined excavation and placing operations shall be such that the materials, when compacted in the miscellaneous fill, will be sufficiently blended to secure the best practicable degree of compaction and stability.

When each layer of the material has been conditioned to have the proper moisture content, it shall be compacted by six trips of a 50-ton pneumatic-tired roller over each 15-foot horizontal width of the layer, as herein provided. Each trip of the roller shall be offset from the path of the previous trip so that the total compactive effort shall be distributed evenly over the entire horizontal layer of Zone 3 embankment.

The pneumatic-tired roller used for compaction shall have a maximum total capacity of 50 tons and shall have a minimum of four wheels equipped with pneumatic tires. The tires shall be of such size and ply as can be maintained at tire pressures between 80 and 100 psi for a 25,000-pound wheel load during roller operations. The roller wheels shall be located abreast, and be so designed that each wheel will carry approximately equal loads in traversing uneven ground. The spacing of the wheels shall be such that the distance between the nearest edges of adjacent tires will not be greater than 50 percent of the tire width of a single tire at the operating pressure for a 25,000-pound wheel load. The roller shall have a rigid steel frame provided with a body suitable for balanced loading such that the load per wheel may be maintained at 25,000 pounds.

Tractors used for pulling pneumatic-tired rollers shall have sufficient power to pull the fully loaded roller satisfactorily under normal conditions of compaction.

e. Measurement and payment.—Measurement, for payment, of miscellaneous fill in dam embankment, Zone 3, will be made as provided in Paragraph 86.

Payment for miscellaneous fill in dam embankment, Zone 3, will be made at the unit price per cubic yard bid therefor in the schedule, which unit price shall include all costs as provided in Paragraph 86.

91. SILT, SAND, GRAVEL, AND COBBLE FILL IN DAM EMBANKMENT, ZONE 4

a. General.—The silt, sand, gravel, and cobble fill, Zone 4 portion of the dam embankment shall be in accordance with this paragraph and Paragraph 86.

Zone 4 material shall also be placed between the tailrace channel and the left abutment, between access road Stations 2+30 and 19+00 to bring this area up to the design grade.

b. Materials.—The Zone 4 portion of the embankment shall consist of a mixture of silt, sand, gravel, and cobbles selected from excavation required for permanent construction under these specifications and from the stockpile of materials from borrow pits in Borrow Area C and C extension. Boulders and rock fragments larger than 12 inches in maximum dimensions, delivered to the embankment with otherwise approved Zone 4 materials, shall be removed and placed on the outer slope of the Zone 4 embankment before the Zone 4 is compacted.

c. Preparation of foundations.—The foundation for the Zone 4 shall be prepared by leveling, moistening, and compacting so that the surface materials will be as compact as specified for subsequent layers of the Zone 4.

d. Moisture control.—The moisture in the silt, sand, gravel, and cobble fill material, prior to and during compaction, shall be distributed uniformly throughout each layer of the material. The placement moisture content shall be the optimum amount required, as determined by the contracting officer, to obtain the maximum dry unit weight of the material in place, when compacted by the specified compaction procedure. Additional moisture, as required, may be applied by sprinkling on the fill.

e. Placing and compacting.—The materials obtained from excavations for permanent construction shall be placed separately from the materials obtained from the stockpile of materials from Borrow Area C and C extension to facilitate measurement for payment.

The silt, sand, gravel, and cobble fill material shall be placed in the Zone 4 portion of the embankment in continuous, approximately horizontal layers not more than 12 inches in compacted thickness. When each layer of material has been conditioned to have the required moisture, as provided in Subparagraph d., it shall be compacted by six trips of a 50-ton, pneumatic-tired roller over each 15-foot horizontal width of the layer, as herein provided. Each trip of the roller shall be offset from the path of the previous trip so that the total compactive effort shall be distributed evenly over the entire horizontal layer of Zone 4 embankment.

The pneumatic-tired roller shall meet the requirements of Subparagraph 90d.

Tractors used for pulling pneumatic-tired rollers shall have sufficient power to pull the fully loaded roller satisfactorily under normal conditions of compaction.

f. Measurement and payment.—Measurement and payment of silt, sand, gravel, and cobble fill in dam embankment, Zone 4, will be made as provided in this paragraph and in Paragraph 86.

Embankment constructed of materials from excavations for permanent construction required under these specifications and placed in silt, sand, gravel, and cobble fill in dam embankment, Zone 4, will be measured separately from embankment constructed of materials obtained from the stockpile of materials from Borrow Area C and C extension and placed in the dam embankment, Zone 4.

Measurement, for payment, of placing silt, sand, gravel, and cobble fill in dam embankment, Zone 4, will be made of the embankment constructed of materials from excavations for permanent construction required under these specifications. Payment for placing silt, sand, gravel, and cobble fill in dam embankment, Zone 4, will be made at the unit price per cubic yard bid therefor in the schedule, which unit price shall include all costs as provided in Paragraph 86.

Measurement, for payment, of procuring and placing silt, sand, gravel, and cobble fill in dam embankment, Zone 4, will be made of the embankment constructed of materials obtained from the stockpile of materials from Borrow Area C and C extension, and of materials obtained directly from Borrow Area C and C extension. Payment for procuring and placing silt, sand, gravel, and cobble fill in dam embankment, Zone 4, will be made at the unit price per cubic yard bid therefor in the schedule, which unit price shall include all costs as provided in Paragraph 86.

92. ROCKFILL IN DAM EMBANKMENT, ZONE 5

The rockfill, Zone 5, portions of the dam embankment shall be in accordance with this paragraph and Paragraph 86.

Materials for rockfill, Zone 5, shall consist of cobbles, boulders, and rock fragments reasonably well graded up to 1 cubic yard in volume, selected from excavation required for permanent construction under these specifications. Rock fragments larger than 1 cubic yard in volume may be embedded in the rockfill, Zone 5, or may be placed on the outer slope of the upstream Zone 5 embankment, or may be wasted as approved by the contracting officer.

The rockfill, Zone 5 portions of the dam embankment shall be placed in approximately horizontal layers not exceeding 3 feet in thickness. Equipment shall be routed over the layers already in place and traffic shall be distributed evenly over the entire width of fill so as to obtain the maximum amount of compaction possible. Successive loads of the material shall be dumped so as to secure the best practicable distribution of the material, as determined by the contracting officer, with the larger pieces placed toward the outer slopes. The material shall be dumped and roughly leveled off in an approved manner so as to maintain reasonably uniform surfaces on the fill and on the outer slopes of the embankment, and to insure that the completed fill will be stable and that there will be no large unfilled spaces within the fill.

Measurement, for payment, of rockfill in dam embankment, Zone 5, will be made as provided in Paragraph 86.

Payment for rockfill in dam embankment, Zone 5, will be made at the unit price per cubic yard bid therefor in the schedule, which unit price shall include all costs as provided in Paragraph 86.

DRILLING AND GROUTING

99. MOBILIZATION AND DEMOBILIZATION FOR DRILLING AND GROUTING

Payment for assembling the drilling and grouting plant and all equipment at the site preparatory to initiating the pressure grouting, and for removing it therefrom when the drilling and grouting program has been completed will be made at the lump-sum price bid in the schedule for mobilization and demobilization for drilling and grouting. Mobilization and demobilization for drilling and grouting shall include moving onto the site, complete assembly in working condition, and removing from the site all equipment and supplies necessary to perform the required drilling and pressure grouting operations.

The lump-sum price bid in the schedule will be paid only once and shall include complete mobilization and demobilization regardless of the amount of any additional equipment which may be required during the progress of the work, and regardless of the number of times the equipment is moved and reassembled. Sixty percent of the lump-sum price bid will be included in the monthly estimate for progress payments for the month during which pressure grouting has commenced. The remaining 40 percent will be included in the estimate for progress payments for the month during which the equipment is demobilized and removed from the site after completion of all drilling and pressure grouting.

100. REQUIREMENTS FOR GROUTING GENERAL

The contractor shall drill and grout under pressure:

(1) The rock foundation of the dam embankment, including blanket grouting of rock foundation in cutoff trench and foundation key trench as may be required, upstream and downstream rows of curtain grout holes as shown on the drawings, and the grout cap row of curtain grout holes with closeout holes as shown on the drawings or as directed;

(2) The rock surrounding portions of the tunnels, gate chambers, adit, and shafts as shown on the drawings;

(3) The rock formation and structures at other locations as shown on the drawings, or as directed; and

(4) Any faults, joints, shear zones, springs, and other foundation defects that may require grouting as determined by the contracting officer.

The amount of drilling and pressure grouting that will be required is uncertain and the contractor shall be entitled to no additional compensation above the unit prices bid in the schedule by reason of increase or decrease of the schedule quantities except as provided in Paragraph 104.

It is anticipated that grouting the foundation of Teton Dam will require more than average time and grout quantities and that considerable experimentation will be required to develop satisfactory procedures for drilling and grouting. The contractor shall schedule his operations to allow for these contingenies and provide ample time for drilling and grouting. The contractor shall be entitled to no additional allowance above the price bid in the schedule by reason of interference or delay to other phases of the work caused by drilling and grouting operations, including any necessary experimentation which is required to develop satisfactory procedures.

Dam embankment foundation grouting from hole collars above elevation 5090 shall be performed prior to placing adjoining Zone 1 embankment within 50 feet in elevation measured from the collar of the hole being grouted.

104. WATER TESTING AND PRESSURE GROUTING FOUNDATIONS

When connections are made to grout holes, cracks, crevices, seams, or where other foundation defects are evident, the defects shall first be washed with water and/or air under pressure to remove as much washable materials as possible.

CONCRETE STRUCTURES

136. CONCRETE IN BACKFILL

The item of the schedule for concrete in backfill includes all lean concrete in backfill beneath the concrete structures placed in additional excavation where directed as provided in Paragraph 67. The concrete backfill shall be placed where directed, to completely refill excavation made to remove unsuitable material from structure foundations. Concrete in backfill shall be lean concrete conforming to the applicable provisions of Paragraph 113 for backfill concrete. The quantity of concrete in backfill is uncertain and the contractor shall be entitled to no additional allowance above the prices bid in the schedule by reason of increased or decreased quantities of concrete in backfill being required.

137. CONCRETE IN GROUT CAP

The item of the schedule for concrete in grout cap includes all concrete placed in excavation for grout cap as provided in Paragraph 70.

The contractor may, at his option, finish or form the top of the grout cap to provide a stair step surface on the abutments: Provided, That any additional costs of labor or materials therefor shall be at the contractor's expense.

APPENDIX D-4

GRAVITY GROUT PLACEMENT

1. Summary of Gravity Grout Placement In Foundation Rock Surface Joints

2. Location of Gravity Grouting Of Surface Rock Defects

Summary
of
Gravity Grout Placement
in
Foundation Rock Surface Joints

Station	Location[1/]	Elev.	Take (cu.yd)	Date	Report By
(no surface grout treatment shown prior to 8/19/74)					
16+30	340U	-	-	8/19/74	Miller
"	355U	-	-	"	"
-	300-350U	-	2	8/22/74	Branson
-	25U	5075±	½	8/26/74	Farrell
-	75D	"	½	"	"
-	80D	"	1	"	"
-	160D	"	2	"	"
-	300U	-	6 3/4	8/23/74	"
-	℄	-	¼	"	"
15+95	75D	-	2	9/3/74	Smith
"	250U	-		"	"
"	160U	-	2	9/5/74	Branson
"	64D	-		"	"
16+20	74D	5083	¼	9/5/74	Michel
"	176U	5084	¼	"	"
16+00	15D	5083	½	"	"
16+20	74D	5085	½	9/6/74	Michel
16+10	45D	5086	¼	"	"
16+00	35D	"	¼	"	"
16+00	190D 5 holes	5085	*2+3	9/10/74	Michel
"	15D-90D	5086	½	"	"
"	300U	5085	½	"	"

* 2 neat cement w/c, .8/1; 3 backfill cement w/c/s, 1/1/1:

Station	Location	Elev.	Take (cu.yd)	Date	Report By
16+00	85D	5038	2+1½ backfill	9/13/74	Michel
16+20	120D	5089	½ backfill	"	"
16+00	60D	5092[5]	1½	9/17/74	Smith
"	25D	"	¼	"	"
	45D	"	¼	"	"
15+90	50D-75D	5087-5090[5]	20	9/24/74	Smith
15+70	65D	-	12	9/25/74	Smith
"	45D	-	20	"	"
"	20U	5096[5]	10	"	"

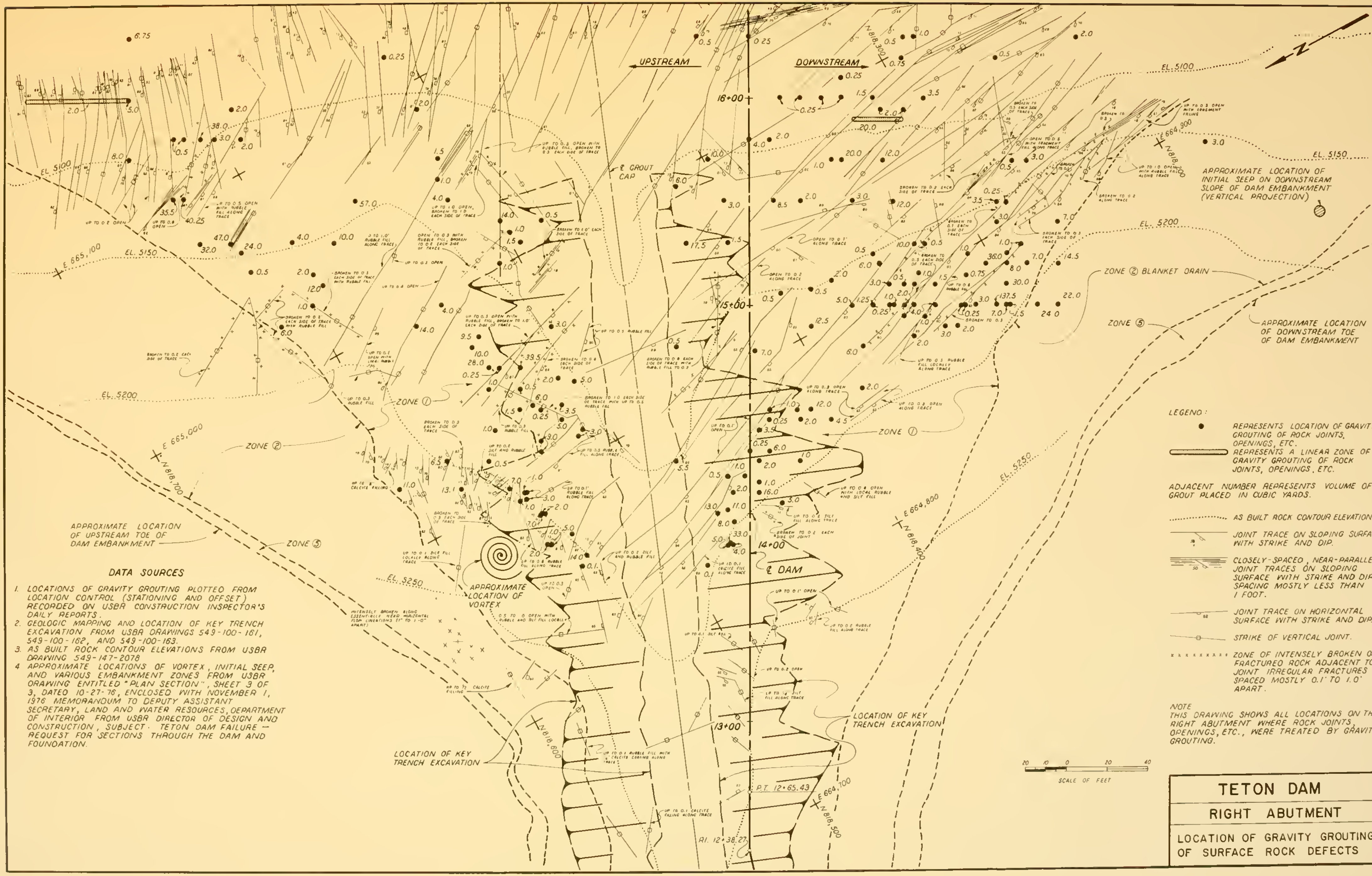
1/ Distance in feet upstream (U) or downstream (D) from centerline of dam.

Station	Location 1/	Elev.	Take (cu.yd)	Date	Report By
15+70	128D	5102	½	10/ /74	Smith
"	300U	5103	5½ 2½	"	"
"	150U	"	1½	"	"
15+57	35U	-	6	10/7/74	Metler
15+70	40D	5107	1	10/10/74	"
"	135D	5109	3	"	"
15+80	10D	5110	2	10/11/74	Metler
"	℄	"	4	"	"
"	265U	"	14 24	"	"
"	258U	"	3	"	"
"	223D	"	3	"	"
15+50	125D	5111		10/14/74	Evans
"	273U	5112	5 3/4	"	"
"	273U	"	35½	10/15/74	"
15+40	100U	5112	½	"	"
15+80	273U	5112	½	10/16/74	"
15+80	278U	"	¼	"	"
15+30	30U	5118	17	"	"
14+80	35U	5118	½	"	"
15+30	30U	5118	½	10/17/74	Evans
15+50	12D	"	8½	"	"
-	50D	5119	3	10/18/74	"
	12U	5121		"	"
	'75 construction season pick up grouting @ elev. 5152				
-	25D	5124	2	11/7/74	Miller
-	70D	5125	12	"	"
-	145U	5126	4	"	"
15+30	265U	5123	20 12	"	"
	250U	5126	6 10	11/8/74	"
"	220U	5127	4	"	"
15+30	250U	5133	15	11/13/74	Elenberger
"	"	"	16	11/14/74	"
4+52	90U	5142	3½	"	"
14+65	92U	5143	2	"	"
15+25	60D	5137	½	"	"
15+20	63D	5138	3	"	"
"	"	"	3	11/15/74	"
15+25	245U	5140	16/8	11/16/74	Elenberger
15+06	15D	5144	½	"	"
15+08	30D	5140	¼	"	"
15+12	40D	5140	2	"	"
15+16	82D	5139	"	"	"
	240U	5145 5/	22 ½	11/22/74	Smith
15+30	10U	5147	1½	"	"
"	85D	5144	½	"	"
"	80D	5148	10	11/25/74	Evans
	200U	"		"	"

Station	Location 1/	Elev.	Take (cu.yd)	Date	Report By
14+63	84U	5152	5	6/3/75	Hoyt
14+37	82U	5155	3	6/3/75	Hoyt
15+15	120D	5154	3½	6/7/75	Smith
14+70	110U	5159	39½	6/7/75	Smith
15+20	150D	5159	14½	6/10/75	Johnson
15+15	115D	5160	3/4	6/11/75	Johnson
15+10	90D	5161	1½	"	"
15+00	60D	5161	1¼	"	"
14+90	30D	5161	½	"	"
14+78	3D	5162	7	"	"
14+25	35U	5162	5½	"	"
14+40	80U	5163	3	"	"
15+30	110U	5163	1½	"	"
15+35	115U	5164	1	"	"
15+60	150U	5164	1	"	"
15+80	245U	5164	2	"	"
15+40	120U	5164	14	"	"
15+40	120D	5164	3	6/12/75	?
15+25	106D	5163	1	6/12/75	?
15+00	125D	? (top of cavern)	57	6/13/75	Johnson
15+00	125D	5166	7	6/18/75	Griffith
15+00	120D	5166	27½	6/19/75	Johnson
15+00	120D	5166	110	6/18/75	Griffith
15+00	150D	5165	7½	6/19/75	Johnson
15+00	140D	5165	20	"	"
15+00	90D	5166	4	"	"
15+00	75D	5167	2	"	"
15+00	150D	5169	14½	"	"
15+00	70D	5167	1	"	"
15+00	127D	5167	1½	"	"
15+00	103D	5167	¼	6/20/75	Johnson
15+00	105D	5169	¼	"	"
15+00	50D	5167	¼	"	"
15+00	68D	5170	¼	"	"
14+45	10D	5168	¼	"	"
14+40	5D	5168	3½	"	"
14+30	0U(Ⱡ)	5168	¼	"	"
14+50	100U	5171	¼	"	"
14+60	110U	5170	½	"	"
14+70	120U	5172	¼	"	"
14+52	104U	5172	6	"	"
14+18	10U	5169	½	"	"
15+00	140D	5169	4	6/20/75	Stocker
14+95	85D	5166	1	"	"
14+95	75D	5166	4	"	"
14+90	30D	5167	12	"	"
14+15	5D	5167	1	"	"
14+45	90U	5167	5	6/20/75	Stocker
14+25	25D	5167	1	"	"
15+20	115D	5168	1	"	"

Station	Location[1]	Elev.	Take (cu.yd)	Date	Report By
15+20	120U	5168	1	6/20/75	Stocker
?	110U	5169	6	7/1/75	?
"	115U	5169	¼	"	"
"	120U	5169	3	"	"
"	100U	5169	¼	"	"
"	12U	5171	7	"	"
	25D	5171	¼	"	"
"	60D	5169	1¼	"	"
"	80D	5169	12	"	"
"	75D	5168	½	"	"
	100D	5168	½	"	"
	125D	5168	24	"	"
15+10	205U	?	12	7/2/75	Hoyt
15+15	210U	"	2	"	"
14+50	110U	"	1½	"	"
14+85	130U	"	9½	"	"
14+35	100U	"	9	"	"
14+50	10D	"	1	"	"
15+10	75D	"	½	"	"
15+25	125D	"	1	"	"
14+30	100U	5179	9	7/9/75	Johnson
14+55	?	5179	3	"	"
14+90	160U	5179	14	"	"
14+90	225U	5179	6	"	"
14+18	2U	5179	1	"	"
14+80	55D	5178	6	"	"
14+85	80D	5178	2	"	"
14+90	101D	5177	2	"	"
14+10	8U	5180	2	7/10/75	Michel
14+25	5D	5180	2	"	"
14+30	10D	5180	6	"	"
15+00	210U	5180	1	"	"
15+20	115D	5175	7	"	"
15+40	150D	5175	6	"	"
15+20	115D	5175	28	7/12/75	Albertson
15+30	132D	5175	1	"	"
15+40	150D	5175	1	"	"
13+85	10U	5180	5	"	"
14+50	23D	5178	1	"	"
14+70	125U	5180	28	"	"
14+00	98U	5185	2	"	"
14+10	107U	5185	3	"	"
14+40	122U	5185	1	"	"
14+60	125US	5185	1	7/12/75	Albertson
15+00	148U	5180	4	"	"
14+80	133U	5180	10	"	"
13+85	8U	5180	4	"	"
15+00	80D	5185	4	"	"
13+75	20U	5180	1	7/15/75	Michel
13+80	78U	5180	14	"	"
14+10	108U	5180	1	"	"

Station	Location [1/]	Elev.	Take (cu.yd)	Date	Report By
14+50	30D	5180	12	7/15/75	Michel
14+45	25D	5183	2	"	"
15+10	80D	5180	2	"	"
15+20	135D	5185	7	7/18/75	Albertson
15+10	125D	5185	30	"	"
15+00	120D	5185	17	"	"
15+20	125D	5185	8	7/18/75	Johnson
15+00	50D	5185	1	"	"
15+10	68D	5185	3	"	"
15+00	110D	5186	3	7/21/75	Smith
14+90	95D	5186	3	"	"
14+60	55D	5185	2	"	"
14+45	40D	5185	4½	"	"
13+90	85U	5186	5	7/21/75	Smith
14+07	107U	5187	3	"	"
14+25	125U	5190	½	"	"
14+10	5D	5188	16	"	"
14+05	16D	5188	5.0	7/22/75	Stocker
14+90	95U		3.0	"	"
14+00	15U	5194	13	7/24/75	Smith
14+02	10U	5194	11	"	"
14+25	145U	5199	6½	7/28/75	Smith
14+06	110U	5196	1	"	"
13+85	97U	5195	2	"	"
13+85	95U	5200	1	8/1/75	Hoht
14+00	100U	5200	7	"	"
13+86	9U	5204	33	8/4/75	Entwisle
14+12	115U	5204	7	"	"
14+12	138U	5203	13	"	"
14+12	158U	5203	11	"	"
13+96	7U	5204	8	"	"
14+00	100U	5203	2	8/5/75	Entwisle
13+75	80U	5204	0.1	"	"
14+12	138U	5205	0.1	"	"


UPSTREAM
DOWNSTREAM
16+00
15+00
14+00
13+00
℄ DAM
℄ GROUT CAP
EL. 5100
EL. 5150
EL. 5200
EL. 5250
E 665,100
E 665,000
E 664,900
E 664,800
E 664,700
N 818,300
N 818,400
N 818,500
N 818,600
N 818,700
N 818,800
ZONE ①
ZONE ②
ZONE ⑤
ZONE ② BLANKET DRAIN
P.T. 12+65.43
P.I. 12+38.27
APPROXIMATE LOCATION OF VORTEX
LOCATION OF KEY TRENCH EXCAVATION
APPROXIMATE LOCATION OF UPSTREAM TOE OF DAM EMBANKMENT
APPROXIMATE LOCATION OF DOWNSTREAM TOE OF DAM EMBANKMENT
APPROXIMATE LOCATION OF INITIAL SEEP ON DOWNSTREAM SLOPE OF DAM EMBANKMENT (VERTICAL PROJECTION)
DATA SOURCES
1. LOCATIONS OF GRAVITY GROUTING PLOTTED FROM LOCATION CONTROL (STATIONING AND OFFSET) RECORDED ON USBR CONSTRUCTION INSPECTOR'S DAILY REPORTS.
2. GEOLOGIC MAPPING AND LOCATION OF KEY TRENCH EXCAVATION FROM USBR DRAWINGS 549-100-161, 549-100-162, AND 549-100-163.
3. AS BUILT ROCK CONTOUR ELEVATIONS FROM USBR DRAWING 549-147-2078
4. APPROXIMATE LOCATIONS OF VORTEX, INITIAL SEEP, AND VARIOUS EMBANKMENT ZONES FROM USBR DRAWING ENTITLED "PLAN SECTION", SHEET 3 OF 3, DATED 10-27-76, ENCLOSED WITH NOVEMBER 1, 1976 MEMORANDUM TO DEPUTY ASSISTANT SECRETARY, LAND AND WATER RESOURCES, DEPARTMENT OF INTERIOR FROM USBR DIRECTOR OF DESIGN AND CONSTRUCTION, SUBJECT: TETON DAM FAILURE — REQUEST FOR SECTIONS THROUGH THE DAM AND FOUNDATION.
LEGEND:
REPRESENTS LOCATION OF GRAVITY GROUTING OF ROCK JOINTS, OPENINGS, ETC.
REPRESENTS A LINEAR ZONE OF GRAVITY GROUTING OF ROCK JOINTS, OPENINGS, ETC.
ADJACENT NUMBER REPRESENTS VOLUME OF GROUT PLACED IN CUBIC YARDS.
AS BUILT ROCK CONTOUR ELEVATIONS
JOINT TRACE ON SLOPING SURFACE WITH STRIKE AND DIP.
CLOSELY-SPACED, NEAR-PARALLEL JOINT TRACES ON SLOPING SURFACE WITH STRIKE AND DIP. SPACING MOSTLY LESS THAN 1 FOOT.
JOINT TRACE ON HORIZONTAL SURFACE WITH STRIKE AND DIP.
STRIKE OF VERTICAL JOINT.
ZONE OF INTENSELY BROKEN OR FRACTURED ROCK ADJACENT TO JOINT. IRREGULAR FRACTURES SPACED MOSTLY 0.1' TO 1.0' APART.
NOTE
THIS DRAWING SHOWS ALL LOCATIONS ON THE RIGHT ABUTMENT WHERE ROCK JOINTS, OPENINGS, ETC., WERE TREATED BY GRAVITY GROUTING.
20 10 0 20 40
SCALE OF FEET
TETON DAM
RIGHT ABUTMENT
LOCATION OF GRAVITY GROUTING OF SURFACE ROCK DEFECTS

APPENDIX D-5

EMBANKMENT MATERIAL PROPERTIES

CONTENTS

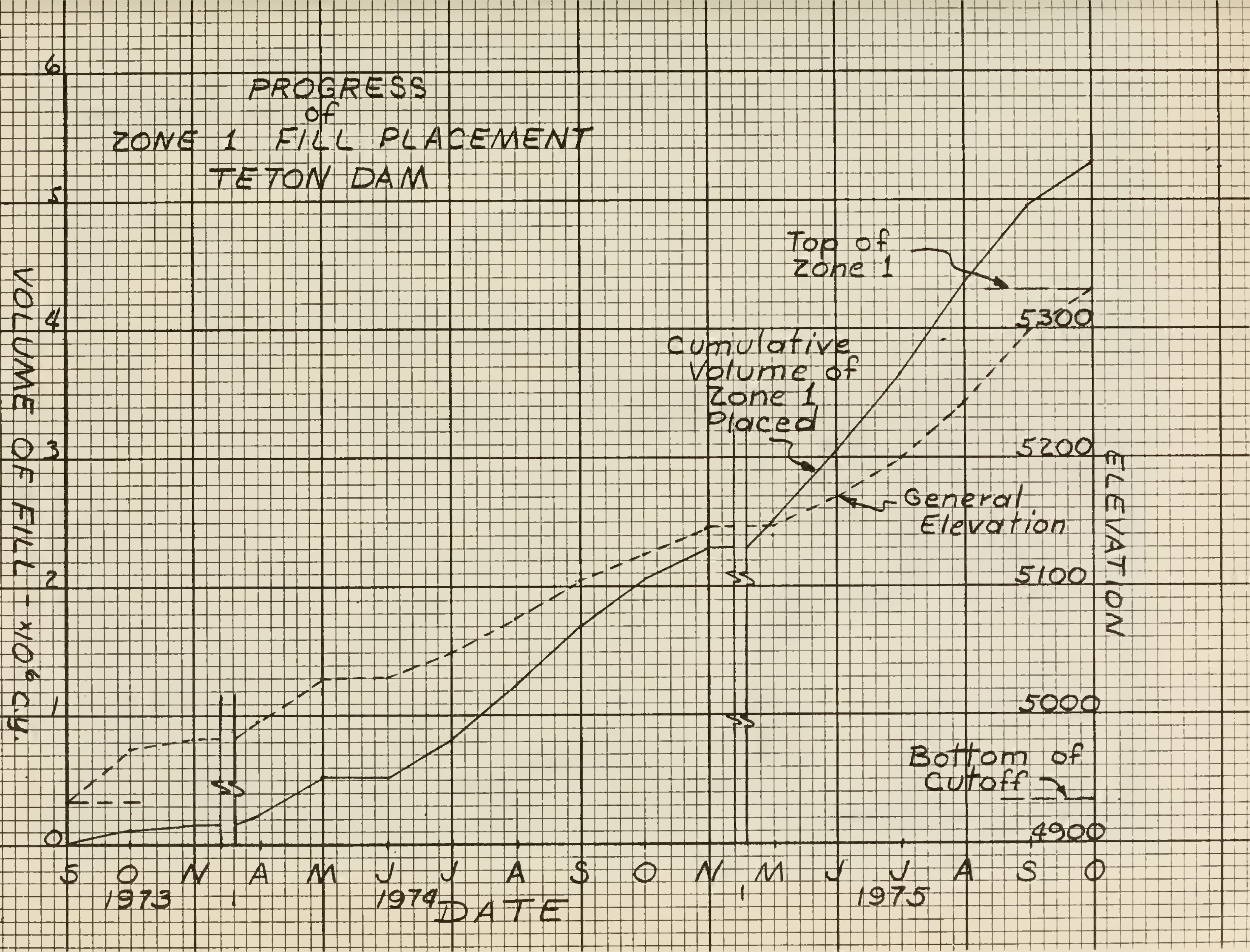
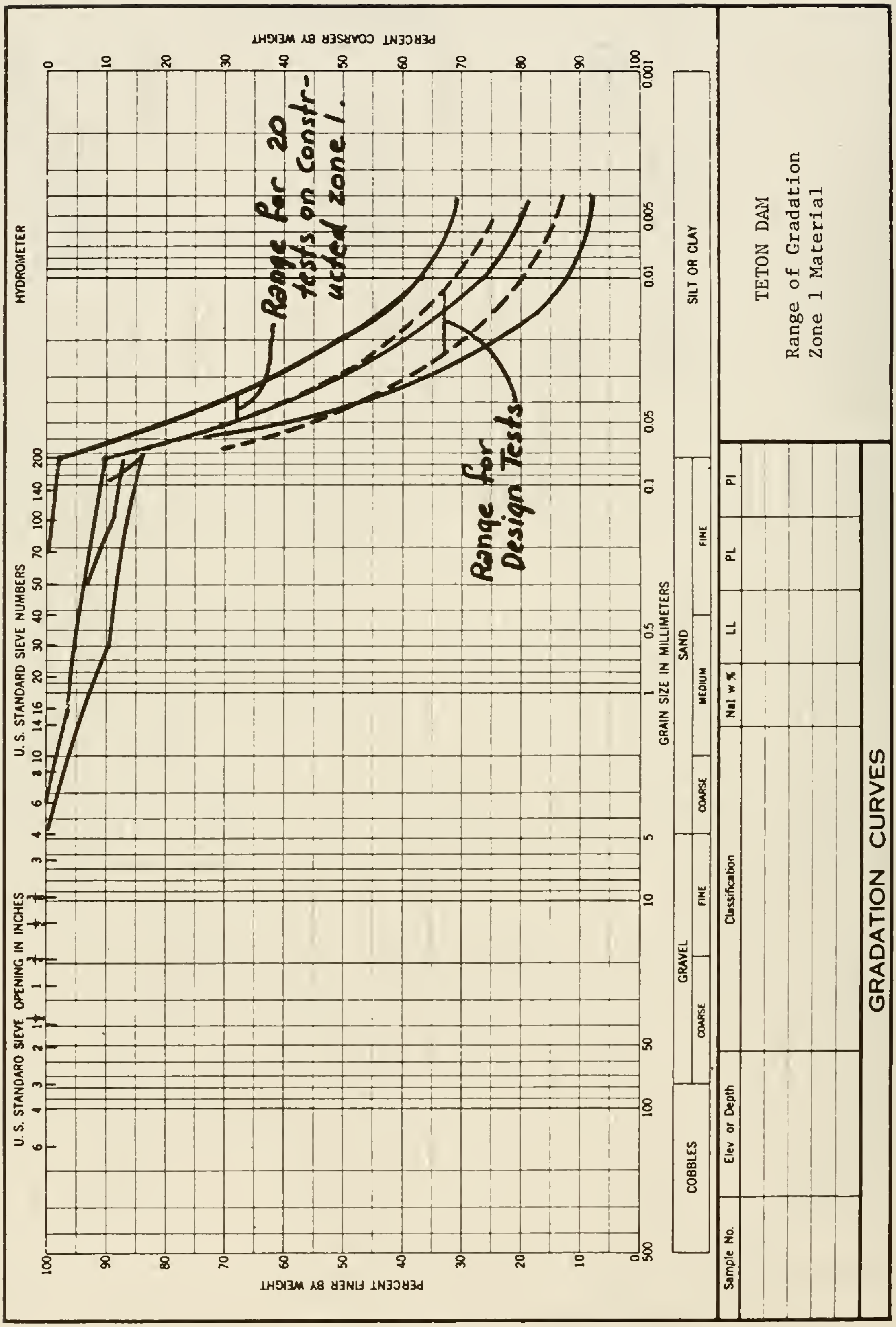

PROGRESS
of
ZONE 1 FILL PLACEMENT
TETON DAM
Top of Zone 1
Cumulative Volume of Zone 1 Placed
General Elevation
Bottom of Cutoff
ELEVATION
5300
5200
5100
5000
4900
VOLUME OF FILL - x10^6 C.Y.
6
5
4
3
2
1
0
S O N A M J J A S O N M J J A S O
1973
1974
1975
DATE

Figure D5-1



U. S. STANDARD SIEVE OPENING IN INCHES
U. S. STANDARD SIEVE NUMBERS
HYDROMETER
PERCENT FINER BY WEIGHT
PERCENT COARSER BY WEIGHT
GRAIN SIZE IN MILLIMETERS
COBBLES
GRAVEL
COARSE
FINE
SAND
COARSE
MEDIUM
FINE
SILT OR CLAY
Range for 20 tests on constructed zone 1.
Range for Design Tests
Sample No.
Elev or Depth
Classification
Nat w %
LL
PL
PI
TETON DAM
Range of Gradation
Zone 1 Material
GRADATION CURVES

Figure D5-2

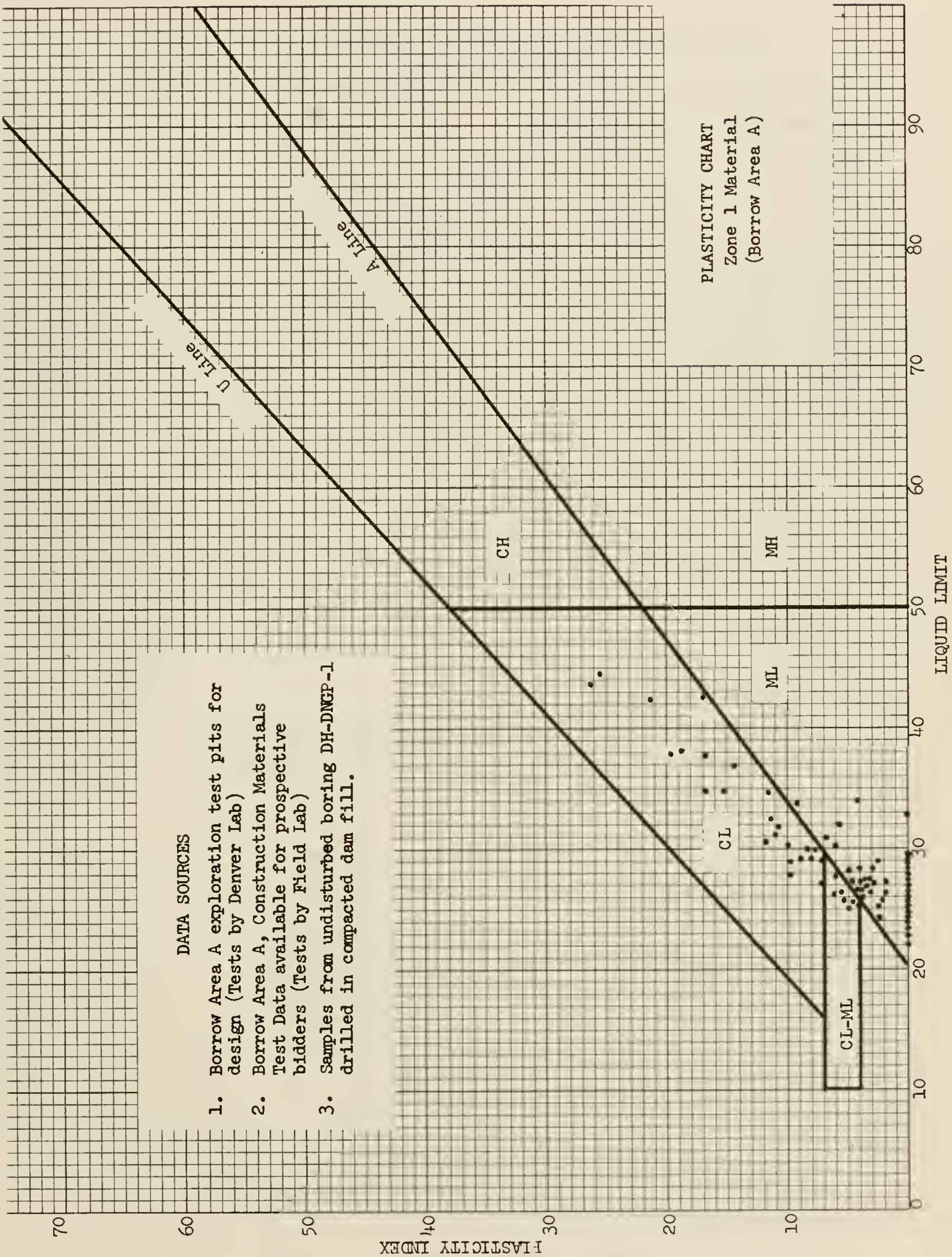

PLASTICITY CHART
Zone 1 Material
(Borrow Area A)
DATA SOURCES
1. Borrow Area A exploration test pits for design (Tests by Denver Lab)
2. Borrow Area A, Construction Materials Test Data available for prospective bidders (Tests by Field Lab)
3. Samples from undisturbed boring DH-DNGP-1 drilled in compacted dam fill.
U Line
A Line
CH
MH
ML
CL
CL-ML
PLASTICITY INDEX
LIQUID LIMIT
0
10
20
30
40
50
60
70
80
90

Figure D5-3

STATISTICAL ANALYSIS OF TEST RESULTS FOR DENSITY CONTROL OF FILL

Project Teton Dam Feature Zone 1 fill

Period Oct. 1973 To Oct. 31, 1975

Fill Quantity: Approx. 5,300,000 yd^3

$$\text{Percent Compaction} = \frac{\text{Fill Dry Density}}{\text{Maximum Laboratory Dry Density}} \times 100$$

	Prev. Cum F	This Period: Frequency (F)	This Period: F	This Period: Cum F	This Period: Cum %	To Date: F	To Date: Cum F	To Date: Cum %
93.0-93.9						6	6	0
94.0-94.9						246	252	9
95.0-95.9						260	512	19
96.0-96.9						356	868	32
97.0-97.9						410	1278	47
98.0-98.9						416	1694	62
99.0-99.9						366	2060	75
100.0-100.9						292	2352	86
101.0-101.9						184	2536	93
102.0-102.9						114	2650	97
103.0-103.9						50	2700	98
104.0-104.9						18	2718	99
105.0-109.9						14	2732	100
Totals		—	—		—	—	2732	—

	Prev.	This Period	To Date
Avg. fill dry density, γ_{df}, pcf			99.1
Avg. maximum dry density, γ_{dL}, pcf (Lab.)			100.8
Mean percentage of lab. max. dry density			98.3

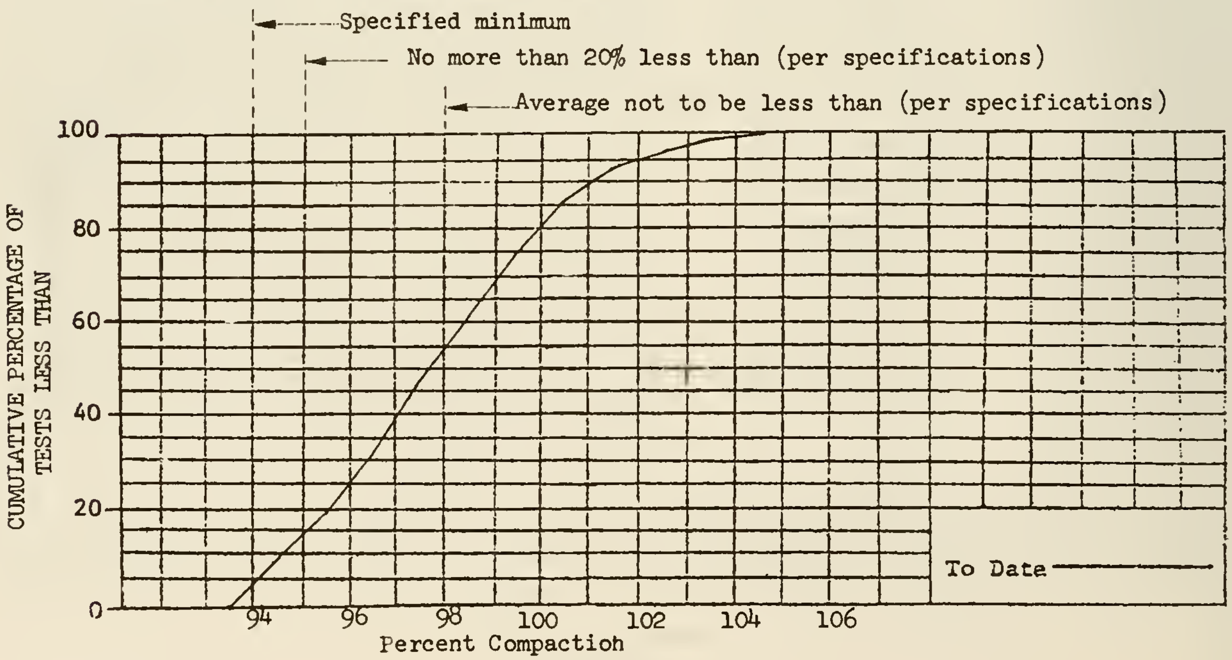


Figure D5-4

STATISTICAL ANALYSIS OF TEST RESULTS FOR MOISTURE CONTROL OF FILL

Project Teton Dam Feature Zone 1 Fill

Period Oct. 1973 To Oct. 31, 1975

Fill Quantity: Approx. 5,300,000 yd^3

OPTIMUM MOISTURE CONTENT MINUS FILL MOISTURE CONTENT (Wo-Wf) IN PERCENT OF DRY WEIGHT

		Prev. Cum F	This Period				To Date		
			Frequency (F)	F	Cum F	Cum %	F	Cum F	Cum %
W_f, Dryer than optimum	3.3-3.7						67	67	2
	2.8-3.2						100	167	6
	2.3-2.7						239	406	15
	1.8-2.2						489	895	33
	1.3-1.7						349	1244	46
	0.8-1.2						558	1802	66
	0.3-0.7						168	1907	72
	0.2 to 0.2						242	2212	81
W_f, Wetter than optimum	0.3-0.7						273	2485	91
	0.8-1.2						212	2697	99
	1.3-1.7						17	2714	99
	1.8-2.2						11	2725	99
	2.3-2.7						4	2729	99
	2.8-3.2						2	2731	99
	3.3-3.7						1	2732	100
	Totals		—	—		—	—	2732	—

	Prev.	This Period	To Date
Avg. fill moisture content, W_f, %			18.5
Avg. optimum moisture content, W_o, %			19.6
Mean variation from optimum moisture content			1.1 Dry

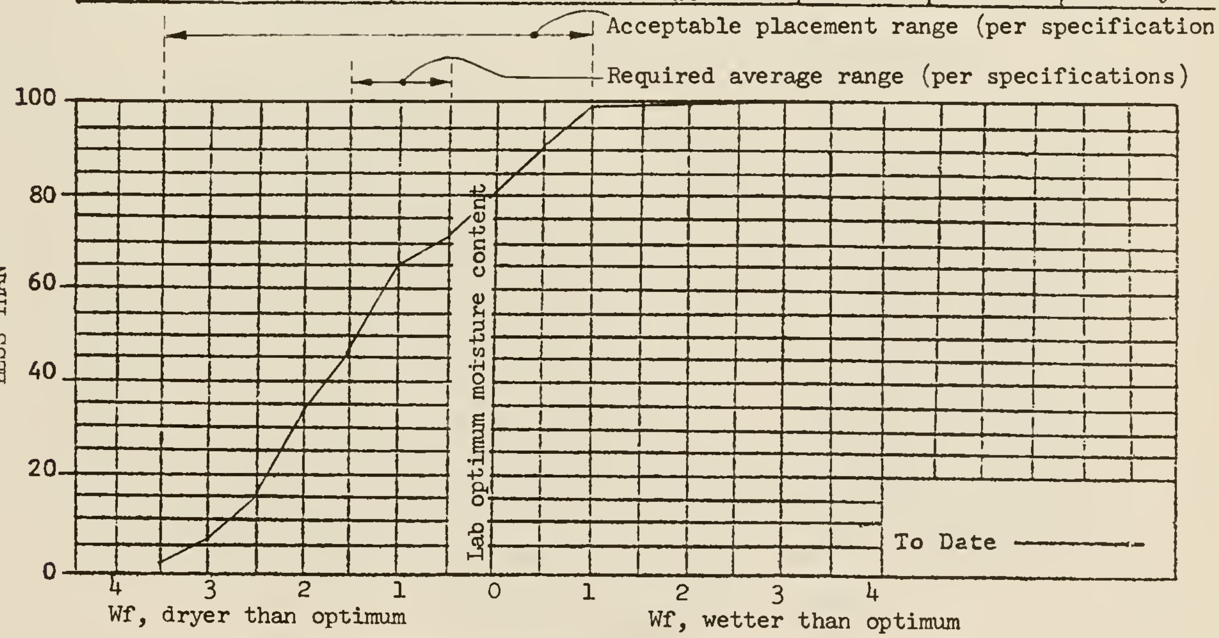


Figure D5-5

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

Project Teton Dam Feature Zone 1

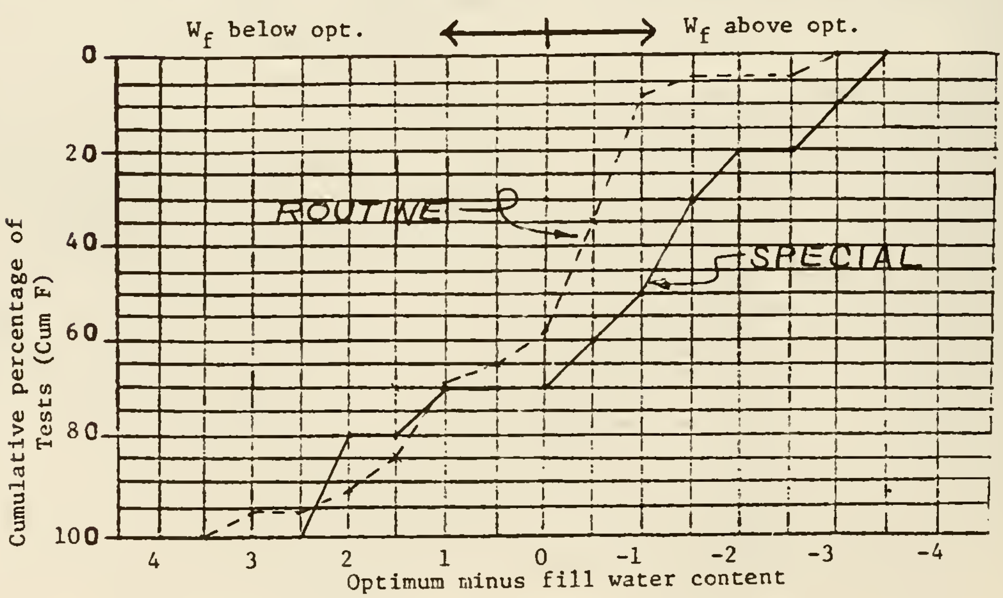
Period 11-1-73 To 11-30-73

Fill Quantity : 55,900 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content (W_o - W_f) in precent of dry weight

		SPECIAL			ROUTINE		
		F*	Cum F	Cum %	F*	Cum F	Cum %
W_f is above opt.	4.2 - 3.8						
	3.7 - 3.3			0			
	3.2 - 2.8	1	1	10			0
	2.7 - 2.3	1	2	20	1	1	4
	2.2 - 1.8		2	20		1	4
	1.7 - 1.3	1	3	30		1	4
	1.2 - 0.8	2	5	50	1	2	8
	0.7 - 0.2	1	6	60	7	9	35
	+0.2 to -0.2	1	7	70	6	15	58
W_f is below opt.	0.3 - 0.7		7	70	2	17	65
	0.8 - 1.2		7	70	1	18	69
	1.3 - 1.7	1	8	80	4	22	85
	1.8 - 2.2		8	80	2	24	92
	2.3 - 2.7	2	10	100	1	25	96
	2.8 - 3.2					25	96
	3.3 - 3.7				1	26	100
	Totals	10	—	—	26	—	—



*F = Frequency distribution of tests

Figure D5-6

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

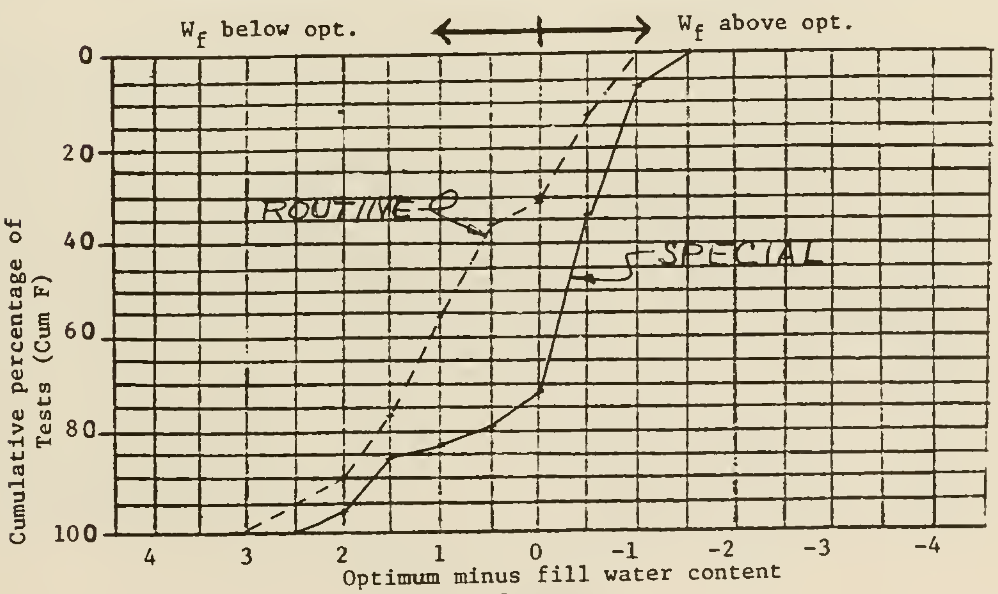
Project Teton Dam Feature Zone 1

Period 4-1-74 To 4-30-74

Fill Quantity: 70,000 yd^3

Optimum minus fill water content (W_o - W_f) in precent of dry weight

		COMPACTED EMBANKMENT					
		SPECIAL			ROUTINE		
		F*	Cum F	Cum %	F*	Cum F	Cum %
W_f is above opt.	4.2 - 3.8						
	3.7 - 3.3						
	3.2 - 2.8						
	2.7 - 2.3						
	2.2 - 1.8						
	1.7 - 1.3			0			
	1.2 - 0.8	2	2	7			0
	0.7 - 0.2	8	10	34	5	5	13
+0.2 to -0.2		11	21	72	7	12	31
W_f is below opt.	0.3 - 0.7	2	23	79	2	14	36
	0.8 - 1.2	1	24	83	8	22	56
	1.3 - 1.7	1	25	86	8	30	77
	1.8 - 2.2	3	28	97	5	35	90
	2.3 - 2.7	1	29	100	2	37	95
	2.8 - 3.2				2	39	100
	3.3 - 3.7						
	Totals	29	—	—	39	—	—



*F = Frequency distribution of tests

Figure D5-7

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

Project Teton Dam Feature Zone 1

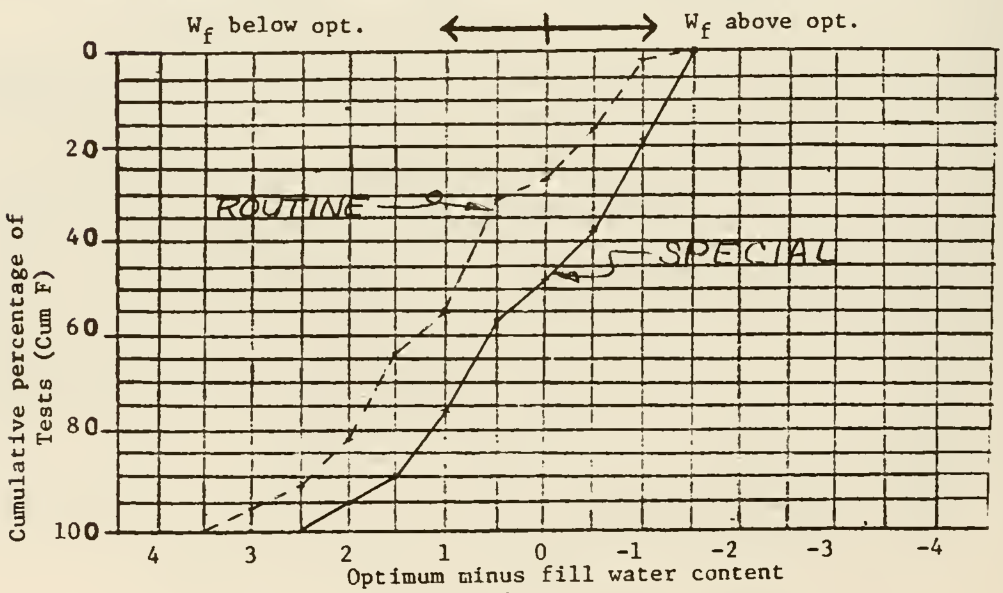
Period 5-1-74 To 5-31-74

Fill Quantity: 295,000 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content (W_o - W_f) in percent of dry weight

		SPECIAL			ROUTINE		
		F*	Cum F	Cum %	F*	Cum F	Cum %
W_f is above opt.	4.2 - 3.8						
	3.7 - 3.3						
	3.2 - 2.8						
	2.7 - 2.3						
	2.2 - 1.8						
	1.7 - 1.3			0			0
	1.2 - 0.8	4	4	19	2	2	2
	0.7 - 0.2	4	8	38	17	19	16
+0.2 to -0.2		2	10	48	14	33	27
W_f is below opt.	0.3 - 0.7	2	12	57	5	38	31
	0.8 - 1.2	4	16	76	28	66	55
	1.3 - 1.7	3	19	90	12	78	64
	1.8 - 2.2	1	20	95	21	99	82
	2.3 - 2.7	1	21	100	12	111	92
	2.8 - 3.2				5	116	96
	3.3 - 3.7				5	121	100
	Totals	21	—	—	121	—	—



*F = Frequency distribution of tests

Figure D5-8

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

Project Teton Dam Feature Zone 1

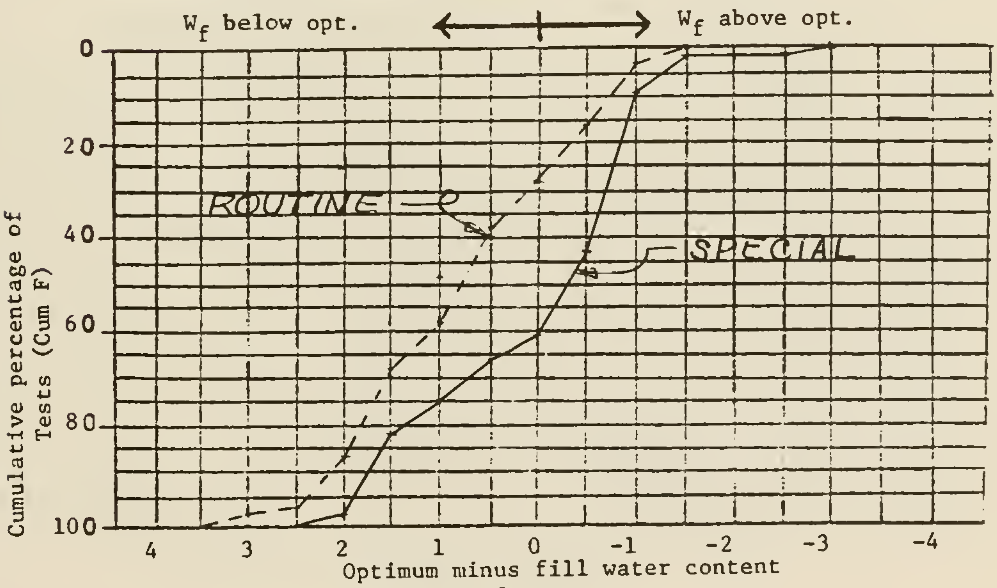
Period 7-1-74 To 7-31-74

Fill Quantity: 294,000 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content (W_o - W_f) in percent of dry weight

		Special F*	Special Cum F	Special Cum %	Routine F*	Routine Cum F	Routine Cum %
W_f is above opt.	4.2 - 3.8						
	3.7 - 3.3						
	3.2 - 2.8			0			
	2.7 - 2.3	1	1	2			
	2.2 - 1.8		1	2			
	1.7 - 1.3		1	2			0
	1.2 - 0.8	3	4	9	5	5	3
	0.7 - 0.2	15	19	43	23	28	16
+0.2 to -0.2		8	27	61	21	49	28
W_f is below opt.	0.3 - 0.7	2	29	66	17	66	38
	0.8 - 1.2	4	33	75	33	99	58
	1.3 - 1.7	3	36	82	18	117	68
	1.8 - 2.2	7	43	98	32	149	87
	2.3 - 2.7	1	44	100	17	166	97
	2.8 - 3.2				3	169	98
	3.3 - 3.7				3	172	100
	Totals	44	—	—	172	—	—



*F = Frequency distribution of tests

Figure D5-9

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

Project Teton Dam Feature Zone 1

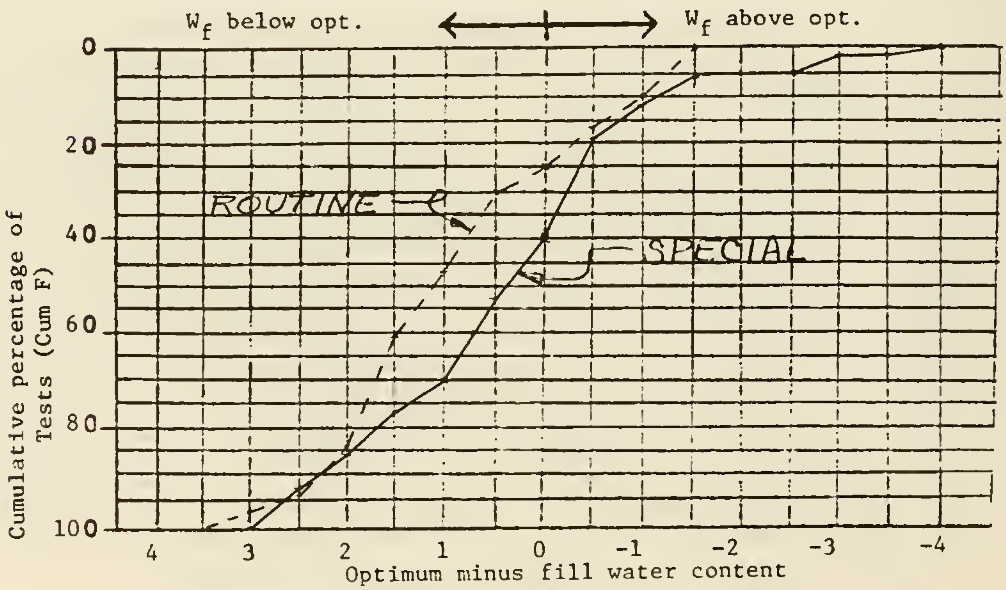
Period 8-1-74 To 8-31-74

Fill Quantity: 443,000 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content (W_o - W_f) in precent of dry weight

		SPECIAL			ROUTINE		
		F*	Cum F	Cum %	F*	Cum F	Cum %
W_f is above opt.	4.2 - 3.8			0			
	3.7 - 3.3	1	1	2			
	3.2 - 2.8		1	2			
	2.7 - 2.3	1	2	5			
	2.2 - 1.8		2	5			
	1.7 - 1.3		2	5			0
	1.2 - 0.8	3	5	12	15	15	10
	0.7 - 0.2	3	8	19	12	27	17
	+0.2 to -0.2	9	17	40	11	38	25
W_f is below opt.	0.3 - 0.7	6	23	53	9	47	30
	0.8 - 1.2	7	30	70	26	73	47
	1.3 - 1.7	3	33	77	22	95	61
	1.8 - 2.2	4	37	86	35	130	84
	2.3 - 2.7	3	40	93	15	145	94
	2.8 - 3.2	3	43	100	5	150	97
	3.3 - 3.7				5	155	100
	Totals	43	—	—	155	—	—



*F = Frequency distribution of tests

Figure D5-10

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

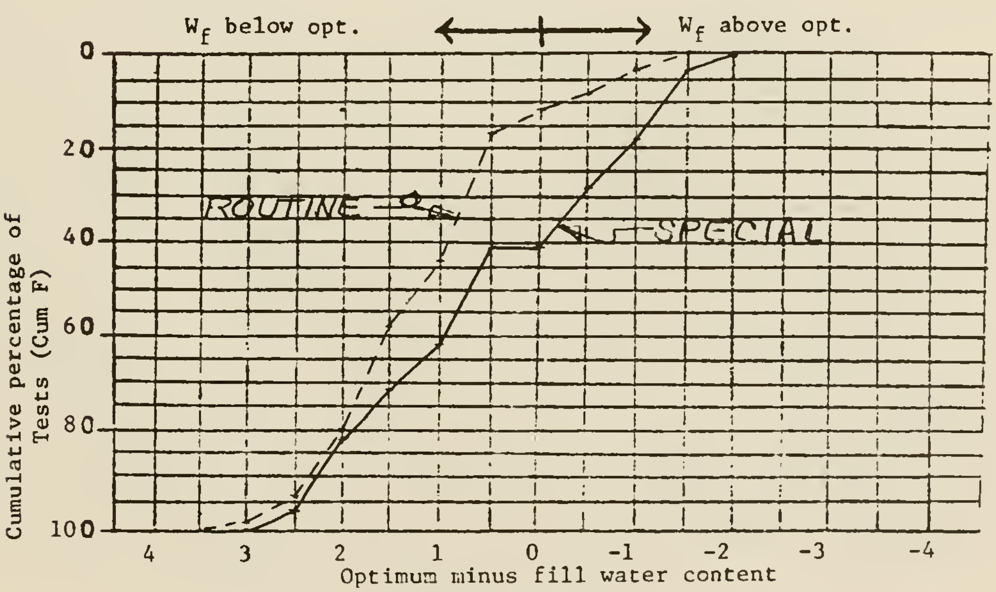
Project Teton Dam Feature Zone 1

Period 9-1-74 To 9-30-74

Fill Quantity: 437,100 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content ($W_o - W_f$) in precent of dry weight		SPECIAL F*	SPECIAL Cum F	SPECIAL Cum %	ROUTINE F*	ROUTINE Cum F	ROUTINE Cum %
W_f is above opt.	4.2 - 3.8						
	3.7 - 3.3						
	3.2 - 2.8						
	2.7 - 2.3						
	2.2 - 1.8			0			
	1.7 - 1.3	1	1	3			0
	1.2 - 0.8	6	7	18	6	6	3
	0.7 - 0.2	4	11	28	9	15	8
+0.2 to -0.2		5	16	41	8	23	12
W_f is below opt.	0.3 - 0.7		16	41	8	31	17
	0.8 - 1.2	8	24	62	50	81	44
	1.3 - 1.7	4	28	72	26	107	58
	1.8 - 2.2	4	32	82	41	148	80
	2.3 - 2.7	6	38	97	26	174	94
	2.8 - 3.2	1	39	100	8	182	98
	3.3 - 3.7				4	186	100
	Totals	39	—	—	186	—	—



*F = Frequency distribution of tests

Figure D5-11

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

Project Teton Dam　　　　Feature Zone 1

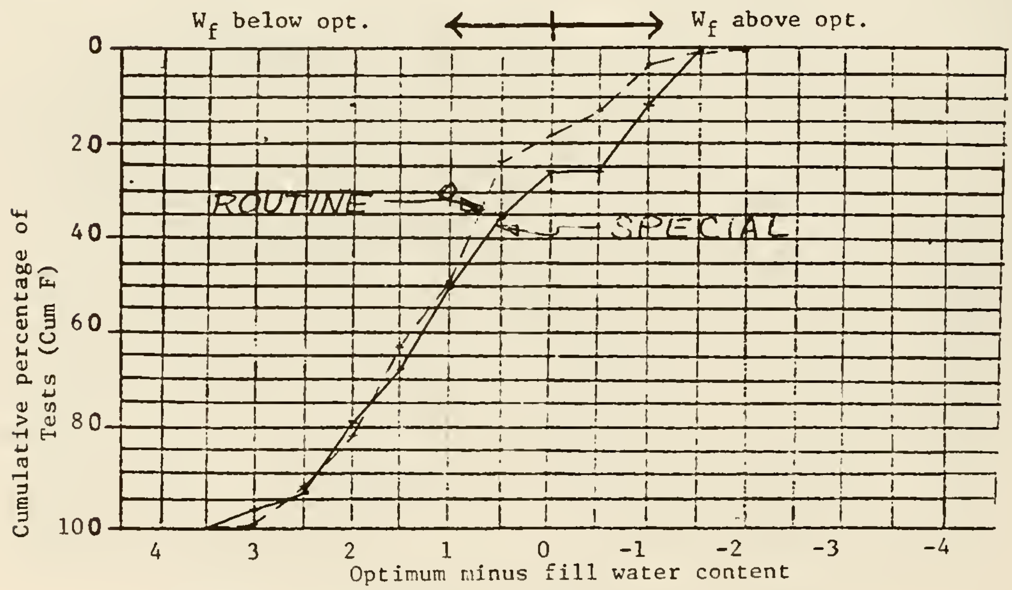
Period 10-1-74 To 10-31-74

Fill Quantity: 364,750 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content ($W_o - W_f$) in percent of dry weight

		SPECIAL			ROUTINE		
		F*	Cum F	Cum %	F*	Cum F	Cum %
W_f is above opt.	4.2 - 3.8						
	3.7 - 3.3						
	3.2 - 2.8						
	2.7 - 2.3						
	2.2 - 1.8						0
	1.7 - 1.3			0	1	1	1
	1.2 - 0.8	4	4	12	3	4	3
	0.7 - 0.2	5	9	26	12	16	13
+0.2 to -0.2			9	26	6	22	18
W_f is below opt.	0.3 - 0.7	3	12	35	7	29	24
	0.8 - 1.2	5	17	50	31	60	49
	1.3 - 1.7	6	23	68	17	77	63
	1.8 - 2.2	4	27	79	23	100	82
	2.3 - 2.7	5	32	94	14	114	93
	2.8 - 3.2	1	33	97	7	121	99
	3.3 - 3.7	1	34	100	1	122	100
	Totals	34	—	—	122	—	—



*F = Frequency distribution of tests

Figure D5-12

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

Project Teton Dam Feature Zone 1

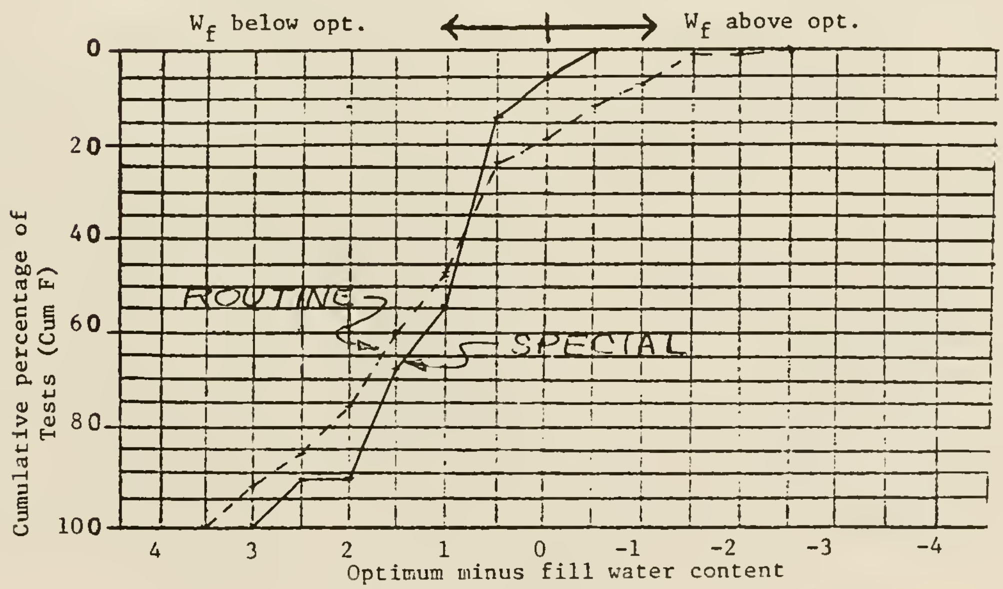
Period 11-1-74 To 11-30-74

Fill Quantity: 249,500yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content ($W_o - W_f$) in precent of dry weight

		SPECIAL			ROUTINE		
		F*	Cum F	Cum %	F*	Cum F	Cum %
W_f is above opt.	4.2 - 3.8						
	3.7 - 3.3						
	3.2 - 2.8						
	2.7 - 2.3						0
	2.2 - 1.8				1	1	1
	1.7 - 1.3					1	1
	1.2 - 0.8				4	5	7
	0.7 - 0.2			0	3	8	12
+0.2 to -0.2		1	1	5	5	13	19
W_f is below opt.	0.3 - 0.7	2	3	14	3	16	24
	0.8 - 1.2	9	12	55	16	32	47
	1.3 - 1.7	3	15	68	9	41	60
	1.8 - 2.2	5	20	91	11	52	76
	2.3 - 2.7		20	91	7	59	86
	2.8 - 3.2	2	22	100	4	63	93
	3.3 - 3.7				5	68	100
	Totals	22	—	—	68	—	—



*F = Frequency distribution of tests

Figure D5-13

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

Project Teton Dam Feature Zone 1

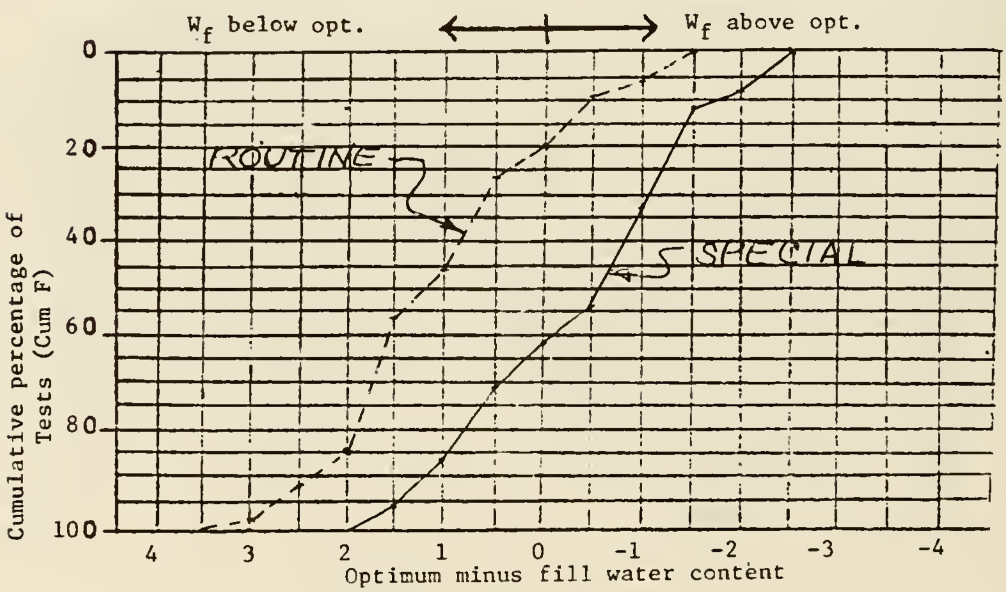
Period 5-1-75 To 5-31-75

Fill Quantity: 190,600 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content (W_o - W_f) in percent of dry weight

		SPECIAL			ROUTINE		
		F*	Cum F	Cum %	F*	Cum F	Cum %
W_f is above opt.	4.2 - 3.8						
	3.7 - 3.3						
	3.2 - 2.8						
	2.7 - 2.3			0			
	2.2 - 1.8	2	2	8			
	1.7 - 1.3	1	3	12			0
	1.2 - 0.8	5	8	33	6	6	6
	0.7 - 0.2	5	13	54	3	9	9
+0.2 to -0.2		2	15	62	11	20	20
W_f is below opt.	0.3 - 0.7	2	17	71	7	27	27
	0.8 - 1.2	4	21	87	19	46	46
	1.3 - 1.7	2	23	96	12	58	57
	1.8 - 2.2	1	24	100	28	86	85
	2.3 - 2.7				7	93	92
	2.8 - 3.2				6	99	98
	3.3 - 3.7				2	101	100
	Totals	24	—	—	101	—	—



*F = Frequency distribution of tests

Figure D5-14

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

Project Teton Dam   Feature Zone 1

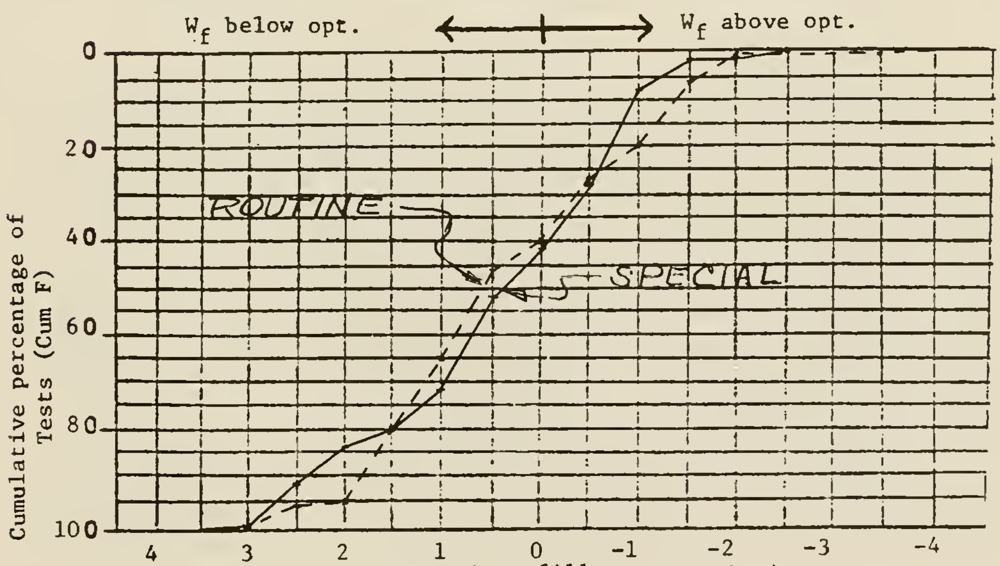
Period 6-1-75 To 6-30-75

Fill Quantity: 556,900 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content ($W_o - W_f$) in percent of dry weight

		SPECIAL			ROUTINE		
		F*	Cum F	Cum %	F*	Cum F	Cum %
W_f is above opt.	4.2 - 3.8						0
	3.7 - 3.3				2	2	1
	3.2 - 2.8					2	1
	2.7 - 2.3			0		2	1
	2.2 - 1.8	1	1	2		2	1
	1.7 - 1.3		1	2	6	8	6
	1.2 - 0.8	3	4	8	20	28	20
	0.7 - 0.2	10	14	28	11	39	27
+0.2 to -0.2		7	21	42	18	57	40
W_f is below opt.	0.3 - 0.7	5	26	52	8	65	46
	0.8 - 1.2	10	36	72	28	93	65
	1.3 - 1.7	4	40	80	20	113	80
	1.8 - 2.2	2	42	84	22	135	95
	2.3 - 2.7	4	46	92	2	137	96
	2.8 - 3.2	3	49	99	3	140	99
	3.3 - 3.7	1	50	100	2	142	100
	Totals	50	—	—	142	—	—



*F = Frequency distribution of tests

Figure D5-15

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

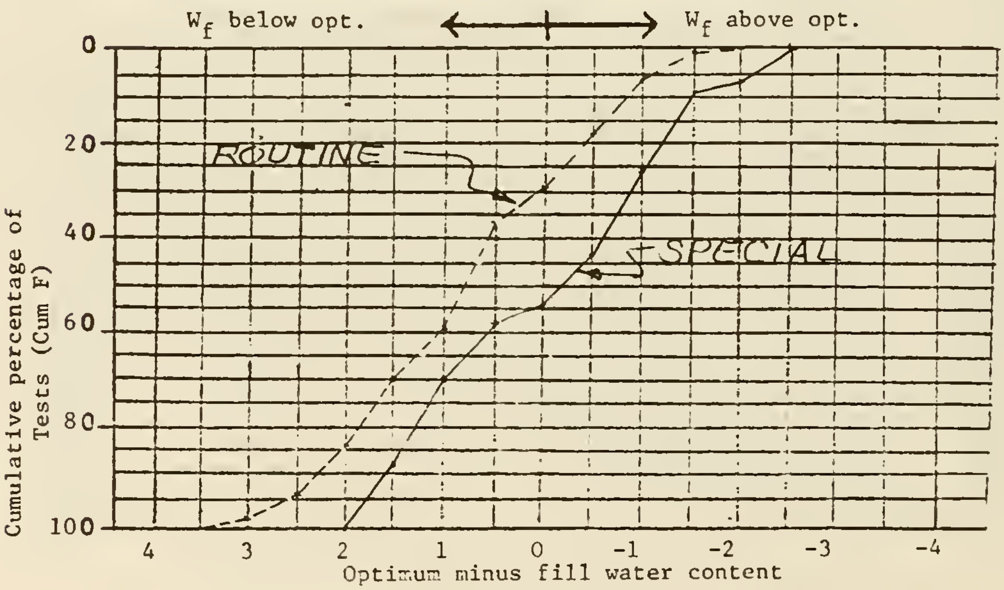
Project Teton Dam Feature Zone 1

Period 7-1-75 To 7-31-75

Fill Quantity: 583,700 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content (W_o - W_f) in percent of dry weight		SPECIAL F*	SPECIAL Cum F	SPECIAL Cum %	ROUTINE F*	ROUTINE Cum F	ROUTINE Cum %
W_f is above opt.	4.2 - 3.8						
	3.7 - 3.3						
	3.2 - 2.8				1	1	0
	2.7 - 2.3			0		1	0
	2.2 - 1.8	5	5	7		1	0
	1.7 - 1.3	1	6	9	1	2	1
	1.2 - 0.8	12	18	26	14	16	6
	0.7 - 0.2	12	30	43	30	46	18
+0.2 to -0.2		7	37	54	27	73	29
W_f is below opt.	0.3 - 0.7	3	40	58	21	94	37
	0.8 - 1.2	8	48	70	57	151	59
	1.3 - 1.7	13	61	88	28	179	70
	1.8 - 2.2	8	69	100	34	213	84
	2.3 - 2.7				25	238	94
	2.8 - 3.2				11	249	98
	3.3 - 3.7				5	254	100
	Totals	69	—	—	254	—	—



*F = Frequency distribution of tests

Figure D5-16

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

Project Teton Dam Feature Zone 1

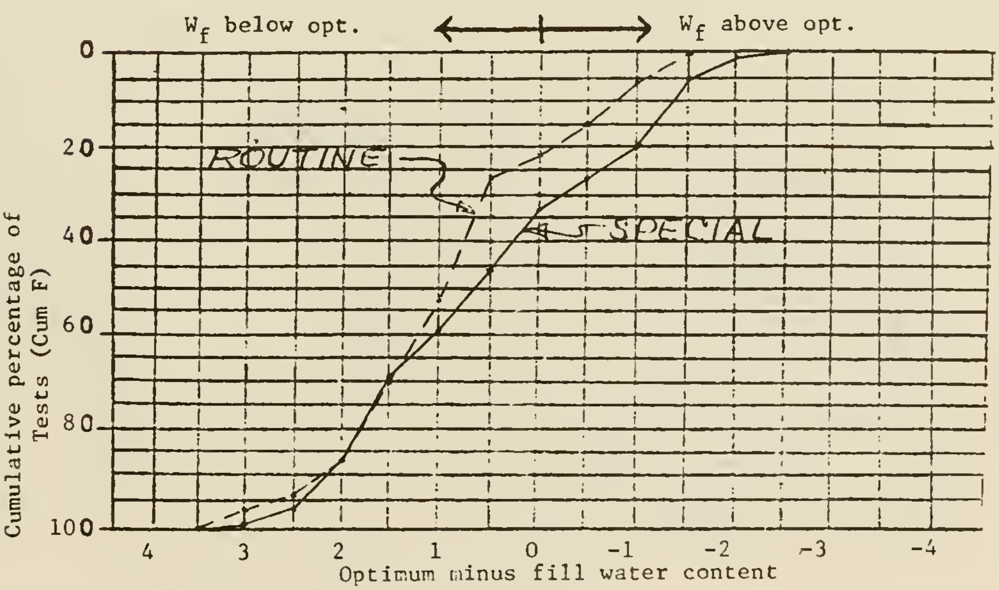
Period 8-1-75 To 8-31-75

Fill Quantity: 729,600 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content (W_o - W_f) in precent of dry weight

		SPECIAL			ROUTINE		
		F *	Cum F	Cum %	F *	Cum F	Cum %
W_f is above opt.	4.2 - 3.8						
	3.7 - 3.3						
	3.2 - 2.8						
	2.7 - 2.3			0			
	2.2 - 1.8	1	1	1			0
	1.7 - 1.3	2	3	5	1	1	0
	1.2 - 0.8	11	14	20	16	17	6
	0.7 - 0.2	5	19	27	27	44	15
+0.2 to -0.2		4	23	33	20	64	22
W_f is below opt.	0.3 - 0.7	9	32	46	17	81	27
	0.8 - 1.2	9	41	59	76	157	53
	1.3 - 1.7	7	48	69	50	207	70
	1.8 - 2.2	13	61	87	50	257	87
	2.3 - 2.7	7	68	97	22	279	94
	2.8 - 3.2	1	69	99	9	288	97
	3.3 - 3.7	1	70	100	9	297	100
	Totals	70	—	—	297	—	—



*F = Frequency distribution of tests

Figure D5-17

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

Project Teton Dam　　Feature Zone 1

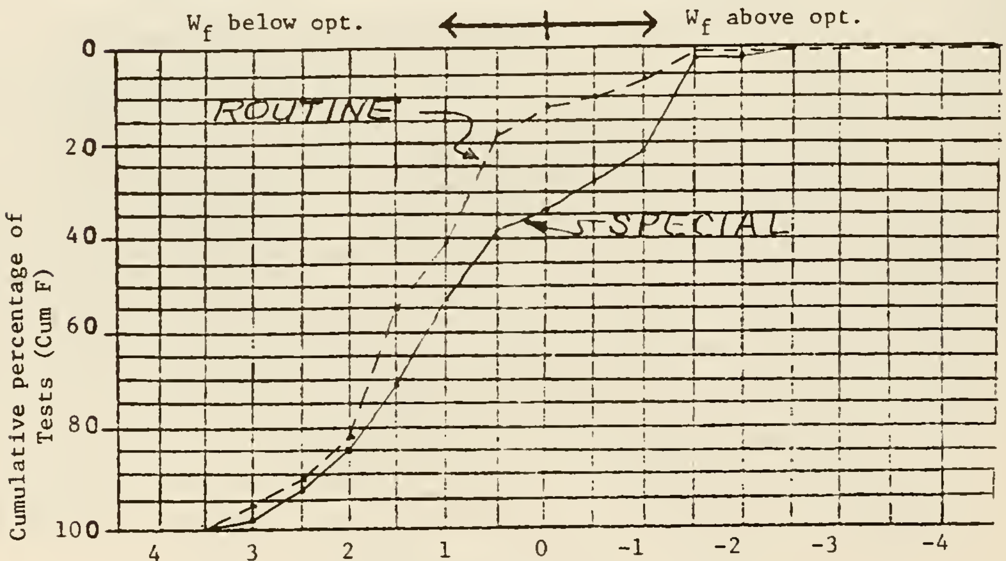
Period 9-1-75 To 9-30-75

Fill Quantity: 577,200 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content ($W_o - W_f$) in precent of dry weight

		SPECIAL			ROUTINE		
		F*	Cum F	Cum %	F*	Cum F	Cum %
W_f is above opt.	4.2 - 3.8				1	2	1
	3.7 - 3.3					2	1
	3.2 - 2.8					2	1
	2.7 - 2.3			0	1	3	1
	2.2 - 1.8	1	1	2		3	1
	1.7 - 1.3		1	2		3	1
	1.2 - 0.8	10	11	22	15	18	6
	0.7 - 0.2	3	14	28	11	29	10
+0.2 to -0.2		3	17	34	6	35	12
W_f is below opt.	0.3 - 0.7	2	19	38	17	52	18
	0.8 - 1.2	8	27	53	66	118	41
	1.3 - 1.7	9	36	71	41	159	55
	1.8 - 2.2	7	43	85	78	237	82
	2.3 - 2.7	4	47	93	26	263	91
	2.8 - 3.2	3	50	98	16	279	96
	3.3 - 3.7	1	51	100	11	290	100
	Totals	51	—	—	290	—	—



*F = Frequency distribution of tests

Figure D5-18

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

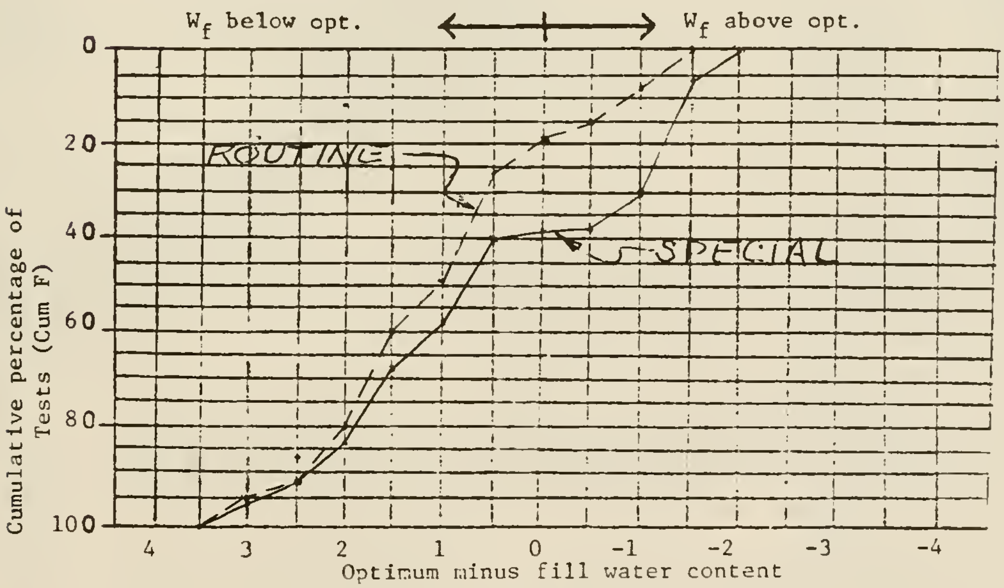
Project Teton Dam Feature Zone 1

Period 10-1-75 To 10-31-75

Fill Quantity: 348,760 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content ($W_o - W_f$) in percent of dry weight		SPECIAL F*	SPECIAL Cum F	SPECIAL Cum %	ROUTINE F*	ROUTINE Cum F	ROUTINE Cum %
W_f is above opt.	4.2 - 3.8						
	3.7 - 3.3						
	3.2 - 2.8						
	2.7 - 2.3						
	2.2 - 1.8			0			
	1.7 - 1.3	3	3	6	0		0
	1.2 - 0.8	12	15	30	9	9	8
	0.7 - 0.2	4	19	38	7	16	15
+0.2 to -0.2			19	38	5	21	19
W_f is below opt.	0.3 - 0.7	1	20	40	7	28	26
	0.8 - 1.2	9	29	58	25	53	49
	1.3 - 1.7	5	34	68	12	65	60
	1.8 - 2.2	8	42	84	22	87	80
	2.3 - 2.7	4	46	92	13	100	92
	2.8 - 3.2	2	48	96	4	104	95
	3.3 - 3.7	2	50	100	5	109	100
	Totals	50	—	—	109	—	—



*F = Frequency distribution of tests

Figure D5-19

SUMMARY OF ZONE 1 EMBANKMENT CONTROL DATA

Date	Volume of Fill Placed cy	Rate of Fill Placement cy/day	Number of Control Tests		Average Moisture (w_o-w_f)				Average Density (D)		E&RC Statiscal Analysis		Average Elevation Top of Zone 1
					Special		Routine						
			Special	Routine	L-29	Control	L-29	Control	Special	Routine	Total (w_o-w_f)	Zone 1 (D)	
Oct 1973	100,000	NA	NA	NA	NA	NA	NA	NA	NA	NA	0.2	99.8	4933-4974
Nov 1973	55,900	11,000	10	26	NA	-1.0	0.2	-0.2	NA	NA	0.2	99.0	4974-4982
Apr 1974	70,000	10,000	29	38	0.5	-0.3	0.35	0.8	NA	NA	0.6	97.5	4982-4995
May 1974	295,000	18,000	21	122	NA	0.1	0.7	0.9	NA	NA	1.1	98.1	4995-5030
July 1974	294,000	25,000	44	162	0.3	-0.3	1.0	0.8	97.3	99.1	0.9	98.7	5030-5050
Aug 1974	443,000	25,000	43	154	0.5	0.35	1.2	1.1	97.2	98.8	1.1	98.5	5050-5077
Sept 1974	437,100	25,000	39	186	0.7	0.7	1.4	1.2	97.0	98.6	1.3	98.3	5077-5104
Oct 1974	364,750	25,000	34	122	0.9	1.0	1.3	1.0	97.4	99.6	1.2	98.5	5104-5124
Nov 1974	249,500	20,000	22	68	1.2	0.95	0.7	1.1	97.2	98.6	1.4	98.2	5124-5174
May 1975	190,600	41,000	24	101	0.1	-0.6	1.3	1.2	98.1	98.0	NA	NA	5115-5140
June 1975	556,900	41,000	50	148	0.5	0.4	0.9	0.6	96.8	97.7	0.7	98.0	5140-5170
July 1975	583,700	40,000	70	250	0.3	-0.2	1.0	0.8	94.7	96.3	3.1	97.7	5170-5200
Aug 1975	729,600	35,000	70	314	0.6	0.65	1.2	0.95	97.0	98.8	1.1	98.2	5200-5242
Sept 1975	577,200	25,000	51	291	0.8	0.85	1.5	1.3	96.6	97.9	1.3	97.7	5242-5298
Oct 1975	348,760	20,000	50	110	0.8	0.8	1.3	1.05	98.4	99.0	1.1	98.2	5298-5330
		Total	557	2012									

Note:
NA = Not available
L-29 = Monthly Construction Reports
Control = Based on Earthwork Control Data

D = Percentage of maximum standard compacted density
Special = Control testa on specially compacted earthfill
Routine = Control tests on zone 1 fill not including specially compacted earthfill
w_o - w_f = Moisture content at optimum minus compacted field moisture
E&RC = Engineering and Research Center, USBR, Denver, Colorado

Figure D5-20

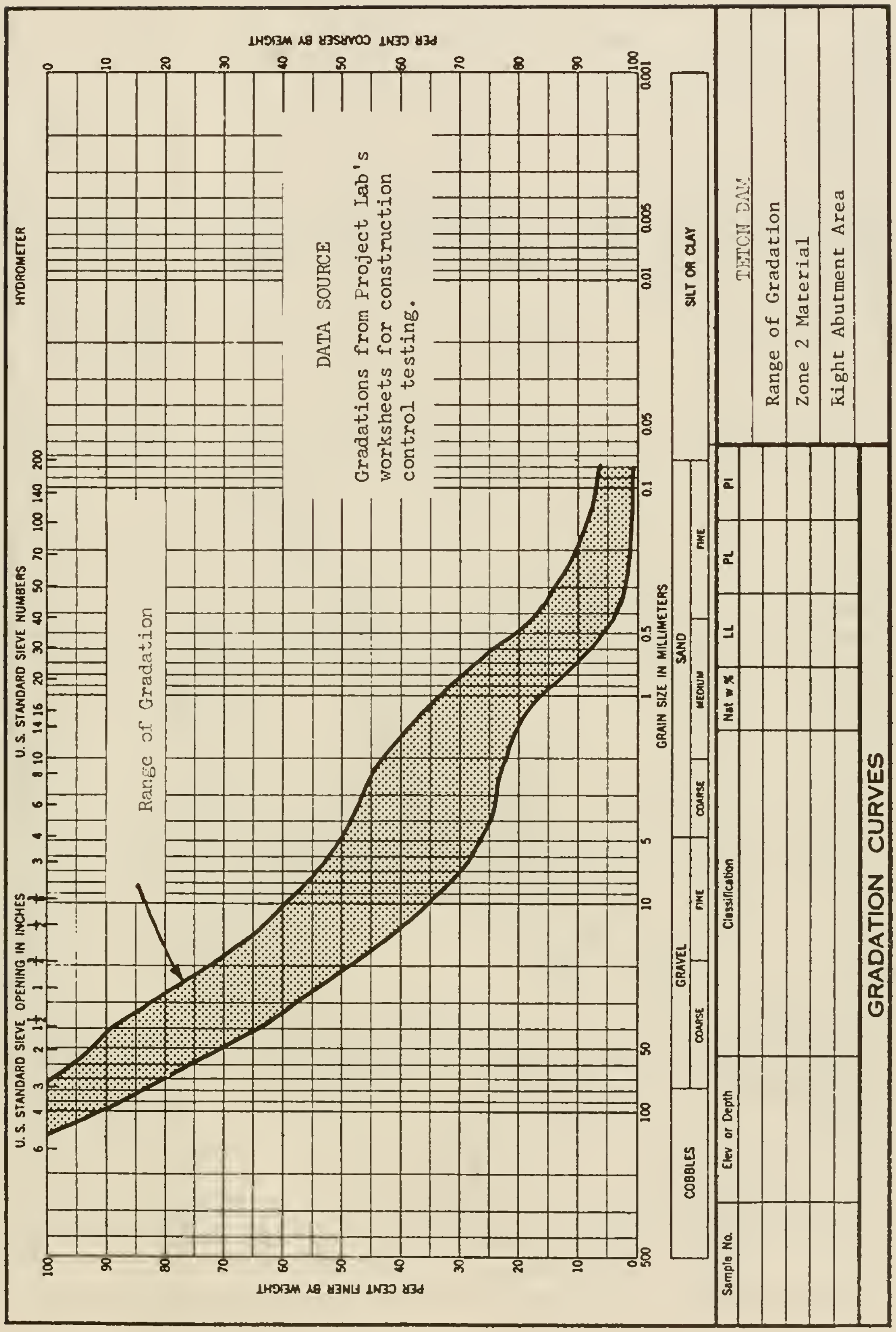
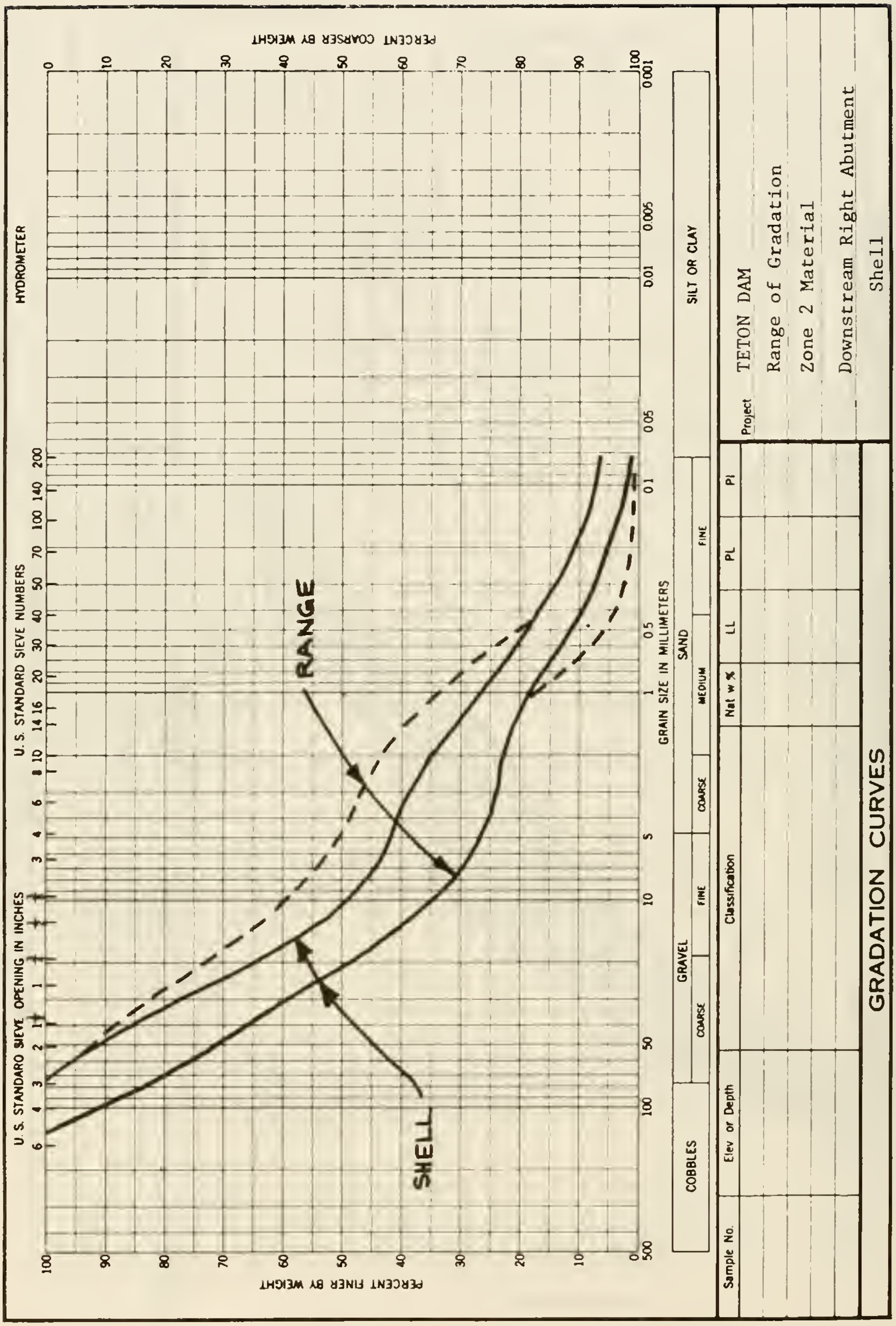

GRADATION CURVES
TETON DAM
Range of Gradation
Zone 2 Material
Right Abutment Area
DATA SOURCE
Gradations from Project Lab's worksheets for construction control testing.
Range of Gradation
U. S. STANDARD SIEVE OPENING IN INCHES
U. S. STANDARD SIEVE NUMBERS
HYDROMETER
PER CENT FINER BY WEIGHT
PER CENT COARSER BY WEIGHT
GRAIN SIZE IN MILLIMETERS
COBBLES
GRAVEL
COARSE
FINE
SAND
COARSE
MEDIUM
FINE
SILT OR CLAY
Sample No.
Elev or Depth
Classification
Nat w %
LL
PL
PI

Figure D5-21



U. S. STANDARD SIEVE OPENING IN INCHES
U. S. STANDARD SIEVE NUMBERS
HYDROMETER
PERCENT FINER BY WEIGHT
PERCENT COARSER BY WEIGHT
GRAIN SIZE IN MILLIMETERS
RANGE
SHELL
COBBLES
GRAVEL
COARSE
FINE
SAND
COARSE
MEDIUM
FINE
SILT OR CLAY
Sample No.
Elev or Depth
Classification
Nat w %
LL
PL
PI
Project
TETON DAM
Range of Gradation
Zone 2 Material
Downstream Right Abutment
Shell
GRADATION CURVES

Figure D5-22

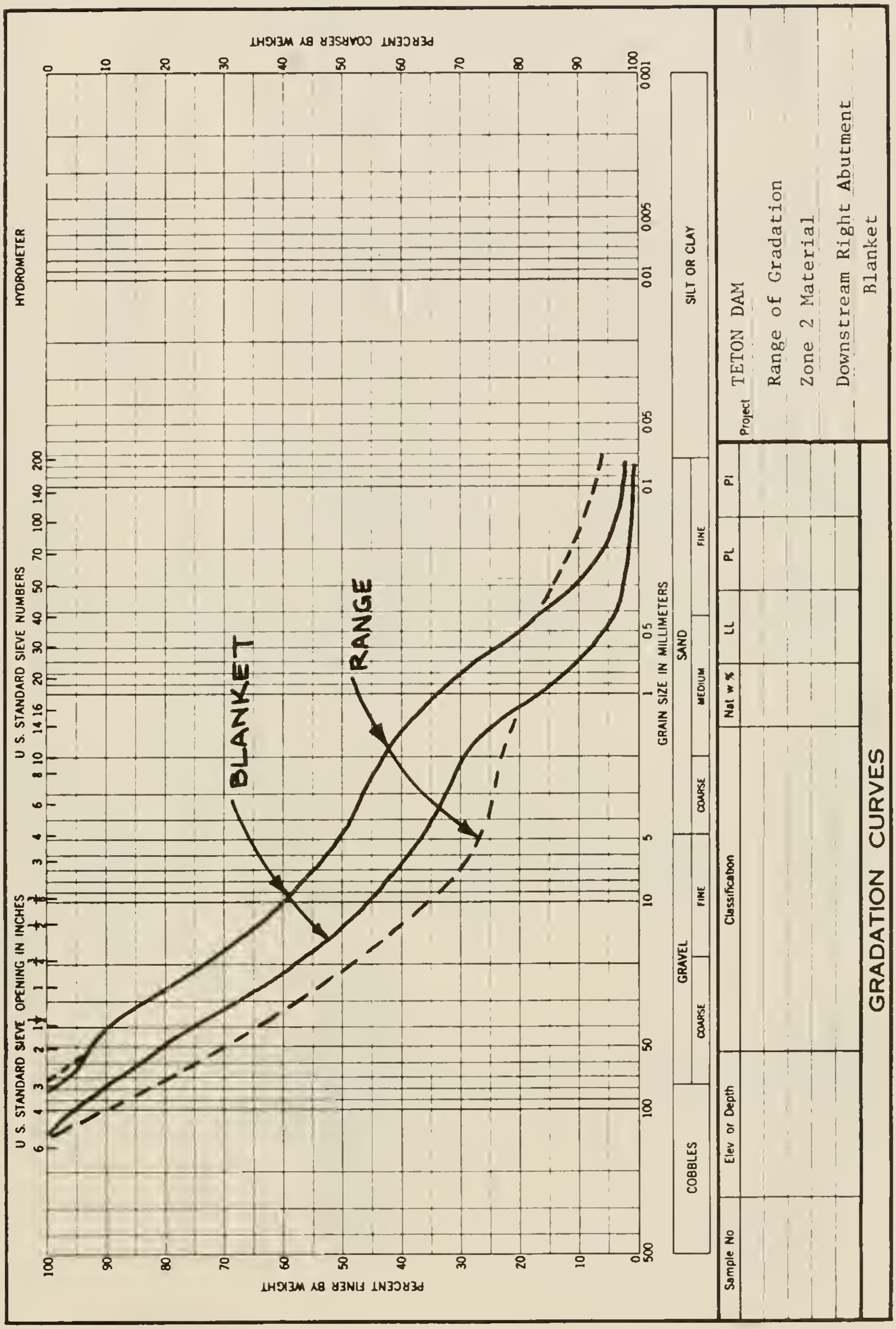

GRADATION CURVES
Project TETON DAM
Range of Gradation
Zone 2 Material
Downstream Right Abutment
Blanket
BLANKET
RANGE
HYDROMETER
U. S. STANDARD SIEVE NUMBERS
U. S. STANDARD SIEVE OPENING IN INCHES
PERCENT FINER BY WEIGHT
PERCENT COARSER BY WEIGHT
GRAIN SIZE IN MILLIMETERS
COBBLES
GRAVEL
SAND
SILT OR CLAY
Sample No
Elev or Depth
Classification
Nat w %
LL
PL
PI

Figure D5-23

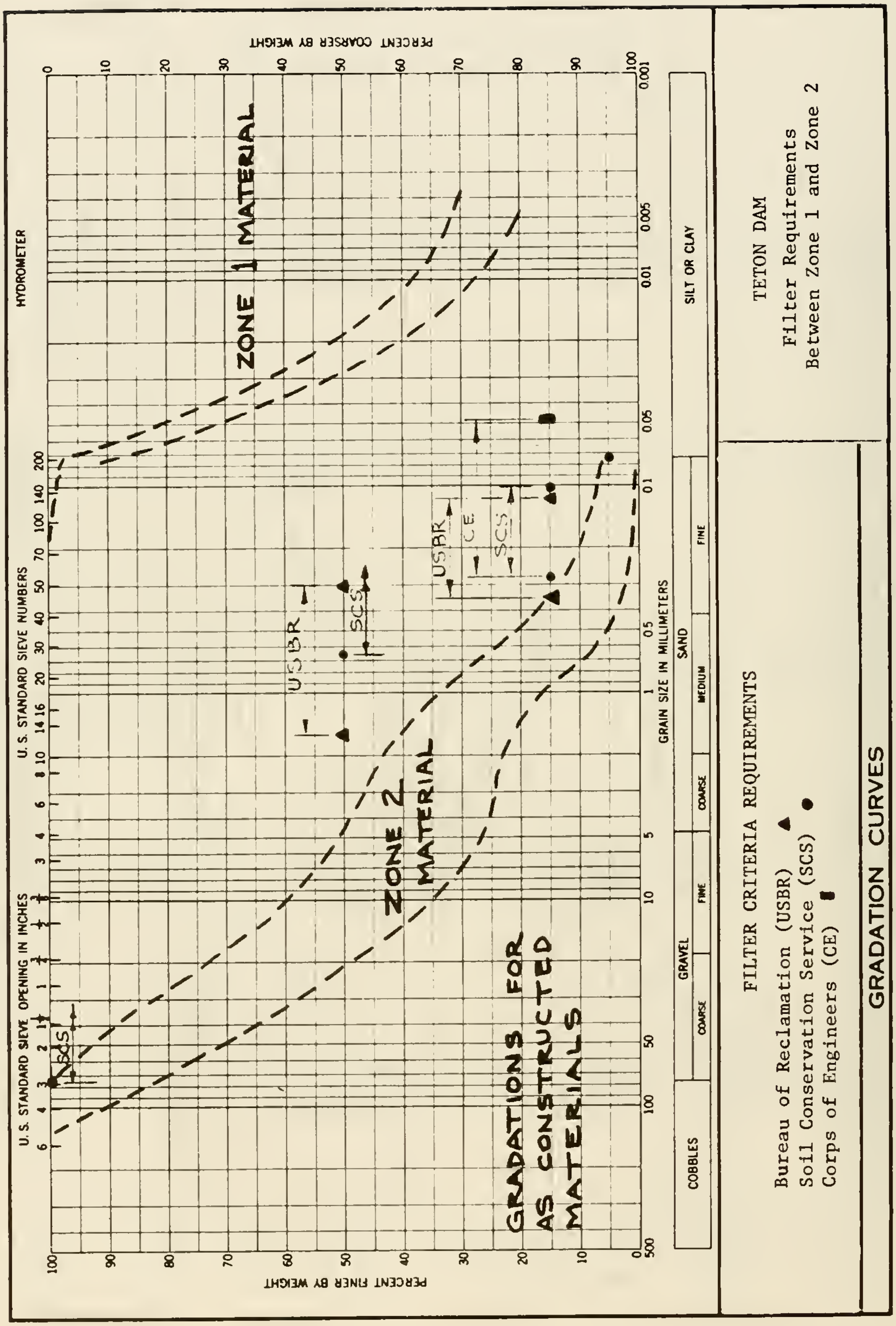
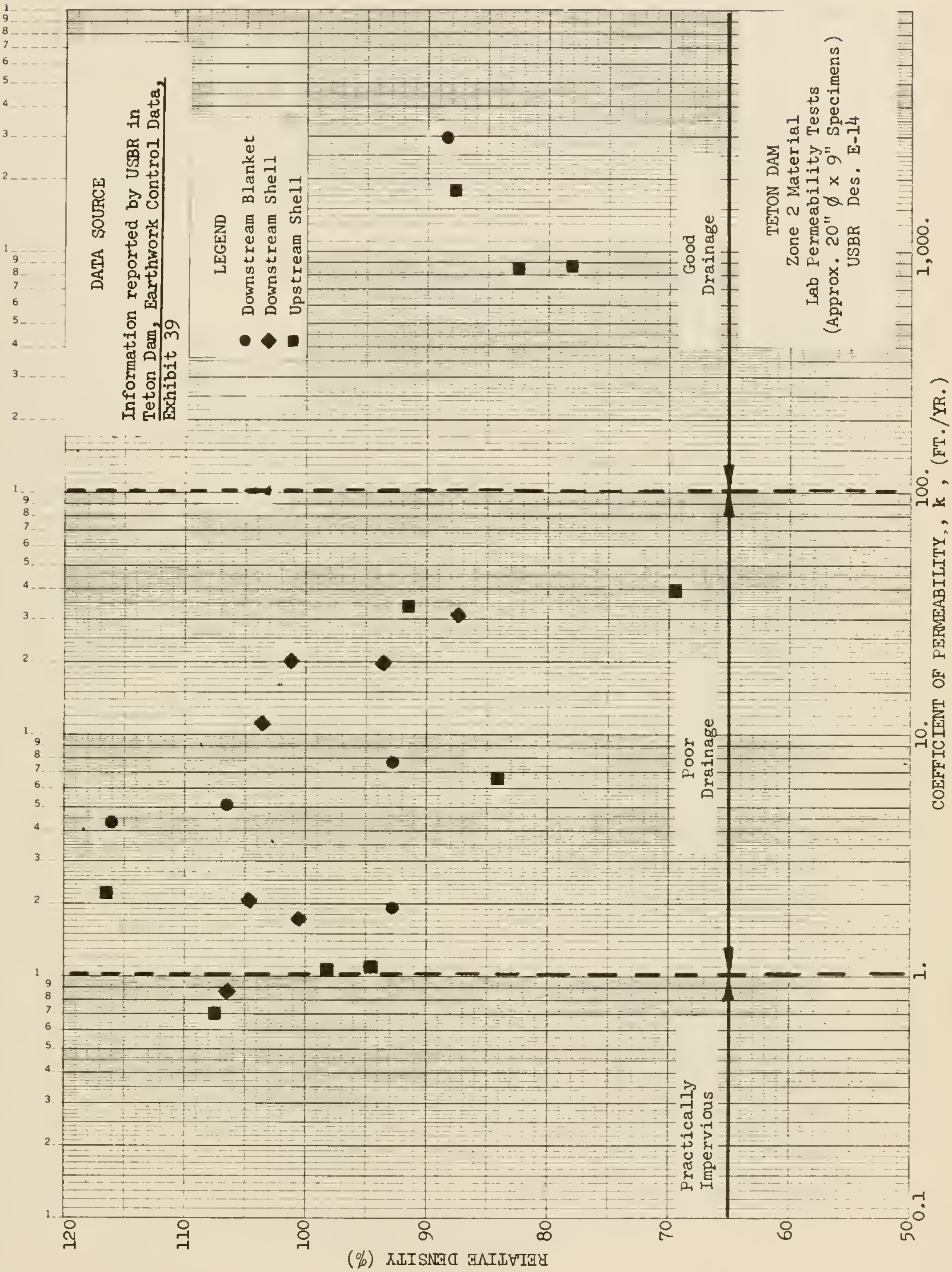

TETON DAM
Filter Requirements
Between Zone 1 and Zone 2
FILTER CRITERIA REQUIREMENTS
Bureau of Reclamation (USBR)
Soil Conservation Service (SCS)
Corps of Engineers (CE)
GRADATION CURVES
ZONE 1 MATERIAL
ZONE 2 MATERIAL
GRADATIONS FOR AS CONSTRUCTED MATERIALS
U. S. STANDARD SIEVE OPENING IN INCHES
U. S. STANDARD SIEVE NUMBERS
HYDROMETER
PERCENT FINER BY WEIGHT
PERCENT COARSER BY WEIGHT
GRAIN SIZE IN MILLIMETERS
COBBLES
GRAVEL
COARSE
FINE
SAND
COARSE
MEDIUM
FINE
SILT OR CLAY
USBR
SCS
CE

Figure D5-24



DATA SOURCE
Information reported by USBR in Teton Dam, Earthwork Control Data, Exhibit 39
LEGEND
Downstream Blanket
Downstream Shell
Upstream Shell
Good Drainage
Poor Drainage
Practically Impervious
TETON DAM
Zone 2 Material
Lab Permeability Tests
(Approx. 20" Ø x 9" Specimens)
USBR Des. E-14
0.1
1.
10.
100.
1,000.
COEFFICIENT OF PERMEABILITY,, k , (FT./YR.)
120
110
100
90
80
70
60
50
RELATIVE DENSITY (%)

Figure D5-25

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

Project Teton Dam Feature Zone 1

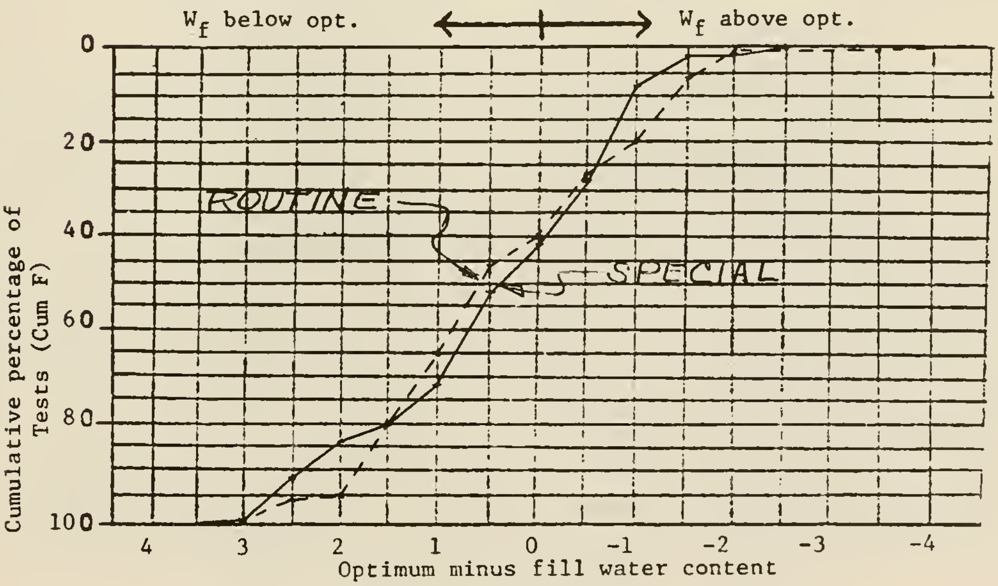
Period 6-1-75 To 6-30-75

Fill Quantity: 556,900 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content (W_o - W_f) in precent of dry weight

		SPECIAL			ROUTINE		
		F*	Cum F	Cum %	F*	Cum F	Cum %
W_f is above opt.	4.2 - 3.8						0
	3.7 - 3.3				2	2	1
	3.2 - 2.8					2	1
	2.7 - 2.3			0		2	1
	2.2 - 1.8	1	1	2		2	1
	1.7 - 1.3		1	2	6	8	6
	1.2 - 0.8	3	4	8	20	28	20
	0.7 - 0.2	10	14	28	11	39	27
+0.2 to -0.2		7	21	42	18	57	40
W_f is below opt.	0.3 - 0.7	5	26	52	8	65	46
	0.8 - 1.2	10	36	72	28	93	65
	1.3 - 1.7	4	40	80	20	113	80
	1.8 - 2.2	2	42	84	22	135	95
	2.3 - 2.7	4	46	92	2	137	96
	2.8 - 3.2	3	49	99	3	140	99
	3.3 - 3.7	1	50	100	2	142	100
	Totals	50	—	—	142	—	—



*F = Frequency distribution of tests

Figure D5-15

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

Project Teton Dam Feature Zone 1

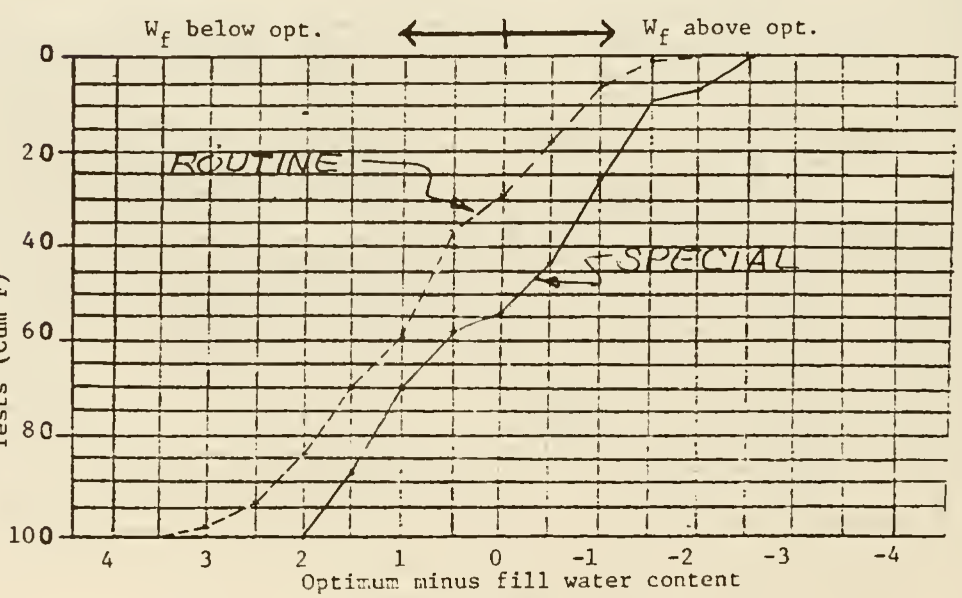
Period 7-1-75 To 7-31-75

Fill Quantity: 583,700 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content ($W_o - W_f$) in percent of dry weight

		SPECIAL			ROUTINE		
		F*	Cum F	Cum %	F*	Cum F	Cum %
W_f is above opt.	4.2 - 3.8						
	3.7 - 3.3						
	3.2 - 2.8				1	1	0
	2.7 - 2.3			0		1	0
	2.2 - 1.8	5	5	7		1	0
	1.7 - 1.3	1	6	9	1	2	1
	1.2 - 0.8	12	18	26	14	16	6
	0.7 - 0.2	12	30	43	30	46	18
+0.2 to -0.2		7	37	54	27	73	29
W_f is below opt.	0.3 - 0.7	3	40	58	21	94	37
	0.8 - 1.2	8	48	70	57	151	59
	1.3 - 1.7	13	61	88	28	179	70
	1.8 - 2.2	8	69	100	34	213	84
	2.3 - 2.7				25	238	94
	2.8 - 3.2				11	249	98
	3.3 - 3.7				5	254	100
	Totals	69	—	—	254	—	—



*F = Frequency distribution of tests

Figure D5-16

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

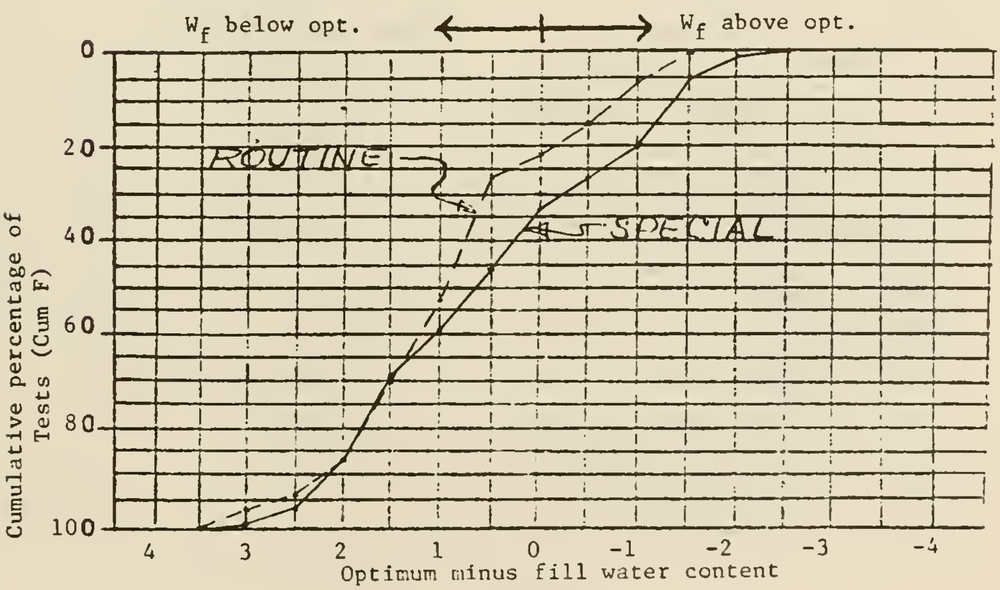
Project Teton Dam Feature Zone 1

Period 8-1-75 To 8-31-75

Fill Quantity: 729,600 yd^3

Optimum minus fill water content ($W_o - W_f$) in precent of dry weight

		COMPACTED EMBANKMENT					
		SPECIAL			ROUTINE		
		F*	Cum F	Cum %	F*	Cum F	Cum %
W_f is above opt.	4.2 - 3.8						
	3.7 - 3.3						
	3.2 - 2.8						
	2.7 - 2.3			0			
	2.2 - 1.8	1	1	1			0
	1.7 - 1.3	2	3	5	1	1	0
	1.2 - 0.8	11	14	20	16	17	6
	0.7 - 0.2	5	19	27	27	44	15
+0.2 to -0.2		4	23	33	20	64	22
W_f is below opt.	0.3 - 0.7	9	32	46	17	81	27
	0.8 - 1.2	9	41	59	76	157	53
	1.3 - 1.7	7	48	69	50	207	70
	1.8 - 2.2	13	61	87	50	257	87
	2.3 - 2.7	7	68	97	22	279	94
	2.8 - 3.2	1	69	99	9	288	97
	3.3 - 3.7	1	70	100	9	297	100
	Totals	70	—	—	297	—	—



*F = Frequency distribution of tests

Figure D5-17

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

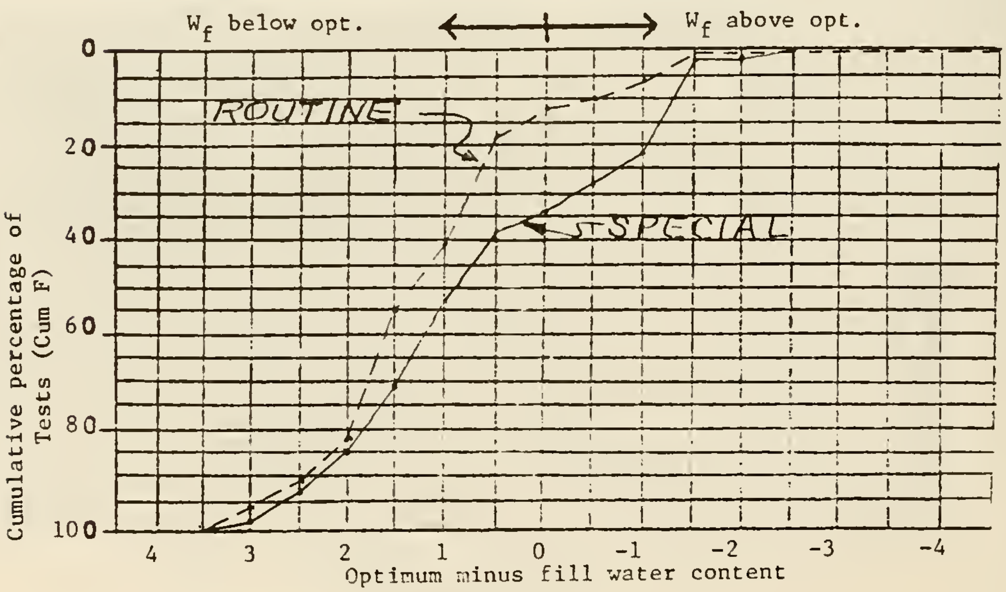
Project Teton Dam   Feature Zone 1

Period 9-1-75 To 9-30-75

Fill Quantity: 577,200 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content ($W_o - W_f$) in percent of dry weight		SPECIAL			ROUTINE		
		F*	Cum F	Cum %	F*	Cum F	Cum %
W_f is above opt.	4.2 - 3.8				1	2	1
	3.7 - 3.3					2	1
	3.2 - 2.8					2	1
	2.7 - 2.3			0	1	3	1
	2.2 - 1.8	1	1	2		3	1
	1.7 - 1.3		1	2		3	1
	1.2 - 0.8	10	11	22	15	18	6
	0.7 - 0.2	3	14	28	11	29	10
+0.2 to -0.2		3	17	34	6	35	12
W_f is below opt.	0.3 - 0.7	2	19	38	17	52	18
	0.8 - 1.2	8	27	53	66	118	41
	1.3 - 1.7	9	36	71	41	159	55
	1.8 - 2.2	7	43	85	78	237	82
	2.3 - 2.7	4	47	93	26	263	91
	2.8 - 3.2	3	50	98	16	279	96
	3.3 - 3.7	1	51	100	11	290	100
	Totals	51	—	—	290	—	—



*F = Frequency distribution of tests

Figure D5-18

STATISTICAL ANALYSIS OF CONSTRUCTION MOISTURE TEST

RESULTS FOR ZONE 1 COMPACTED EMBANKMENT MATERIALS

Project Teton Dam Feature Zone 1

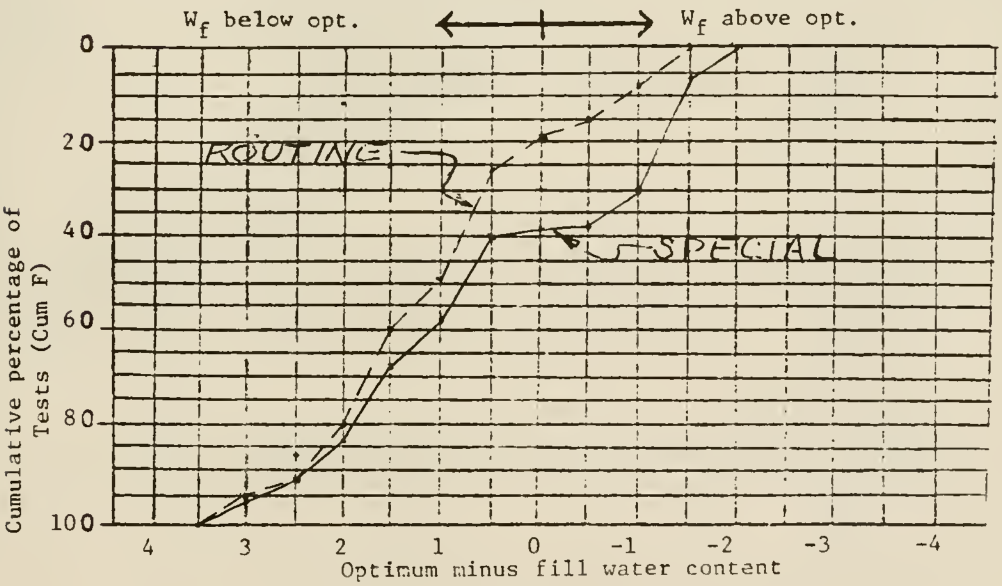
Period 10-1-75 To 10-31-75

Fill Quantity: 348,760 yd^3

COMPACTED EMBANKMENT

Optimum minus fill water content ($W_o - W_f$) in precent of dry weight

		SPECIAL			ROUTINE		
		F *	Cum F	Cum %	F *	Cum F	Cum %
W_f is above opt.	4.2 - 3.8						
	3.7 - 3.3						
	3.2 - 2.8						
	2.7 - 2.3						
	2.2 - 1.8			0			
	1.7 - 1.3	3	3	6	0		0
	1.2 - 0.8	12	15	30	9	9	8
	0.7 - 0.2	4	19	38	7	16	15
+0.2 to -0.2			19	38	5	21	19
W_f is below opt.	0.3 - 0.7	1	20	40	7	28	26
	0.8 - 1.2	9	29	58	25	53	49
	1.3 - 1.7	5	34	68	12	65	60
	1.8 - 2.2	8	42	84	22	87	80
	2.3 - 2.7	4	46	92	13	100	92
	2.8 - 3.2	2	48	96	4	104	95
	3.3 - 3.7	2	50	100	5	109	100
	Totals	50	—	—	109	—	—



*F = Frequency distribution of tests

Figure D5-19

SUMMARY OF ZONE 1 EMBANKMENT CONTROL DATA

Date	Volume of Fill Placed cy	Rate of Fill Placement cy/day	Number of Control Tests		Average Moisture (w_o-w_f)				Average Density (D)		E&RC Statiscal Analysis		Average Elevation Top of Zone 1
			Special	Routine	Special L-29	Special Control	Routine L-29	Routine Control	Special	Routine	Total (w_o-w_f)	Zone 1 (D)	
Oct 1973	100,000	NA	NA	NA	NA	NA	NA	NA	NA	NA	0.2	99.8	4933-4974
Nov 1973	55,900	11,000	10	26	NA	-1.0	0.2	-0.2	NA	NA	0.2	99.0	4974-4982
Apr 1974	70,000	10,000	29	38	0.5	-0.3	0.35	0.8	NA	NA	0.6	97.5	4982-4995
May 1974	295,000	18,000	21	122	NA	0.1	0.7	0.9	NA	NA	1.1	98.1	4995-5030
July 1974	294,000	25,000	44	162	0.3	-0.3	1.0	0.8	97.3	99.1	0.9	98.7	5030-5050
Aug 1974	443,000	25,000	43	154	0.5	0.35	1.2	1.1	97.2	98.8	1.1	98.5	5050-5077
Sept 1974	437,100	25,000	39	186	0.7	0.7	1.4	1.2	97.0	98.6	1.3	98.3	5077-5104
Oct 1974	364,750	25,000	34	122	0.9	1.0	1.3	1.0	97.4	99.6	1.2	98.5	5104-5124
Nov 1974	249,500	20,000	22	68	1.2	0.95	0.7	1.1	97.2	98.6	1.4	98.2	5124-5174
May 1975	190,600	41,000	24	101	0.1	-0.6	1.3	1.2	98.1	98.0	NA	NA	5115-5140
June 1975	556,900	41,000	50	148	0.5	0.4	0.9	0.6	96.8	97.7	0.7	98.0	5140-5170
July 1975	583,700	40,000	70	250	0.3	-0.2	1.0	0.8	94.7	96.3	3.1	97.7	5170-5200
Aug 1975	729,600	35,000	70	314	0.6	0.65	1.2	0.95	97.0	98.8	1.1	98.2	5200-5242
Sept 1975	577,200	25,000	51	291	0.8	0.85	1.5	1.3	96.6	97.9	1.3	97.7	5242-5298
Oct 1975	348,760	20,000	50	110	0.8	0.8	1.3	1.05	98.4	99.0	1.1	98.2	5298-5330
		Total	557	2012									

Note:
NA = Not available
L-29 = Monthly Construction Reports
Control = Based on Earthwork Control Data

D = Percentage of maximum standard compacted density
Special = Control tests on specially compacted earthfill
Routine = Control tests on zone 1 fill not including specially compacted earthfill
w_o - w_f = Moisture content at optimum minus compacted field moisture
E&RC = Engineering and Research Center, USBR, Denver, Colorado

Figure D5-20

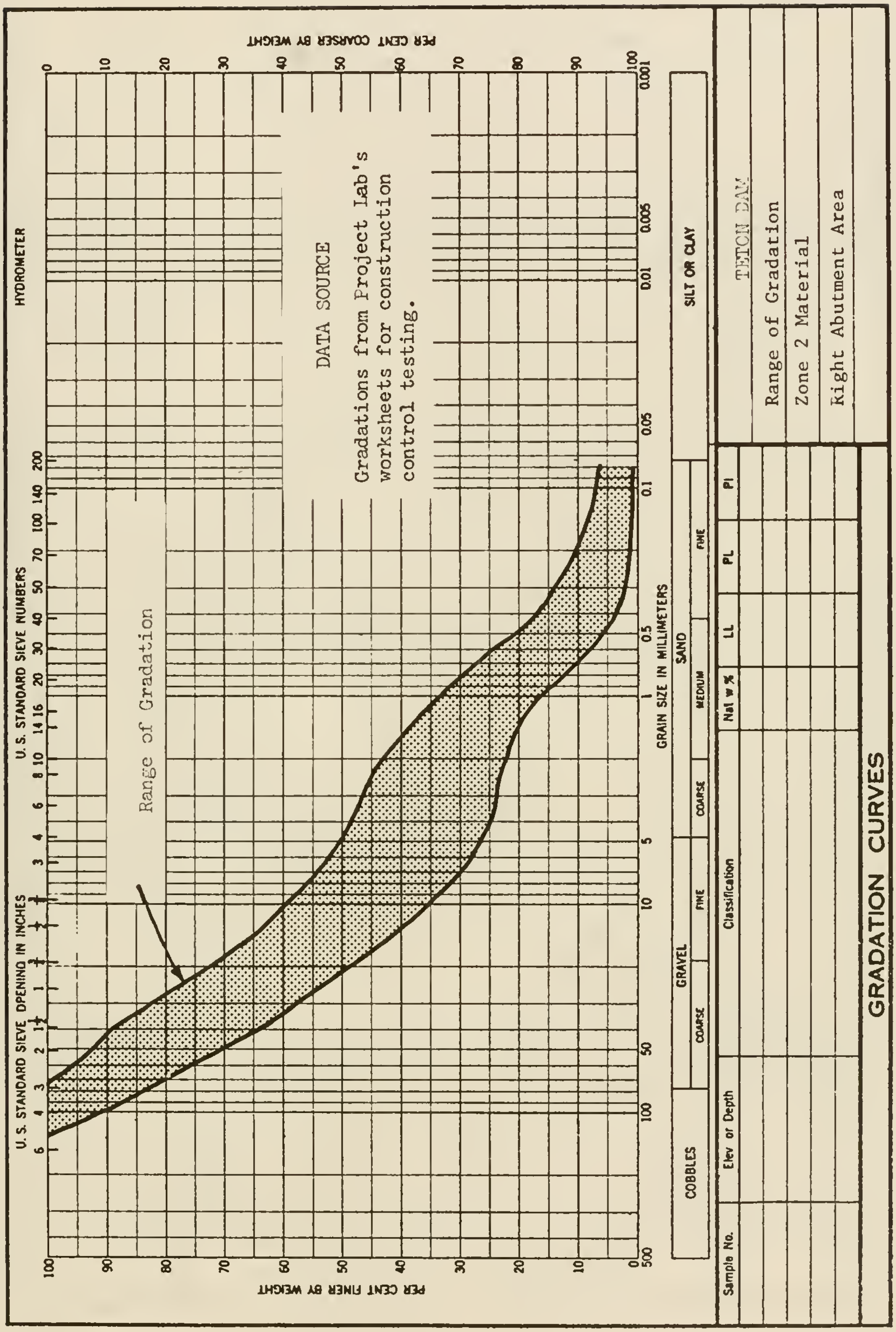
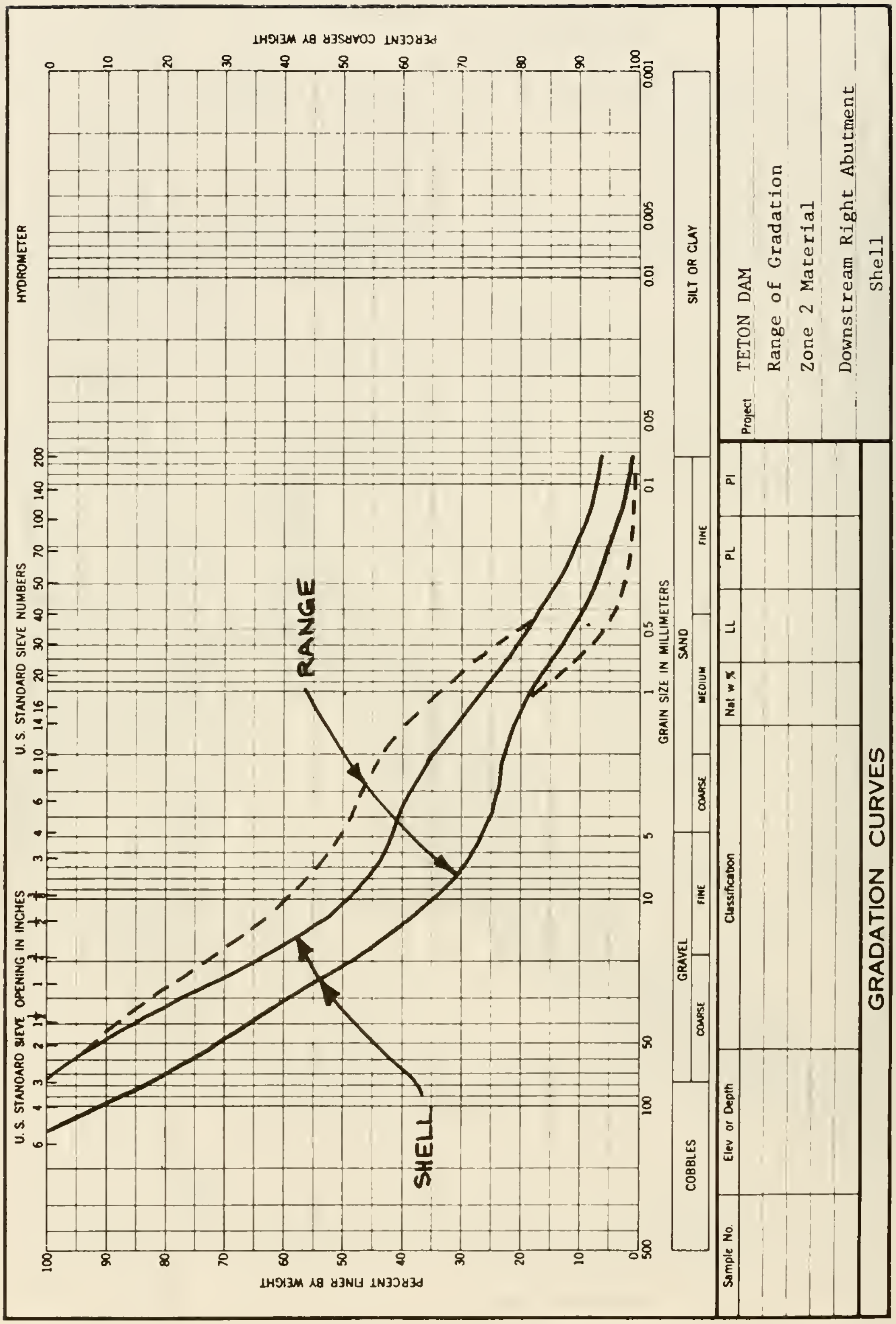

U. S. STANDARD SIEVE OPENING IN INCHES
U. S. STANDARD SIEVE NUMBERS
HYDROMETER
PER CENT COARSER BY WEIGHT
PER CENT FINER BY WEIGHT
GRAIN SIZE IN MILLIMETERS
COBBLES
GRAVEL
COARSE
FINE
SAND
COARSE
MEDIUM
FINE
SILT OR CLAY
DATA SOURCE
Gradations from Project Lab's worksheets for construction control testing.
Range of Gradation
TETON DAM
Range of Gradation
Zone 2 Material
Right Abutment Area
Sample No.
Elev or Depth
Classification
Nat w %
LL
PL
PI
GRADATION CURVES

Figure D5-21



U. S. STANDARD SIEVE OPENING IN INCHES
U. S. STANDARD SIEVE NUMBERS
HYDROMETER
PERCENT FINER BY WEIGHT
PERCENT COARSER BY WEIGHT
GRAIN SIZE IN MILLIMETERS
RANGE
SHELL
COBBLES
GRAVEL
COARSE
FINE
SAND
COARSE
MEDIUM
FINE
SILT OR CLAY
Sample No.
Elev or Depth
Classification
Nat w %
LL
PL
PI
Project
TETON DAM
Range of Gradation
Zone 2 Material
Downstream Right Abutment
Shell
GRADATION CURVES

Figure D5-22

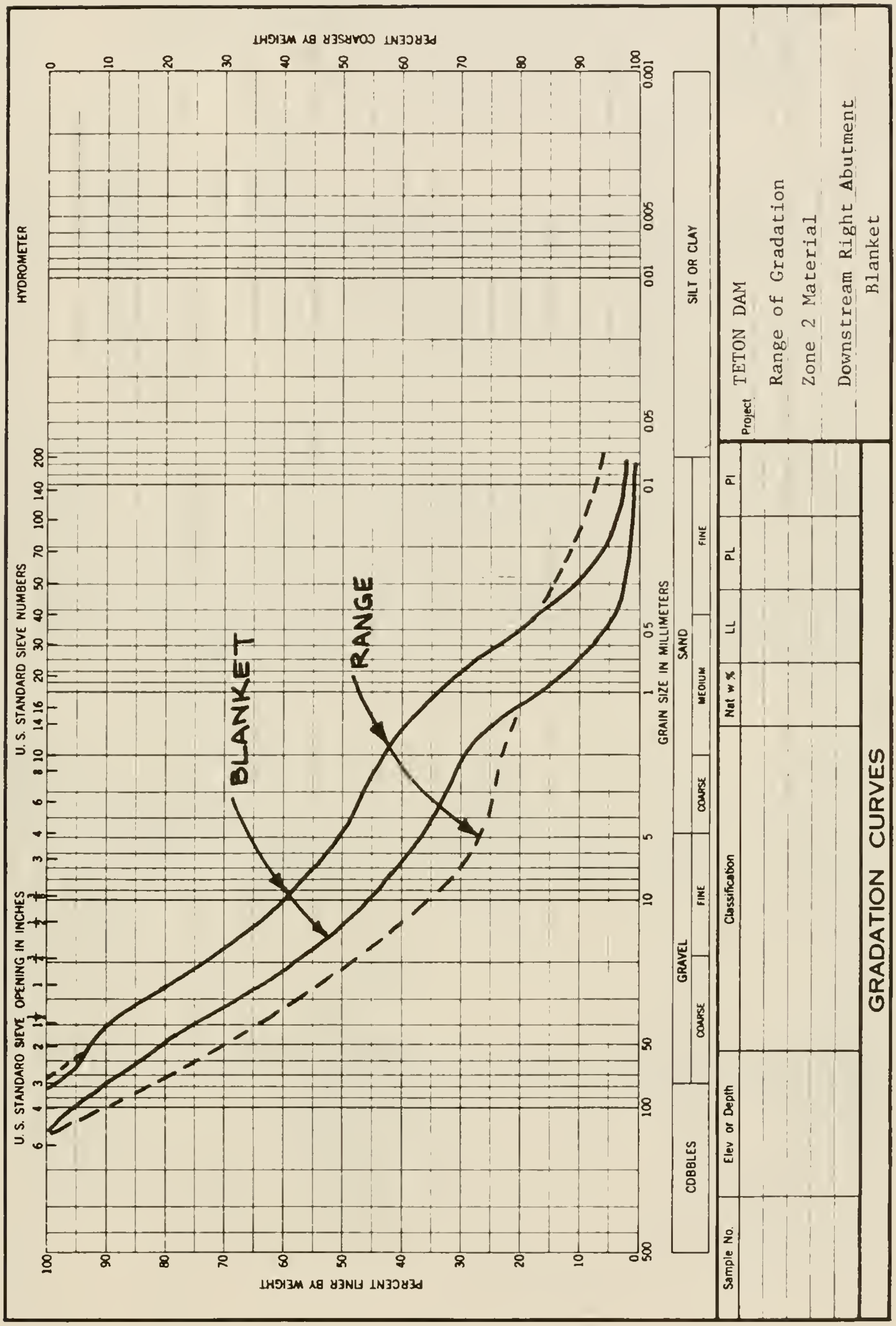
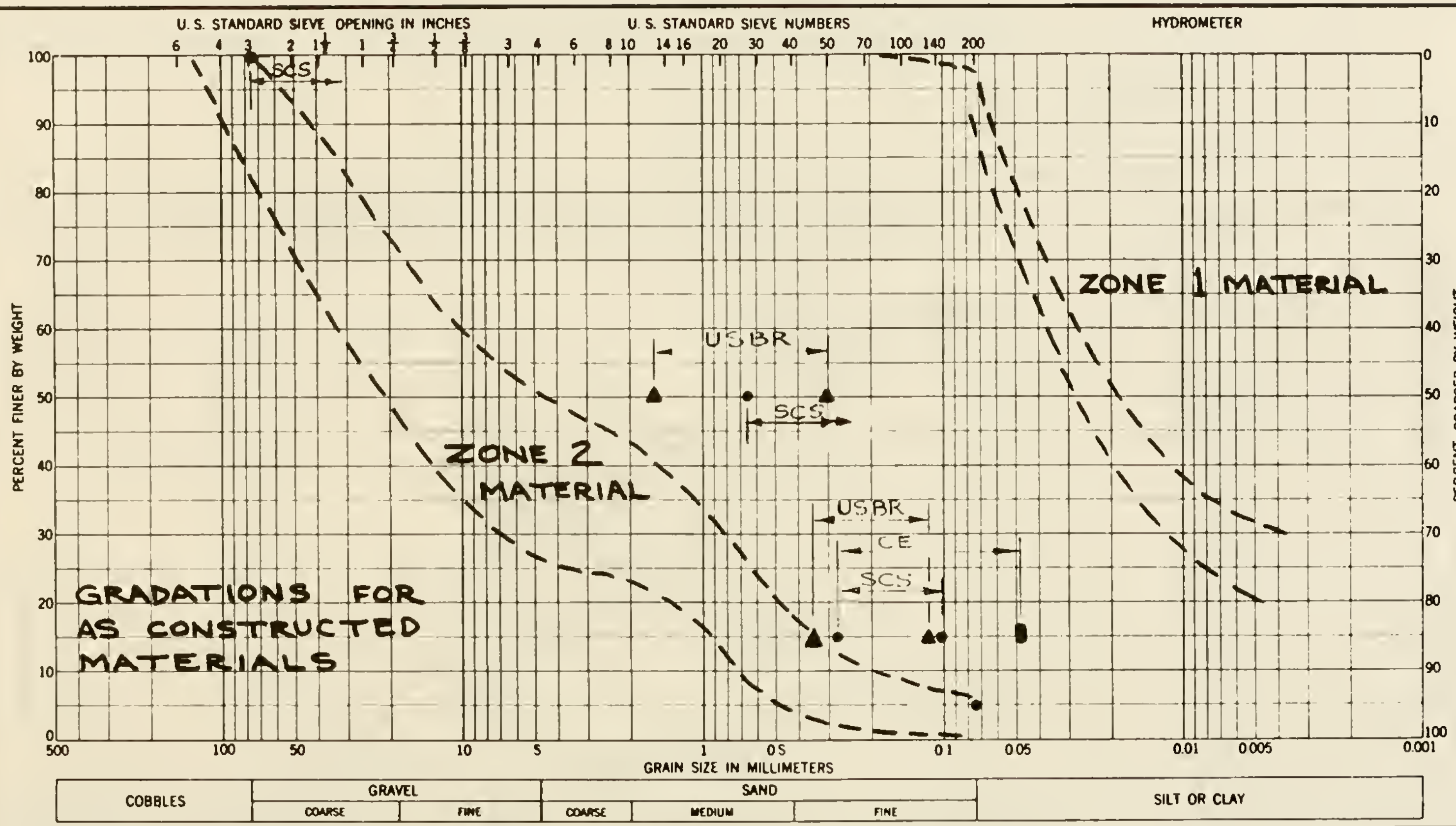

U. S. STANDARD SIEVE OPENING IN INCHES
U. S. STANDARD SIEVE NUMBERS
HYDROMETER
PERCENT FINER BY WEIGHT
PERCENT COARSER BY WEIGHT
GRAIN SIZE IN MILLIMETERS
BLANKET
RANGE
COBBLES
GRAVEL
COARSE
FINE
SAND
COARSE
MEDIUM
FINE
SILT OR CLAY
Sample No.
Elev or Depth
Classification
Nat w %
LL
PL
PI
Project TETON DAM
Range of Gradation
Zone 2 Material
Downstream Right Abutment
Blanket
GRADATION CURVES

Figure D5-23



FILTER CRITERIA REQUIREMENTS

Bureau of Reclamation (USBR) ▲
Soil Conservation Service (SCS) ●
Corps of Engineers (CE) ▮

GRADATION CURVES

TETON DAM

Filter Requirements
Between Zone 1 and Zone 2

Figure D5-24

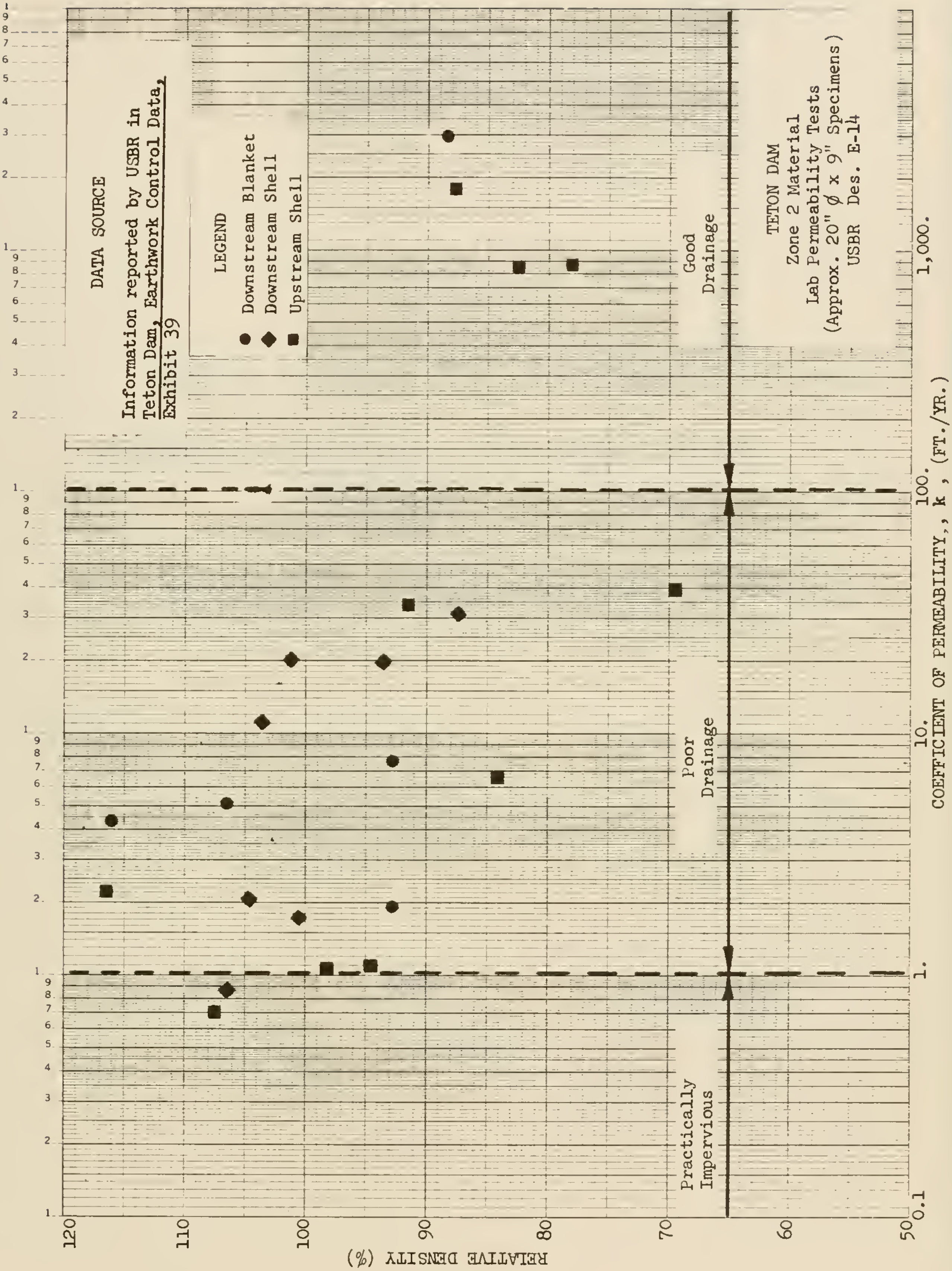

DATA SOURCE
Information reported by USBR in Teton Dam, Earthwork Control Data, Exhibit 39
LEGEND
● Downstream Blanket
◆ Downstream Shell
■ Upstream Shell
Good Drainage
Poor Drainage
Practically Impervious
TETON DAM
Zone 2 Material
Lab Permeability Tests
(Approx. 20" ∅ x 9" Specimens)
USBR Des. E-14
COEFFICIENT OF PERMEABILITY,, k , (FT./YR.)
0.1
1.
10.
100.
1,000.
RELATIVE DENSITY (%)
120
110
100
90
80
70
60
50

Figure D5-25

List of the Teton Dam Failure Exhibits Furnished to the IRG by the Bureau of Reclamation

(Updated to December 12, 1976)

As of July 8, 1976
(Items 1 through 35)
(Updated to December 12,
1976 – through 57)

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 ±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^{o} by 2^{o} 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
1.25 Seismic monitoring program–Teton Dam and Reservoir (including memorandum from Acting Chief Geologist Robert C. Davis to D. J. Duck, dated early 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

No.

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"
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

No.

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

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

No.

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

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 L-29 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 L-29 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

23.1 Analysis of Eye Witnesses to Teton Dam Failure, June 5, 1976, dated July 2, 1976 plus three more accounts

No.

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 and 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

(Added Subsequent to July 8, 1976)

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

(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

No.

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

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

57. Teton Dam Project Organization for Earthwork Construction Inspection (with resume of work experience of most of the individuals listed)

Summary of Eyewitness Accounts

Reproduced from Appendix C of the Independent Panel Report "Teton Dam Failure," December 1976

APPENDIX C

WITNESS ACCOUNTS OF FAILURE

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

INTERVIEW GUIDELINE QUESTIONS

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 in 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 visable?

Any tremors earlier?

Check inspection route on previous shifts.

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

COPY

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.

wha

I drove back upstream ~~to~~ toward 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~~t~~. 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. to 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 ~~two~~ one dozer~~s~~ RRB 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

Dick R. Berry

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, Charles L. Entwisle, 440 N. Seventh W., 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 ~~[illegible]~~ 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

Charles L. Entwisle

Subscribed and sworn to before
me on this 21st 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, Clifford Felkins , ~~430 N. 3rd W., Rigby, Idaho~~

, being duly

sworn make the following voluntary statement to Betty J. Foyes ,

who has identified ~~him~~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 pick up 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

Clifford Felkins

Subscribed and sworn to before me this 22nd day of June 1976.

Betty J. Foyes

Betty J. Foyes, Special Agent
U. S. Department of the Interior

STATE OF)
) SS
COUNTY OF)

I, Myra A. Ferber, Box 124, 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 Survey Technician, GS-4, Teton Dam Project, Bureau of Reclamation, Newdale, Idaho and have held this position for one and one-half years. I previously worked at the Project as a secretary for one and one-half years.

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. ~~[illegible]~~ HARRY Parks, Richard Berry, Clifford Felkins, all surveyors, and myself, proceeded downstream from the dam on the south or left side canyon wall to check sitings in preparation for survey work on the spillway on the north or right side of the dam. While checking the sitings 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 proceeded to the office~~ and reported the leak to Jan Ringel.

We then proceeded across the dam to do our survey work. 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 there we started survey work on the left wall of the spillway and I was unable to observe the leak in the dam. At about 10:15 a.m. we finished surveying the left wall and went up to the top of the spillway. At this time I noticed that the leak in the dam had opened to about 10 to 15 feet in diameter. The water was turbid and flowing fast, but I cannot estimate the volume.

At about 10:45 a.m. to 11:00 a.m. we prepared to survey the right wall of the spillway. Before leaving the position of being able to see the face of the dam I noticed two dozers were going down the downstream face toward the hole.

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~~ heard several people trying to get the attention of the dozer operators and get them off the dam. ~~dozers, which had [illegible] been working in that area, start to head across the top of the dam from the north side toward the south 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

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, 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.

AJH.

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, but did ~~to~~ assist ~~in~~ 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

Alvin J. Heintz

Subscribed and sworn to before me on 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, 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

Kenneth C. Hoyt

Subscribed and sworn to before me
on 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, 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 to me as ~~a~~ 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

Harry Parks

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

Betty J. Foyes, Special Agent

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 Ringel
Jan R. Ringel

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, Robert R. Robison, 581 Taurus Drive, Rexburg, Idaho

, being duly

sworn make the following voluntary statement to Vincent L. Duran and Betty J. Foyes who have identified themselves to me as Special Agents 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.

RRR

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.

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 15 to 20 feet from the right abutment wall~~. The whirlpool was approximately $1\frac{1}{2}$ 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.

ABS

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

STATE OF IDAHO

ss

COUNTY OF MADISON

I, Stephen Elenberger, , Victor, Idaho,

______________________, being duly sworn make the following voluntary statement to Vincent L. Duran, who has identified himsel 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 a Construction Inspector, GS-7, Teton Dam Project, Bureau of Reclamation, Newdale, Idaho for ABOUT four and one-half years and have a total of ~~nine~~ EIGHT years with the Bureau.

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 signs 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.

I was not at the dam on Saturday, June 5, 1976, and can furnish no information about the events of that day.

I have carefully read the foregoing statement, consisting of 1 page, and declare it to be true and correct.

Stephen Elenberger

Stephen Elenberger

Subscribed and sworn to before me on this 21st 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 280 Wilson Drive Idaho Falls, Idaho	524-1396
Perry Ogden, Mechanic Rexburg, Idaho	356-7920
Lynn Walker, Superintendent Behind June's Bar Teton	458-4304

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

COPY

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, David Burch, P.O. Box 384

Ashton, Idaho, being duly

sworn make the following voluntary statement to Betty J. Foyes,

who has identified ~~himself~~ 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 mechanic with Gibbons and Reed, the contractor who is building the irrigation canals at the Teton Dam Project, Newdale, Idaho. I have been employed by Gibbons and Reed since May 30, 1976, and was formerly employed by Morrison-Knudsen and Kiewit on the Teton Dam Project in the same capacity for almost four years.

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. We were not concerned at that time that there was any real problem and we went on with our work at the G-R trailer.

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.

At approximately 10:30 we went down in the canyon to the beaver slide to get our equipment--a scraper and a D-8 cat. There was another D-8, cat in the field south of the project parking lot on the canal and we also brought that to the dam. We put the 2 D-8 cats to work on the upstream face of the dam driving them from the south to the north side. This was about 11:00 a.m. when we got on top of the dam.

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 dozdr operators were pusing 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 988 cat loader to laod 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 slpe 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 soutside 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 dem. 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

Jerry Darsteler

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

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 (Temporary)
695 East 1st North, Pleasant Grove, Utah (permanent), 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

JLW

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 developped 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 plm. 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

Jerry Lynn Walker

Subscribed and sworn to before me this 23rd day of June 1976

Betty J Foyes

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. Buckert
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-Knudswn-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 powerhouse 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 dozerspushing 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

COPY

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Llewellyn L. Payne, P.O. Box 37, Ashton, 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 concrete superintendent with Morrison-Knudsen and Kiewit, on the Tetan Dam Project, Newdale, Idaho. I have worked there for three years and one month.

On Saturday, June 5, 1976, Duane Buckert called my house and left a message for me to come to the dam. I got the message about 10:00 a.m. and arrived at 10:20 a.m. I travelled by the lower road and arrived at the powerhouse. As I approached I could see muddy water in river but not the actual leak. As I got closer, I could see the leak, which was about 75 - 100 feet from top of the dam on north side against abutment and zone material. I cannot estimate volume and at this time I did not see actual hole.

After my arrival I got Archie J. Zuern, Claude Rhodes, Michael Powell and Charles Powell and went into the river outlet tunnel. The purpose was to get painting equipment out in order to let water through. We went into the tunnel about 10:30 a.m. At the time, one dozer was working on downface of the dam and another was on its way. I was in and out of the tunnel and watching leak so could pull men if danger became too great.

The leak grew larger - water was muddy, and at about 11:20 or 11:30 a.m. the two dozers were washed out. We went into the tunnel one more time to move anything. We left for the top of the dam shortly thereafter on foot. The water was flowing heavily and began coming around the powerhouse. When I was about half-way up, I could see dozers working on top. I could see the dam washing out and radioed to move the cats because the dam was going. I saw the dam go, but cannot make a guess. I did not look at a watch and just never gave the time factor a thought.

At about the time the dozers were lost - 11:30 a.m., I was scared and had the feeling the dam was going to collapse.

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

COPY

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 3-½ 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 down stream 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. Poxleitner, 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

COPY

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-Knudwen, 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 ~~care~~ 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 waterwas 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 greenpickup 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

COPY

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 reservior 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 comparatively 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

STATE OF Idaho)
) SS
COUNTY OF Madison)

I, Ted V. Gould, 455 N. South W., 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 self employed.

Saturday, June 5, 1976, I was going to Teton, Idaho, to work on my trucks. This was about 8:30 a.m. I had two-way radios on same frequency as Gibbins and Reed and others and heard talk about leak being at dam. I thought little of this, but continued to ~~have~~ see talk regarding equipment movement and the leakage. Then I heard someone say there was a hole in the dam.

I arrived about 9:30 a.m. and went to visitor viewpoint. Small hole about half-way up about 20 feet into dam on downstream side. The water washing away material and this made it muddy. The dozers were just heading toward area to fill in rock at the hole area. The hole gradually got bigger and more water flowing as I watched. More and more volume. One dozer, D-8, started slipping into washed out area and D-15 tried to pull out. Unsuccessful and about 10:00 am. the dozers washed away.

Then big chunks of dirt fell out of hole and water appeared to be running out of side abutment rock.

I was back in my truck about this time and heard someone on the radio mention whirlpool on upstream face and I heard the person talking say that there was big trouble and probably not be able to stop. This was possibly about 10:30 a.m.

I left shortly thereafter and went to Teton to check on my parents. At about 11:00 a.m., I heard a radio message over Gibbons and read about top of dam going pusing riprap into whirlpool.

I called my wife and told her about the incident. I talked Gibbons and Reed over the radio and he told me top washing out and dam going. This was about 11:30 a.m. I did not see the actual collapse.

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 $2\frac{1}{2}$ 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 th 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 aid 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 onthe 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 callapsed shortly thereafter. He said he did not see the collapse, because he was preparing to leave for Rexburg, Idaho, and do what he couldto protect his home from the flood.

This is not a signed statement because Mr. Hopkins was departing for California.

COPY

STATE OF) Idaho
) SS
COUNTY OF) Fremont

I, Elizabeth A. Howard, P.O. Box 342, St. Anthony, Id. 83445, 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 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, EH 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 obervations 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.

Subscribed and sworn to before me
this 22nd day of June, 1976.

Ivan L. Kestner, Special Agent
U.S. Department of the Interior

COPY

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 visable 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

copy

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, KRKX, 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.

Eunice J. Olson

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

COPY

STATE OF Idaho)
) SS
COUNTY OF Madison)

We, Mr. Lynn Schwendiman
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 Schewendeman 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. Siepert, 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

Design Questions and Answers

- Design questions asked by the Independent Panel of the Bureau of Reclamation and the Morrison-Knudson Co., Inc. and their response.

- Bureau of Reclamation's response to the IRG's 24 questions.

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.

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.

Please accept our appreciation in advance for your cooperation in supplying this important supplementing information.

Very truly yours,

Robert B. Jansen

for Wallace L. Chadwick

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

K. W. Smith
ASSISTANT GENERAL COUNSEL

October 7, 1976

Mr. Harold G. Arthur, Director
Design and Construction Bureau of Reclamation
Building 67
Denver Federal Center
Denver, Colorado 80225

Re: Teton Dam

Dear Mr. Arthur:

Pursuant to the request of Mr. E. M. Armstrong, enclosed for your information is a copy of the joint venture's response of this date to Mr. Wallace L. Chadwick, representing the Independent Panel to Review the Cause of the Teton Dam Failure.

Sincerely,

K. W. Smith

KWS:jl

Encl.

CONTRACTORS
ENGINEERS
DEVELOPERS

MORRISON-KNUDSEN COMPANY, INC.

EXECUTIVE OFFICE
ONE MORRISON KNUDSEN PLAZA / PO BOX 7808 / BOISE, IDAHO 83729 / USA
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

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.

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

/s/
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

RECEIVED

AUG 31 1976

Morrison-Knudsen - Kiewit
Contract No. 2594

Morrison-Knudsen Company, Inc.
P. O. Box 127
Newdale, Idaho 83436

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 drilled.

DIAMOND CORE DRILLING -- ROTARY DRILLI
PRESSURE GROUTING — FOUNDATION TESTI
BOX 1892, IDAHO FALLS, IDAHO 83401
PHONE 522-5437

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 slop, 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 13 + 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
PRESSURE GROUTING — FOUNDATION TESTING
BOX 1892, IDAHO FALLS, IDAHO 83401
PHONE 522-5437

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
President

ELM:RJN

...m DFC-11
2-61)
Bureau of Reclamation

INFORMATIONAL ROUTING

510. 210 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 13, 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

H. G. Arthur
Director
Design and Construction

Enclosures

Copy to: Morrison-Knudsen Company, Inc.
Post Office Box 7808
Boise, Idaho 83723
Attention: Mr. W. X. Smith
(with copy of enclosures)

QUESTIONS ASKED BY INDEPENDENT PANEL
of H. G. Arthur and Morrison-Knudson
August 18, 1976

and

THE BUREAU RESPONSE
October 19, 1976

(Note: Most of the information in our answers was furnished by the Project Construction Engineer.)

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 alinement of the centerline curtain was in the same alinement 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 desirous 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:

Water accepted in 5 minutes	Starting Grout Mixture
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:

Water accepted in 5 minutes	Starting Grout Mixture
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 diaphram 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 $2\frac{1}{2}$-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 11+19 5' U/S is comp

③ 9+40 10' D/S 0- 260'

④ 10+44 12' U/S 0-80 after 10+44 2½ D/S is comp
also 10+59 5' U/S is comp to 80'

⑤ 10+74 19½' D/S 33-80 after grouting WL

⑥ 10+66 19½' D/S 0- 80 after 10+74 19½' D/S
is complete to 80 feet.

10+59 5' U/S 38-80 after grouting WL.

Grouting

Cont. 10+31 15' U/S 0-10 WL

Cont 10+44 12½' D/S 0-80

Cont 10+89 5' U/S 0-80

Drill Ht 10+74 19½' D/S 33' WL

① 11+19 5' U/S 0-80

If ~~[illegible]~~ 10+74 19½' D/S proceeds to take at the same rate as before - Thicken up quicker and use Ca Cl on 1:1 + .8:1 mixes to bring Temp to 90° or there abouts.

[illegible]

Spillway - D/S Curtain 3-22-15

1) Drilling

1) Cont 11+20 5' D/S 0-310

) 12+00 10' D/S 0-260

) 9+40 10' D/S 0-260

) 10+44 12' U/S 0-80 after 10+59
5' U/S is complete to 80'

) 10+74 19½' D/S 33-80 after grouting WL

10+59 5' U/S 38-80 after grouting WL

10+66 19½' D/S 0-80 after 10+74 19½' D/S is complete to 80 feet

Grouting

1) Cont 10+31 15' U/S 0-10

1) Cont 10+59 5' U/S 38' WL

Cont 10+74 19½' 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 max on D/S curtain holes between station 11+20 and 9+60

Also let's correlate our labor reports on shift. We're listing some people twice.

Sample of Log Book Sheet

13+96

9-13-74

1 SK
9/12/74

20'
H_2O — 3.5 CF
DAY 9-12-74
D.m.

40'
H_2O 1.7 CF
9-12-74 DAY
D.m.

60'
H_2O 1.6 CF
DAY 9-12-74
D.m.

80'
H_2O 3.0 CF
DAY 9-12-74
D.m.

100'
H_2O .? CF
DAY
9-?-74
D.m.

130'
H_2O 0 CF
DAY
9-12-74
D.m.

160'

DRILLING COMPLETE
9/11/74 SWING
HOLE COMPLETE
DAY 9-13-74 D.m.

14+06
HOLE COMPLETE
9/13/74

H_2O 0.6 C.F.
9/12/74

20
H_2O 0.3 C.F
9/13/74

40
H_2O 0.0 CF
9/13/74

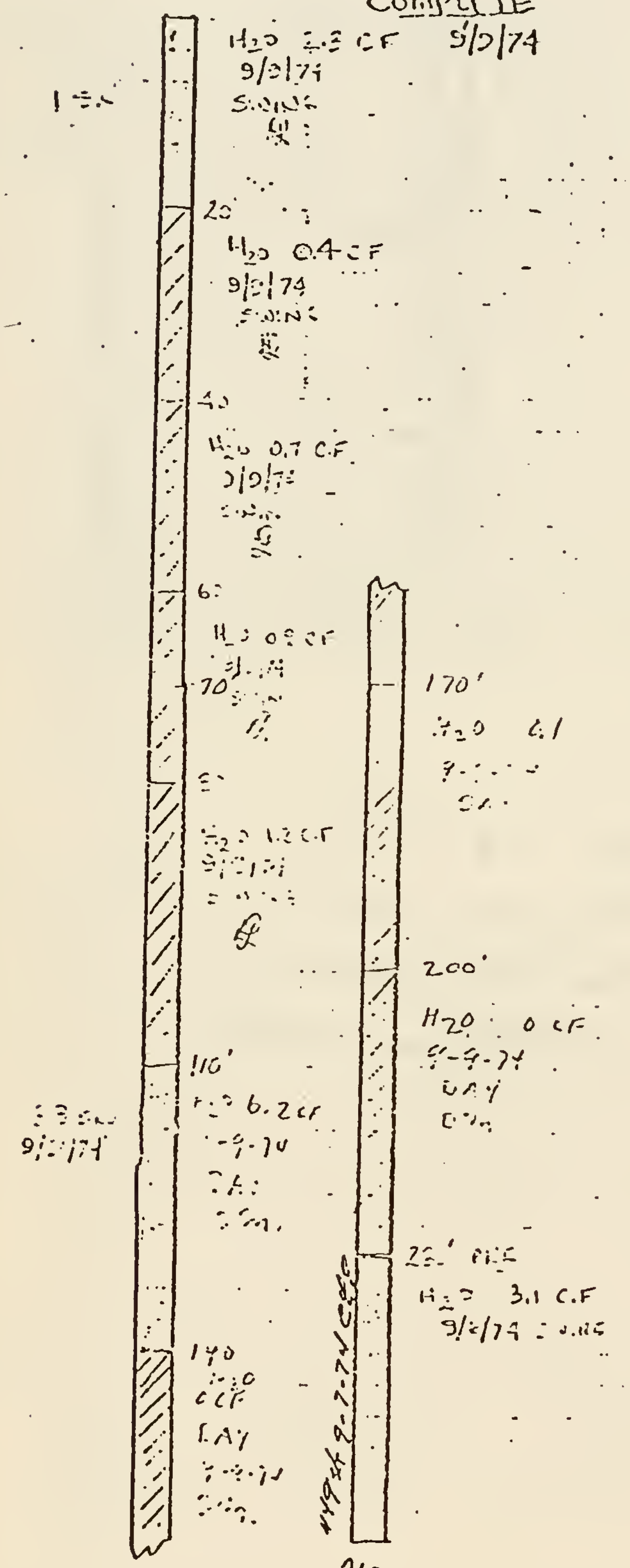
60'

DRILLING COMP
DAY 9-13-74 D.m.

Page 38 from
Inspectors Log book.
Holes 13+96 and 14+06
on Centerline Curtain

2½' x 11"
14+15

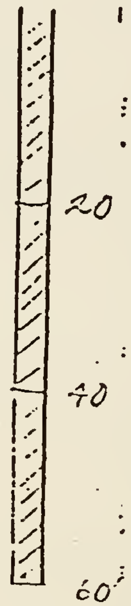
HOLE COMPLETE



DRILLING COMP
DAY 9-5-74

BACKFILLED 9/5/74

2½' x 11"
14+25



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 Centerline Curtain

GROUTING INSPECTOR'S REPORT

Sample Grout Sheet

SHEET 1 OF [illegible]

PROJECT Teton Basin [illegible] SHIFT [illegible]

FEATURE Teton Dam
(DAM, TUNNELS, SPILLWAY, ETC.)

INSPECTOR A. Stocker

SPECS NO DC-6910 LINE ____
(A, B, C ETC.)

RELIEVED BY [illegible]
" [illegible]

HOLE NO. 6+70 10' U/S	
DEPTH DRILLED	60
PROPOSED FINAL DEPTH	60
PREVIOUSLY GROUTED	0

CEMENT SUMMARY (C.F.)	PLACED	WASTED GOV'T.	WASTED CONTR.
WITH PACKER	134		2 [illegible]
WITHOUT PACKER			[illegible]
TOTAL			

(Guage O.K. 11:00 P.M.)

STAGE DEPTH	TIME	CEMENT TOTAL	CEMENT CHANGE	SACKS PER HR.	W/C	PUMPING PRESSURE	REMARKS
40-60	11:00 P	Water test			H_2O	26	start test
	11:05				"	"	0.2 c.f. in 5 min
							22# HP 15# BP
20-40	11:13	Water test			H_2O	13	Start test
	11:18				"	"	0.1 c.f. in 5 min.
							13# HP 10# BP
0-20	11:27	Water test			H_2O	10	start test
	11:32				"	"	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	"	"	
	5:00	48	10	10	5-1	10	
	6:00	56	8	8	"	"	
	7:00	64	8	8	"	"	
	8:00	70	6	6	5-1	10	End S. Guage O.K.
Day [illegible]	[illegible]	12-5	74	[illegible]			
	9:00	90	10	8	5:1	10	
	[illegible]	98	8	8	"	"	Told Cont. to change to 4:1
	[illegible]	[illegible]	2	8	"	"	Change to 4:1
	11:55AM	94	4	8	4:1	"	
	[illegible]	100	6	6	"	"	
	1:50 PM	106	6	6	"	"	
	2:00 PM	112	6	6	4:1	10	
	3:00 PM	116.	4	4	"	"	
	4:00 PM	118	2	2	4:1	"	END OF SHIFT.
SW 12/5/74	5:00	120	2	2+	"	"	1/2 c.f. in 5 min
	6:00	122	2	2+	"	"	
	7:00	124	2	2+	"	"	
	8:00	126	2	2+	"	"	1/2 c.f. in 5 min.
	9:00	128	2	2+	"	"	
	10:00	130	2	2+	"	"	
	11:00	132	2	2+	"	"	1.25 c.f. in 5 min
	12:00	134	2	2	"	"	Water tested grout

6# HP

This report should show a complete record of G-25 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.

[illegible]

GROUTING INSPECTOR'S REPORT

SHEET __ OF __

PROJECT: Teton Dam — Dec. 5 — SHIFT: Swing — 1974

FEATURE (DAM, TUNNELS, SPILLWAY, ETC.): Spillway Blanket grout — INSPECTOR: G. Harris

SPECS NO.: DC-6910 — LINE (A, B, C ETC.): 12' D/S — RELIEVED BY: R. P. Michel

HOLE NO.	
11+04 12' D/S	
DEPTH DRILLED	80
PROPOSED FINAL DEPTH	80
PREVIOUSLY GROUTED	0

CEMENT SUMMARY (C.F.)	PLACED	WASTED GOV'T.	WASTED CONTR.
WITH PACKER			
WITHOUT PACKER			
TOTAL			

STAGE DEPTH	TIME	CEMENT TOTAL	CEMENT CHANGE	CEMENT SACKS PER HR.	W/C	PUMPING PRESSURE	REMARKS
60-80	7:30		water test			45	0.6 CF in 5 min
	7:35						
40-60	7:40		water test			30	5.5 CF in 5 min
	7:45						
40-60	1:45				5-1	30	start grouting
	2:15	2	2	4	5-1	30	
	3:00	4	2	2+	5-1	30	
	3:20	4	0	—	5-1	30	refused grout
20-40	3:52		water test		H_2O	15	0.8 C.F. in 5 min.
	3:57						
0-20	4:06		water test		H_2O	10	10.6 C.F. in 5 min.
	4:11						
0-20	4:20				5-1	10	start grouting
	5:00	4	4	6	5-1	10	
	6:00	10	6	6	5-1	10	
	7:00	16	6	6	5-1	10	
	8:00	22	6	6	5-1	10	E.O.S.
0-20'	8:00				5:1	10	Continued grouting
	9:05	24	2	2	5:1	10	[illegible] grout
							filled [illegible]
							Hole complete

Hole Complete

This report should show a complete record of G-26 eatment 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.

GPO 840-701

7-1744 (2-71)
Bureau of Reclamation

INSPECTOR'S DRILL RECORD
(FOR GROUT OR DRAINAGE HOLES)

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"X41" PIPE		
11+04	4:40PM		0		STARTED DRILLING	GY 12-10-74'	DM
18' u/S					DRILLER: G. NIELSEN		
		5:00PM		5'	20% WATER LOSS		
	5:00PM		5'		CONTINUED DRILLING		
		7:30PM		42'	40% WATER LOSS	GY 12-10-74	DM
	4:40 FF		42'			[illegible]	[illegible]
		6:30		60'	100% water loss at 59'	"	"
					Stopped to grout		

*NOTE DEPTH OF CONCRETE IF ANY, FORMATION CHANGES, WATER LOSS OR GAIN, CAVE, ETC.

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 then 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 kyeway 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 roles 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 handwork 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

570.

LOWER TETON

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.

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.

August 18, 1976

Please accept our appreciation in advance for your cooperation in supplying this important supplementing information.

Very truly yours,

Robert B. Jansen

for Wallace L. Chadwick

Sample of Hole Summary Sheet from
L-10 Report

RECORD OF DRILLING AND GROUTING OPERATIONS

Project Teton Dam
Feature Rt. Abut. Key Trench (Dam, Spillway, Tunnel, etc.)
Drawing 549-147-193 Line Upstream Curtain (A,B,C, etc.)

Date May 1975
Sheet 10 of 76

STA. OR HOLE	DRILLING: DEPTH OF HOLE: IN CONC	IN ROCK	STAGE DEPTH	DATE DRILLED	REMARKS	GROUTING: DATE GROUTED	CEMENT PLACED	PRESSURE: MAX.	MIN.	W/C: MAX.	MIN.	MAX. SKS/HR	REMARKS
10+44			0-3		Pipe								
U/S		17	3-20	5-10		5-11	0	10	10	W	W	—	H_2O Test-Took 1.3 c.f. in 5 min.
		20	20-40	"		"	0	13	13	W	W	—	H_2O Test-Took 1.4 c.f. in 5 min.
		20	40-60	"		"	0	26	26	W	W	—	H_2O Test-Took 1.8 c.f. in 5 min.
		20	60-80	"		"	0	39	39	W	W	—	H_2O Test-Took 1.4 c.f. in 5 min.
		20	80-100	"		"	0	52	52	W	W	—	H_2O Test-Took 1.9 c.f. in 5 min.
		30	100-130	"		"	0	65	65	W	W	—	H_2O Test-Took 1.7 c.f. in 5 min.
		30	130-160	"		"	5	84	84	8:1	8:1	4	H_2O Test-Took 2.5 c.f. in 5 min.
		30	160-190	"		"	0	100	100	W	W	—	H_2O Test-Took 0.1 c.f. in 5 min.
		30	190-220	"		"	0	100	100	W	W	—	H_2O Test-Took 0.2 c.f. in 5 min.
		30	220-250	"		"	1	100	100	8:1	8:1	1	H_2O Test-Took 2.3 c.f. in 5 min.
		30	250-280	"		"	2	100	100	8:1	8:1	2	H_2O Test-Took 3.3 c.f. in 5 min.
		30	280-310	"		5-10	11	100	100	8:1	8:1	12	H_2O 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_2O Test
		20	20-40	"		"	1	13	13	8:1	8:1	2	H_2O Test-Took 2.8 c.f. in 5 min.
		20	40-60	"		"	0	26	26	W	W	—	H_2O Test-Took 1.6 c.f. in 5 min.
		20	60-80	5-11		"	0	39	39	W	W	—	H_2O Test-Took 0.8 c.f. in 5 min.
		20	80-100	"		"	0	52	52	W	W	—	H_2O Test-Took 0.6 c.f. in 5 min.
		30	100-130	"		"	0	65	65	W	W	—	H_2O Test-Took 0.7 c.f. in 5 min.
		30	130-160	"		"	3	84	84	8:1	8:1	2+	H_2O Test-Took 2.9 c.f. in 5 min.
		30	160-190	"		"	0	100	100	W	W	—	H_2O Test-Took 1.8 c.f. in 5 min.
		30	190-220	"	Grout seams @ 210' & 220'	"	0	100	100	W	W	—	H_2O Test-Took 1.9 c.f. in 5 min.
		30	220-250	"		"	0	100	100	W	W	—	H_2O Test-Took 1.0 c.f. in 5 min.
		30	250-280	"		"	0	100	100	W	W	—	H_2O Test-Took 0.9 c.f. in 5 min.
		30	280-310	"		"	0	100	100	W	W	—	H_2O Test-Took 0.5 c.f. in 5 min.
													Hole Complete

THIS CONSOLIDATED REPORT TOGETHER WITH PLANS AND PROFILE SHOWING LOCATION OF HOLES TO BE FURNISHED MONTHLY TO THE CHIEF ENGINEER – ATTENTION CODE 280

7-1-C-11
Bureau of Reclamation

INFORMATIONAL ROUTING

BP 11:24
Guy

200

August 26, 1976

Memorandum

To: Project Construction Engineer, Newdale, Idaho

From: Director of Design and Construction

Subject: Teton Dam Failure

Enclosed is a copy of a letter dated August 18, 1976, from the Independent Panel addressed jointly to Mr. McMurren and myself which asks for information regarding the construction of Teton Dam.

Also enclosed is a copy of my diary entry regarding the telephone conversation held with Mr. Armstrong of M-K on August 24, 1976, in which we discussed the procedure to be followed in replying to this letter.

Please prepare a draft reply obtaining information from the contractor's field personnel as appropriate. The draft should be forwarded to me for consideration. I will handle the coordination with Mr. McMurren.

Note the Panel's statement that partial replies are encouraged and that transmittals for individual questions will be helpful.

This, of course, should be given high priority.

H. G. ARTHUR

Enclosures

Copy to: W. H. McMurren
(with copy diary entry)
Blind to: Commissioner, Attention: 105 (Blue Envelope)
200
1300 (Guy)

☆ GPO - 1974 - 678 - 564

Cole Bros. Drilling, Inc.

DIAMOND CORE DRILLING — ROTARY DRILLING
PRESSURE GROUTING — FOUNDATION TESTING

BOX 1892, IDAHO FALLS, IDAHO 83401
PHONE 522-5437

August 25, 1976

RECEIVED
AUG 26 1976
Morrison-Knudsen - Kiewit
Contract No. 2594

Morrison-Knudsen Company, Inc.
P. O. Box 127
Newdale, Idaho 83436

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.

Cole Bros. Drilling, Inc.

DIAMOND CORE DRILLING — ROTARY DRILLING
PRESSURE GROUTING — FOUNDATION TESTING

BOX 1892. IDAHO FALLS. IDAHO 83401
PHONE 522-5437

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 specificed pressure. If the above hole was required to go deeper, we then drilled to the specificed depth or until we had a water loss of 50% of 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 stageup, 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 continously until leakage stopped.

Referring to Question: Details of any doubts over the effectiveness of this Axial distribution in any particular location

DIAMOND CORE DRILLING — ROTARY DRILLING
PRESSURE GROUTING — FOUNDATION TESTING
BOX 1892, IDAHO FALLS, IDAHO 83401
PHONE 522-5437

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.

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

H. G. Arthur, Director of Design and Construction

Tuesday, August 24, 1976

Call from Gene Armstrong, Morrison-Knudsen, Boise, re Investigation of Teton Dam Failure

Armstrong read to me a letter just received by M-K which was addressed jointly to McMurren and to myself. The date of the letter was August 20, 1976. I told Armstrong that I had not yet received my copy.

The letter was from Chadwick who was asking for information regarding the grouting of the dam foundation and the construction of the embankment. He asked a number of detailed questions relating the construction to specific stations of the dam.

I suggested to Armstrong that the Bureau take the lead in preparing a response to this letter since I felt that our Project office probably had better records than the contractor. I said that any information available in the M-K field office which would be helpful in preparing the response should be furnished to our Construction Engineer. I further suggested that a draft of the proposed reply be reviewed by M-K and that the reply be made jointly by Mr. McMurren and myself.

Armstrong agreed with this procedure saying that he had already discussed it with key personnel in M-K and he felt this would be an appropriate means of being responsive to Chadwick's request. He said that he had advised his staff that all factual information available should be furnished but he felt that speculation not founded on the fact should be avoided as it would not be helpful. I agreed with this concept.

Copy of this diary entry was furnished to Code 1300 and to the Project Construction Engineer by blue envelope.

USBR RESPONSE

TO

INTERIOR REVIEW GROUP'S

24 QUESTIONS

February 1, 1977

Question No. 1

What was the organizational structure of the Division of Design during the design and construction of Teton Dam?

ANSWER

From January 1, 1969, to September 1, 1970, the following structure existed for the segments concerned with Teton Dam:

The Division of Design and all other divisions at the Engineering and Research Center reported to:

Chief Engineer
Associate Chief Engineer

Division of Design

Chief Designing Engineer
Assistant to Chief Designing Engineer - Civil and Structural
Assistant to Chief Designing Engineer - Electrical and Mechanical

Chief, Dams Branch
Head, Concrete Dams Section
Head, Earth Dams Section
Head, Spillways and Outlets Section
Head, Highway and Structural Behavior Section

Division of Engineering Geology

Chief
Associate Chief

Chief, Geophysics Branch
Chief, Construction and O&M Geology Branch
Chief, Geology Branch No. 1
Chief, Geology Branch No. 2
Chief, Cartographic and Geologic Specifications Branch

From September 1, 1970 to August 20, 1972

The Division of Design absorbed the Division of Engineering Geology and reported to the Office of Design and Construction

Director of Design and Construction
Deputy Director of Design and Construction

Division of Design

Chief, Division of Design
Assistant Chief, Division of Design

Chief, Hydraulic Structures Branch
- Head, Concrete Dams Section
- Head, Earth Dams Section
- Head, Structural Behavior and Analysis Section
- Head, Tunnels and Pipelines Section
- Head, Canal Structures and Bridges Section

Chief, Geology and Geotechnology Branch
- Head, Geotechnical Section
- Head, Engineering Geology Section No. 1
- Head, Engineering Geology Section No. 2
- Head, Cartographic and Specifications Section

On August 20, 1972, the structure of the Hydraulics Structures Branch was changed to:

Chief, Hydraulics Structures Branch
- Head, Concrete Dams Section
- Head, Earth Dams Section
- Head, Spillways, Outlets, and Experimental Analysis Section
- Head, Tunnels and Pipelines Section
- Head, Canal Structures and Bridges Section

All other pertinent organizational segments remained the same.

On December 2, 1974, the structure of the Geology and Geotechnology Branch was changed to:

Chief, Geology and Geotechnology Branch
- Head, Geotechnology Section
- Head, Engineering Geology and Specifications Section
- Head, Engineering Geology and Special Studies Section

All other pertinent organizational segments remained the same.

On August 18, 1975, the Washington Office approved dividing the Hydraulic Structures Branch into two branches. Branch chief positions were filled October 26, 1975, for the Dams Branch and the Water Conveyance Branch.

Chief, Dams Branch
- Head, Concrete Dams Section
- Head, Earth Dams Section
- Head, Spillways and Outlets Section
- Head, Technical Analysis Section

Chief, Water Conveyance Branch
- Head, Tunnels Section
- Head, Pipelines Section
- Head, Canals and Bridges Section
- Head, Canals and Diversion Structures Section

All other pertinent organizational segments remained the same. This is the structure in effect on June 5, 1976, and at the present time (2/1/77).

Question No. 2

Explain the functions of the various levels of the organization in the design phase.

ANSWER

The Office of Design and Construction (formerly Chief Engineer's Office) has primary responsibility for supervision and coordination of activities relating to the Bureau's design and construction of water resources developments and for providing technical support services for resource planning and management through the appropriate divisions under its jurisdiction. These divisions are Design, Construction, and Engineering Support.

The Division of Design is the principal office of the Bureau for design activities. It supervises and coordinates the Bureau design program, including development of policies, guidelines, standards, and procedures relating to design work. Appropriate branches under its supervision:

Prepare final designs and technical specifications

Prepare appraisal and feasibility designs and quantity estimates as requested by Regional Directors

Prepare and issue design action schedules

Apply theory and mathematical analysis to highly technical engineering problems in the solution and development of design practices

The Division Office staff supervises design and geological activities throughout the Bureau, including guidance and review of such activities to assure conformance to policies and standards, and consideration of water quality control and environmental compatibility of structures. It also originates, coordinates, and participates in a Bureau-wide program of technical investigations and studies of a developmental and research nature directed toward new approaches and potential solutions of unresolved design problems or improvement in existing and accepted design methods and criteria both as to cost and technical excellence with particular emphasis on civil, mechanical, structural, hydraulic, and electrical power engineering applications.

The Earth Dams Section of the Dams Branch prepares appraisal and feasibility designs, estimates, and layouts; develops new approaches and methods of structural and stability analysis for earth dams and other earth structures; prepares specifications designs and quantity estimates, including general plan and section drawings; prepares layouts and final designs and drawings for construction of access roads and appurtenant structures, railroads, and highways; designs and develops instrumentation for earth dams; collects, processes and analyzes performance data and determines structural behavior, directs and analyzes soils testing programs for earth dams; reviews earth dam designs submitted for the Small Loans Program; analyzes and assesses earth dams in the Safety of Dams Program; develops and maintains ADP programs; coordinates design data with the field offices and inspects construction sites; prepares and coordinates various reports and instructions such as design considerations, designers' operating criteria, and final design reports; consults with other Federal agencies and foreign governments on soils problems; and reviews and comments on planning reports for proposed projects.

The Spillways and Outlets Section of this branch prepares appraisal, feasibility, and specifications designs and quantity estimates for spillways, outlets, diversion works, and migratory fish control works; prepares construction drawings including concrete outlines and reinforcement design; evaluates and designs structures for control of water quality; coordinates the structural and hydraulic designs with other design sections and branches; reviews hydraulic and structural designs of existing structures under the Safety of Dams Program and prepares designs for modifying structures as required; reviews designs and specifications prepared by regional offices; reviews designs and specifications prepared by consulting engineering firms under Public Law 984 (Small Projects Program); prepares design considerations reports for use by construction engineers; provides technical assistance to other agencies and to foreign governments; performs research and development related to structures for which the section is responsible; participates in examination of new structure sites and structures under construction; prepares designers' operating criteria; prepares final design reports and other technical reports, reviews definite plan reports and environmental impact statements; assists in evaluating results of hydraulic model testing; evaluates structural and hydraulic performance data; coordinates materials testing with Division of General Research; and develops and maintains ADP programs.

The Geology and Geotechnology Branch provides technical supervision of Bureau engineering geology and geophysics functions and the rock mechanics and soil mechanics aspects of natural slopes and foundations, except for associated laboratory investigations, testing, and analysis. It develops and recommends related Bureau technical standards and procedures; advises on the geological features of investigations, including geothermal, and projects at all stages from inception through operation and maintenance; collaborates with the Division of General Research and the Division of Planning Coordination in the development of plans for geologic study and exploration of engineering structures and reservoir sites, project areas, and sources of natural construction materials, including necessary field and laboratory tests to secure geologic data for Bureau program purposes; reviews and evaluates geologic data and reports; conducts geophysical investigations; prepares geologic data for use in specifications for bidding and construction purposes; interprets geologic conditions from aerial photographs, prepares aerial mosaics, and makes studies utilizing photogrammetric methods; and conducts and advises on research relating to engineering geology matters.

The functions of the various organizational segments have been relatively unchanged by internal reorganization since the start of work on Teton Dam.

Question No. 3

Explain how the geologists and designers reached agreement on depth of excavation, foundation treatment, etc.

ANSWER

The geologists and designers reached an agreement on the depth of excavation by examining the following information:

1. Preconstruction Geology Report
2. Pilot Grouting Study
3. Pump-in test data
4. Core taken from drill holes prior to and after grouting

and by verbal discussions. The items which were most influential in determining the excavation depth were the grout takes from the pilot grouting program, water loss results from the pump-in tests, and openness of fractures at depth. Decisions on foundation treatment and other items were made by examination of the Preconstruction Geology Report, and other available written information, and verbal discussion between the concerned parties. Design Considerations were prepared and issued in October 1971. This volume was available to and reviewed by geologists, designers, and construction personnel.

Question No. 4

During construction of the dam, what was the policy for site visits by key designers to verify design assumptions and conditions and/or determine necessity for any redesign or design modifications? Were inspections made by the dam designers or by liaison personnel acting as interface between design and construction with no direct design or construction responsibility. Furnish information concerning inspection visits with dates, names, and responsibilities of all individuals from design and geology who visited the dam during construction and give specific reason for visits.

ANSWER

Construction liaison personnel visited the damsite to ensure conformance with specifications. These visits were initiated by either the project personnel or by the Denver Office. Designers visited the site when the foundation excavation was complete and prior to the placement of any zone 1 material. Designers and geologists also visited the site when specific problems arose or when requested to do so by project personnel. Attached is a list of travel reports covering trips made by designers and geologists during construction which were pertinent to the dam and its foundation.

TETON DAM

Travel Reports of Designers and Geologists
Pertinent to the Dam and its Foundation

September 1, 1972

Traveler: J. D. Gilbert, Geologist, and Ed Webb, ADP

Dates of visit: August 14-17, 1972

Subject: Collection of Teton Dam Joint Data in a Form Suitable for Computer Processing

(a) Discusses examination of river outlet works tunnel, (b) left abutment foundation key trench, and (c) joint data to be computerized. (Visit suggested by July 18, 1972 letter for Director of Design and Construction to Regional Director.)

October 25, 1972

Traveler: M. A. Jabara, Head, Spillways and Outlets and Experimental Analysis Section, and former Head, Earth Dams Section, and Donald Colgate, Hydraulics Engineer

Dates of visit: September 13-14, 1972

Subject: To examine excavated key trench on left abutment, partially excavated key trench on right abutment, river outlet works shaft, and the downstream portion of the river outlet works tunnel. (Letter for Director, Design and Construction, to Regional Director dated August 15, 1972, and reply dated August 23, 1972.)

October 10, 1972

Traveler: Andris Viksne, Geophysicist

Dates of visit: September 20-23,1972

Subject: Electromagnetic Subsurface Profiling

To monitor the performance of the contract with Geophysical Survey Systems, Inc., for delineating alluvium-covered, near-surface fractures in the bedrock - left abutment. (Internal memorandum for Chief, Geology and Technology Branch, to Director of Design and Construction, September 6, 1972.)

November 23, 1973

Visitor: Ira Klein, Supervisory Geologist, and Richard Bock, Head, Earth Dams Section

Dates of visit: October 16-17, 1973

Subject: Review of Construction

Review of cutoff trench excavation prior to placement of zone 1 embankment and other construction work and observations regarding evidence of possible faults in damsite vicinity. (Telephone call from Project Construction Engineer, October 12, 1973.)

March 29, 1974

Visitor: L. R. Gebhart, Construction Liason, W. G. Harber, Design Engineer, J. D. Gilbert, Geologist

Dates of visit: March 20-21, 1974

Subject: Examination of Right Abutment Key Trench

To examine voids exposed in right abutment key trench between stations 3+45 and 4+44. (Project Construction Engineer letter dated March 14, 1974.)

April 16, 1975

Visitor: A. Viksne, Geophysicist, and D. Route, Engineering Technician, Geotechnology Section

Dates of visit: March 24-27, 1975

Subject: Shear Wave Velocity Measurements

Performed in situ shear wave velocity measurements for zone 1 and zone 2 and compressional wave velocity in zone 1, utilizing down hole, cross hole, and up hole methods. (Memorandum to Project Construction Engineer, October 7, 1974, and reply to Director of Design and Construction, November 1, 1974.)

June 12, 1975

Visitor: H. Ham, Groundwater Geologist, and R. Farina, Head, Engineering Geology and Special Studies Section

Subject: Field Review of Geologic Investigations for Fissures and Ground-water Observation and Monitoring Program

Reviews the monitoring system to be set up to measure anticipated reservoir seepage losses, discusses large fissures exposed by new haul road. (Memorandum from Assistant Regional Director, May 9, 1975, and Acting Director, Design and Construction's reply, May 22, 1975.)

Question No. 5

Within the design organization, was there a particular individual or a particular position, knowledgeable on all aspects of the Teton Project, that had the responsibility of monitoring construction to ensure that the dam and all appurtenant structures were being constructed as designed? Who was the "key designer?" Who was the supervisor of the coordinating unit? How was the construction monitored?

ANSWER

Design and construction of a Bureau of Reclamation earth dam project is a joint effort among many sections in the Division of Design such as the Earth Dams Section, the Spillways and Outlet Works Section, the Engineering Geology Section, and the Special Studies Section, etc. Specific individuals within these sections are responsible for being knowledgeable about the project for their specific discipline. This is a team effort and no one individual in the design organization has an in-depth knowledge of all aspects of a project. Responsibility for monitoring the construction of the various portions of the total design of a project is assigned to the section that prepared that specific portion of the design. The supervisor of the coordinating unit of the coordinating section coordinates the design effort and closely follows construction activity. Teton Dam Monthly Construction Progress Reports (which included earthwork control data) and earthwork information in Weekly Progress Reports were carefully and continuously reviewed.

For Teton Dam, the Earth Dams Section was the coordinating section. During the major portion of the design of Teton Dam, the key designer was Mr. W. G. Harber; Mr. R. W. Bock was his unit supervisor, and Mr. F. C. Walker was Head of the Earth Dams Section.

Question No. 6

How were design parameters established and selected for each of the zones of the dam embankment and its alluvial foundation? What design assumptions were made related to these various materials for stability and seepage analyses?

ANSWER

For zone 1 materials the design parameters were determined by laboratory test. Zone 3 parameters were assumed similar to those of zone 1 since the zones were constructed of the same materials. The design parameters for zone 2, zone 4, and zone 5 were estimated. Strength and weight parameters for zone 4 were assumed similar to those of zone 1.

The parameters for the alluvial foundation were assumed similar to those of zone 2 since zone 2 was constructed from alluvial materials.

For stability analyses the following parameters were used:

Zone	tan ϕ	c (wet)	c (sat)	γ_d (pcf)
1	0.61	11.5	2.5	99.8
2	0.70	0	0	126.9
3	0.61	11.5	2.5	99.8
4	0.61	11.5	2.5	99.8
5	0.70	0	0	105.4
Alluvium	0.70	0	0	126.9
Foundation	1.00	0	0	137.5

For seepage analyses permeability in zones 1 and 3 was determined from laboratory testing to be approximately 10^{-6} cm/s and the horizontal permeability was assumed as four times as great as vertical permeability. Zones 2, 4, and 5 were assumed to have permeabilities significantly greater than zone 1.

Question No. 7

Zone 2 material does not meet the generally accepted criteria for a filter against zone 1. What is Bureau criteria for use of filters and what was the rationale on the need for designed filters at Teton?

ANSWER

The grain-size curves that were available for design are included in exhibit 24 which has been distributed to the Independent Panel and the Review Group. The interpolated average D_{85} size of the zone 1 curves from 21 preconstruction borrow area A samples is approximately 0.122 mm as shown in the attached figure 1. Using the average D_{85} size of the zone 1 as 0.122 mm, the most generally used filter criteria (Terzaghi filter criteria) states that the D_{15} size of the filter must not be greater than five times the D_{85} size of the protected soil (base) (reference 1, p. 175) and this requires that $D_{15} < 0.61$ mm. Plotting this diameter on the attached preconstruction grain-size curves for the zone 2 material from test pits C2, figure 2, and C8, figure 3, shows that the zone 2 curves do in fact satisfy this filter criteria.

USBR filter criteria are as follows for subrounded particles and reasonably well-graded filter materials (Earth Manual, 2nd edition, p. 307).

$$R_{50} = \frac{\text{50 percent size F.M.}}{\text{50 percent size B.M.}} = \text{12 to 58, and}$$

$$R_{15} = \frac{\text{15 percent size F.M.}}{\text{15 percent size B.M.}} = \text{12 to 40}$$

The average D_{15} and D_{50} sizes of zone 1 from figure 1 are approximately 0.006 mm and 0.03 mm, respectively. Using these values, the USBR range of gradations for a filter is shown on the grain size curves for TP-C2 and TP-C8. It can be readily seen that the zone 2 curves lie to the right of the implied range. These requirements are, as stated in the Earth Manual, "given as a guide for filters used in canal structures or other hydraulic structures involving high-water heads where rapid dissipation of uplift pressure is desired" (Earth Manual, 2nd edition, p. 306).

In addition, concerning filter width, the Earth Manual states as a guideline (p. 309):

> "(4) The filter layers for coarse filter material (3-inch maximum size) are usually not less than 8 inches in thickness, and layers of finer filter material are often of 6-inch minimum thickness. However, for severe field conditions such as high head, variations

in base material, or filter gradations which are near the extreme coarse limit, the minimum thickness of 8 inches may be specified. For zoned filters these minimum thicknesses may be specified and are maintained for each layer."

Clearly, the filter criteria given in the Earth Manual are developed for the extreme conditions of rapid dissipation of high hydraulic gradients with relatively thin layers of filter material with the implied possibility of filter surging.

The "* * * rationale on the need for designed filters at Teton" is that filters were definitely needed and that a thick zone of silty gravel which was close to USBR criteria and which did in fact meet the Terzaghi filter requirements would be used rather than a processed filter whose expense was unjustified. In addition, excessive exit gradients were not expected to be involved.

Cedergren, a generally accepted authority on seepage, states the following in his text (reference 1, p. 178):

> "If a protected soil is a plastic clay, the piping ratio often can be much higher than 5 or 10, as indicated by U.S. Army Corps of Engineers practice previously noted. But if cohesionless silts, fine sands, or similar soils are in direct contact with filter materials which have piping ratios much above 5 or 10, erosion is very likely to occur."

In this statement Cedergren's reference to "much above 5 or 10" is essentially allowing that the Terzaghi filter criteria may be relaxed. If the upper filter limit criteria of $D^F_{15}/D^B_{85} < 10$ are used for the Teton material, the D_{15} size of the filter may be as large as 1.22 mm. The range of D_{15} filter limits for these criteria ($D^F_{15}/D^B_{85} < 5$ to 10) is shown on the grain size curve of TP-C2 and TP-C8.

The following quotations (Justin, Greager, and Hinds, ref. 2, p. 690) demonstrate that it has been generally accepted by authorities in embankment dam design that bank run material may serve very adequately as a filter and a drain:

> "There are many cases where a single layer of run of bank sand gravel is all that is required to serve both as filter and drain. One should be sure that the sizing of the material will be such that impregnation will be insignificant."

They continue

> "In many cases a run of bank sand gravel may be used successfully, as in fig. 21. So long as the run of bank material contains the necessary range of sizes, it will make its own

filter if thick enough. Fine sand or gravel when placed next to stone will perhaps run right through the interstices in the stone, but the larger gravel will stick and a filter will thus be gradually built up. The first thing to be sure of is that the previous run of bank materials contains some particles that are of greater diameter than the biggest voids in the stone."

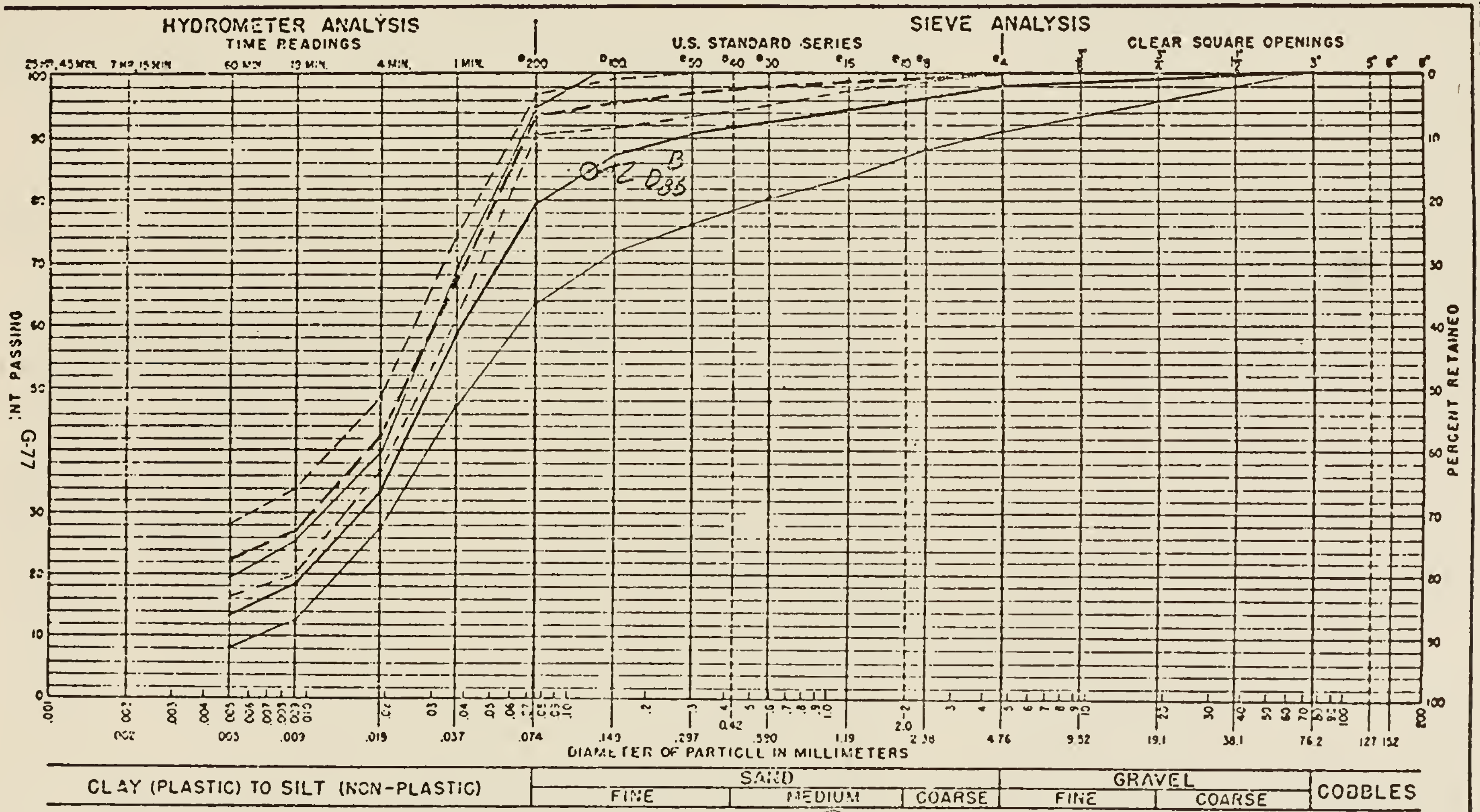
Reference 1 - "Seepage, Drainage, and Flownets," H. R. Cedergren, John Wiley and Sons, 1967

Reference 2 - Justin, Creager, and Hinds, "Engineering for Dams," John Wiley and Sons, 1945

(3-53)
Bureau of Reclamation

RUN BY RWK CHECKED BY __________ DATE 11-27-76

TETON DAM



NOTES:

GRADATION TEST

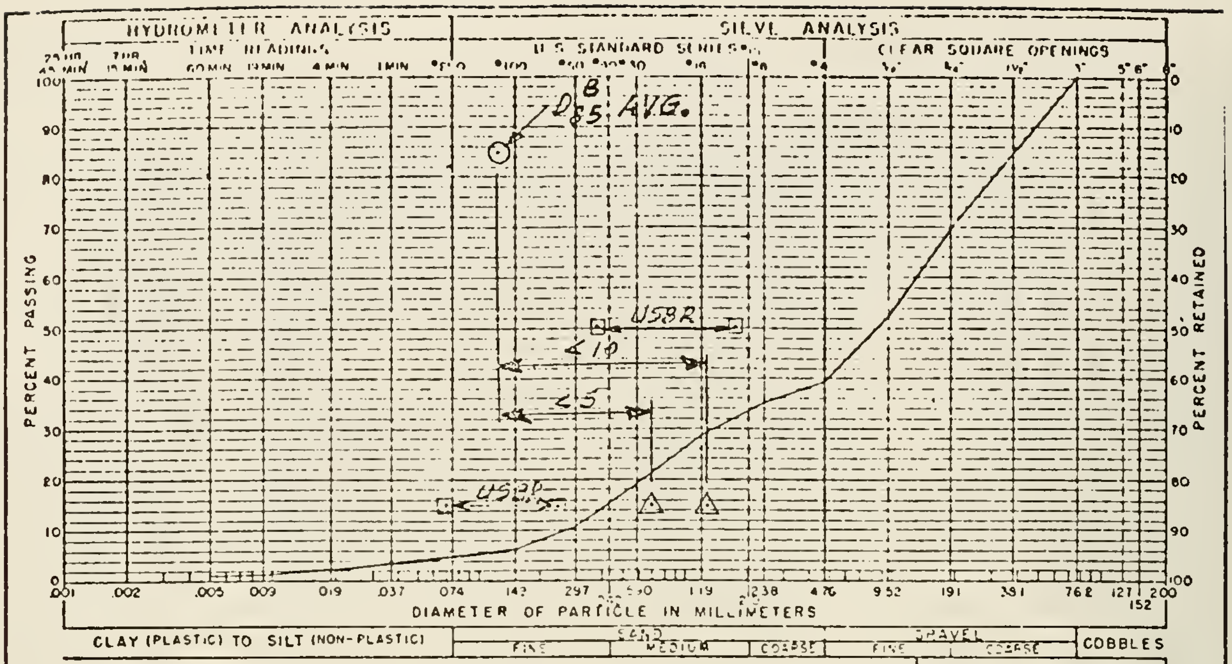
m+σ
m
m−σ
Mean & ± one standard deviation for tests of material from Borrow Area "A", 1969 (21 tests)

m+σ
m
m−σ
Mean & ± one standard deviation for tests of material from undisturbed samples of embankment from geophysical test hole DH-DNGP-1, 1975 (20 tests)

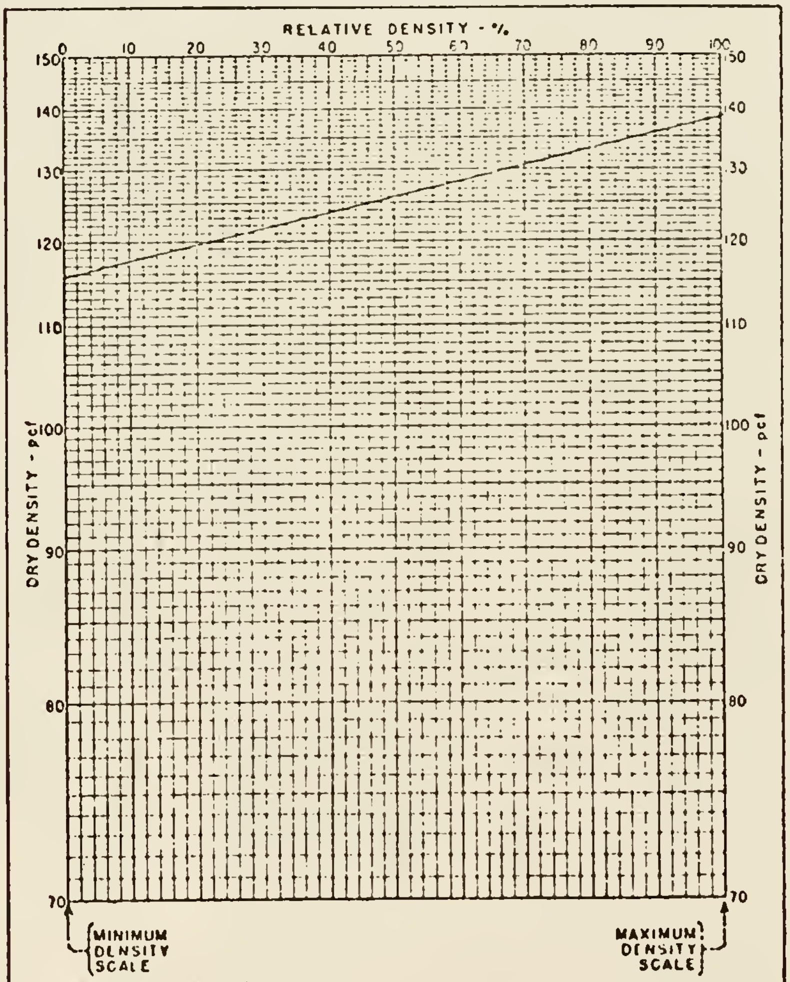
FIGURE 1

LABORATORY SAMPLE No. __________ FIELD DESIGNATION __________ EXCAVATION No. __________ DEPTH __________ FT.

STANDARD PROPERTIES SUMMARY



GRAVEL	60 %
SAND	36 %
SILT TO CLAY	4 %



LABORATORY SAMPLE NO ________

HOLE NO ________ DEPTH ______ ft

RELATIVE DENSITY VS DRY DENSITY
(Scaled to plot as a straight line)

STANDARD PROPERTIES SUMMARY

CLASSIFICATION SYMBOL	GP
SPECIFIC GRAVITY	
ATTERBERG LIMITS	
LIQUID LIMIT	
PLASTICITY INDEX	
SHRINKAGE LIMIT	
RELATIVE DENSITY	
MINIMUM DENSITY (P.C.F.)	116.6
MAXIMUM DENSITY (P.C.F.)	138.5
IN-PLACE DENSITY (P.C.F.)	
PERCENT RELATIVE DENSITY	
PERCOLATION SETTLEMENT	
PLACEMENT CONDITION	
PERMEABILITY (FT/YR)	
SETTLEMENT (%) UNDER ______ P.S.I. LOAD	

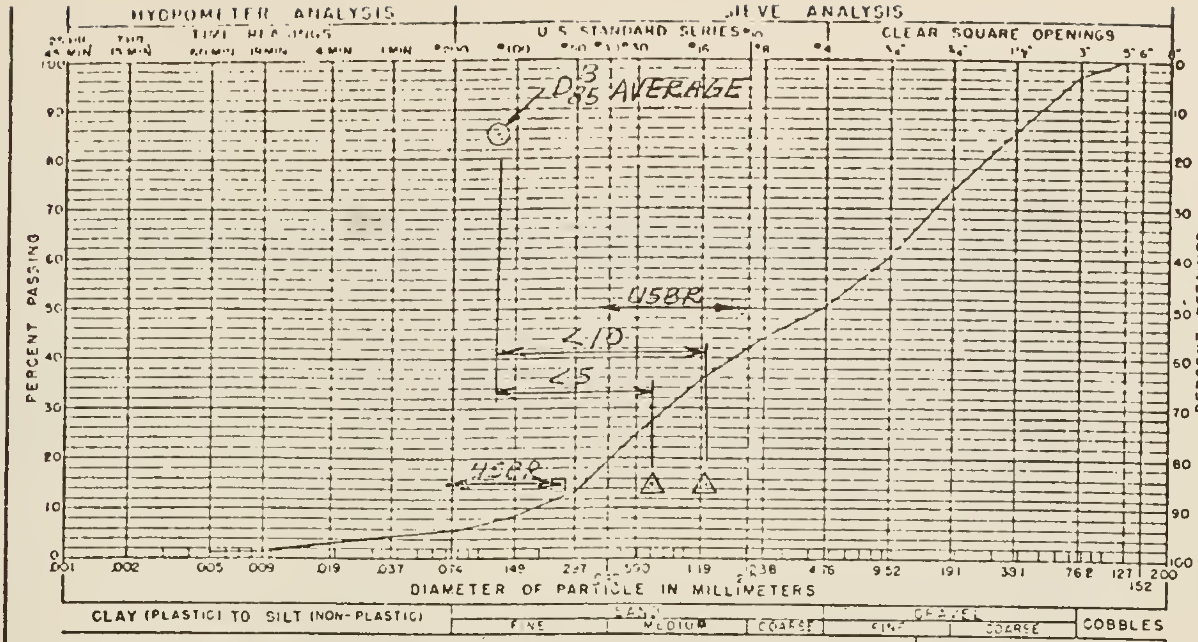
NOTES: ________________

FIGURE 2

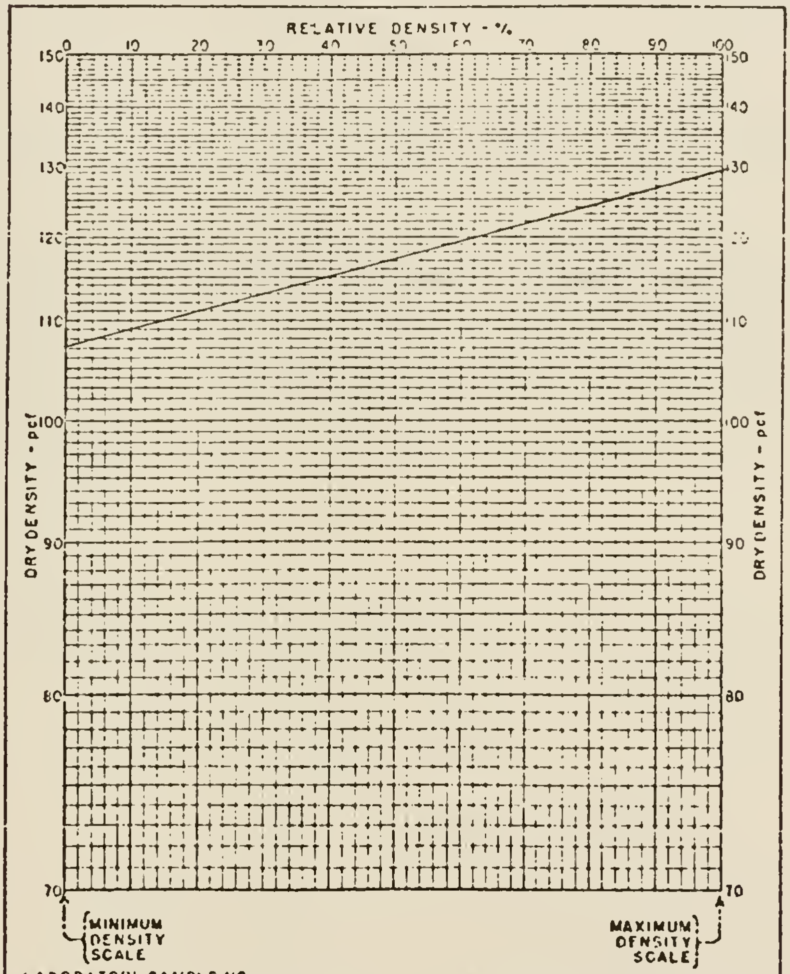
LABORATORY SAMPLE NO. 518-38 FIELD DESIGNATION ________ EXCAVATION NO. TP-C2 DEPTH 2.0 - 12.5 FT.

LWD 11/30/76

FIGURE NO. 20



GRAVEL	50 %
SAND	44 %
SILT TO CLAY	6 %



LABORATORY SAMPLE NO ________

HOLE NO. ________ DEPTH ______ ft

RELATIVE DENSITY VS DRY DENSITY
(Scaled to plot as a straight line)

STANDARD PROPERTIES SUMMARY

CLASSIFICATION SYMBOL GP - GM

SPECIFIC GRAVITY ________

ATTERBERG LIMITS

LIQUID LIMIT ________

PLASTICITY INDEX ________

SHRINKAGE LIMIT ________

RELATIVE DENSITY

MINIMUM DENSITY (P.C.F.) 107.3

MAXIMUM DENSITY (P.C.F.) 129.2

IN-PLACE DENSITY (P.C.F.) ________

PERCENT RELATIVE DENSITY ________

PERCOLATION SETTLEMENT

PLACEMENT CONDITION ________

PERMEABILITY (FT/YR) ________

SETTLEMENT (%) UNDER

______ P.S.I. LOAD ________

NOTES: ________

FIGURE 3

LABORATORY SAMPLE NO. 510-39 FIELD DESIGNATION ________ EXCAVATION NO. TP-C9 DEPTH 1.5 - 4.2 FT.

LWD 11/30/76

GPO 838-102 FIGURE NO. 21

Question No. 8

Since much of zone 3 material is the same as zone 1 silt, why wasn't a filter layer included between zone 3 and the zone 5 rockfill?

ANSWER

The zone 2 pervious material upstream from the zone 3 would control the phreatic line and prevent saturation of the zone 3. No water, other than precipitation, was available to carry zone 3 material into zone 5; therefore, a filter was not required.

Question No. 9

Surface treatment, by concreting or nonpressure grouting of joints, fissures, and openings in the rock beneath zone 1 was not a requirement in the construction specifications. Was surface treatment of such rock defects considered unnecessary or were the defects unknown to the designers? Explain the following statements from the Design Considerations for Teton Dam:

a. Page 9, paragraph IV.A. "Erosive seepage under the embankment will be eliminated by injecting the foundation with a grout mixture."

b. Page 10, paragraph IV.A. "To cover exceptional conditions, the specifications also provide for blanket grouting so that the curtain can be reinforced in areas where the near surface rock in the bottom of the cutoff trench or key trench contains open joints or cracks or other foundation defects."

Considering the condition of the near surface rock, it does not seem credible that required treatment not be included in the specifications.

Submit evidence that the design considered use of gravity grouting of fractured rock. Also, forward instructions provided to the project engineer on the type, extent, location, and other information about the purpose and determination of need. List the types, methods, etc., of rock treatment from which selection was to be made. What basis was to be used for selection of one method over another? Submit evidence of the guidance provided by design to the construction office.

ANSWER

Answer to First Part of Question in Sentence No. 2 "Was the surface treatment of such rock defects considered unnecessary * * *":

The surface treatment of any rock defects was considered necessary. This is made clear by the Design Considerations, paragraph IV.I, page 16 which is quoted below:

"I. Open Joints, Cracks, and Springs

Open joints or cracks found in the bottom of the foundation key trench and cutoff trench are 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. Springs may be treated in a similar manner. However, if considerable water is involved, it may be necessary to extend grout pipe or dewatering pipes through the fill from gravel drains until the embankment level permits sufficient grouting pressure to seal the spring. This would usually be part of the contractor's dewatering obligation."

Answer to Second Part of Question in Sentence No. 2 "* * * or were the defects unknown to the designers?":

The above quote from the Design Considerations in conjunction with the facts that (1) 17,824 feet of core in 102 preconstruction drill holes were obtained, (2) a test grouting program was performed in 1969, and (3) extensive pump-in tests on the right abutment were conducted in 1970 make it clear that the Bureau was well aware that there were defects in the foundation which would require treatment.

Answer to Third Sentence, Part a and Part b

The statement in the Design Considerations, page 9, paragraph IV.A. "Erosive seepage under the embankment will be eliminated by injecting the foundation with a grout mixture." is under the heading entitled "Foundation Pressure Grouting" and is in its first section entitled "General." This statement is the first sentence in the first paragraph of this section and is simply trying to give the field personnel a general feeling as to the function and rationale of the grout curtain. The term "erosive seepage" was meant to be a general concept and could just as well have meant that a high flow of water under the foundation would erode the downstream alluvial foundation or would erode the zone 4 berm at the downstream toe. It was not the intent of the designers to imply that the grout curtain was substituting for surface treatment.

The statement on page 10, paragraph IV.A. "To cover exceptional conditions, the specifications also provide for blanket grouting so that the curtain can be reinforced in areas where the near surface rock in the bottom of the cutoff trench or key trench contains open joints or cracks or other foundation defects," is again under the heading entitled "Foundation Pressure Grouting" and under the section entitled "General." Here the reference is to a specific area, namely, "the near surface rock in the bottom of the cutoff trench or key trench." This was to provide a vehicle for ensuring that the grout curtain was effective.

Question No. 9 continues with "Considering the condition of the near surface rock, it does not seem credible that required treatment would not be included in the specifications."

ANSWER

The Bureau considers it difficult and many times impracticable to specify the exact treatment of a foundation without examining the foundation firsthand. The Bureau's general policy is to have design, construction, and field personnel examine the foundation prior to placement of zone 1 material and to decide at that time the method of surface treatment. Whether the surface treatment should be specified at the time of the cutoff trench inspection or whether it should be included at the time the specifications are written is debatable.

Question No. 9 continues: "Submit evidence that the design considered use of gravity grouting of fractured rock."

ANSWER

To the best of our knowledge there are no travel reports or design notes that specifically say the designers considered gravity grout. However, the Bureau considers the fact that it did use gravity grout as prima facie evidence that it did consider gravity grouting in the design.

Question No. 9 continues: "Also forward instructions provided to the project engineer on the type, extent, location, and other information about the purpose and determination of need for surface grouting.

ANSWER

In October of 1973 when the left abutment key trench was opened for inspection, a meeting between the designers, the geologists, and the construction personnel was held in the field to discuss how key trench walls would be treated. Discussions between the designers, the geologists, the construction liaison personnel, and the field personnel resulted in a verbal agreement that gravity slush grouting would be the most reasonable and economical method of treating the surface of the key trenches.

To illustrate the fact that group decisions involving design, construction liaison, geology, and field personnel were standard policy and that instructions, albeit many times verbal, were given to field personnel, a memorandum dated March 14 from the field and a travel report of March 29, 1974, concerning examination and treatment of the voids encountered in the right abutment key trench are appended.

Question No. 9 continues: "List the types, methods, etc. of rock treatment from which selection was to be made."

ANSWER

The following types of surface rock treatment were considered:

1. Sealing and guniting as the embankment was raised
2. Shotcreting
3. Concrete wall facing
4. Slush or gravity grouting
5. Blanket grouting upstream and downstream from the key trench faces
6. Filter construction along the downstream face of the key trench

Question No. 9 continues: "What basis was to be used for selection of one method over another?"

ANSWER

The basis for the method of selection was verbal discussion of the pros and cons of each method by experienced personnel from design, construction liaison, geology, and the field.

Question No. 9 continues: "Submit evidence of guidance provided by design to the construction office."

ANSWER

Guidance provided by design came in three forms: (1) by verbal discussion with the field personnel when design personnel visited the site, (2) by telephone contact with the field personnel, and (3) by travel reports written after visiting the damsite.

As an example of "guidance provided by design personnel to the construction office" a travel report dated November 23, 1973, which was written after a review of the cutoff trench excavation prior to placement of the zone 1 embankment, is attached.

Paragraph 4, page 2 of this report specifically gives guidance in how to treat "the large mass of loosely jointed, blocky rock on the downstream side of the left abutment trench."

The report gives guidance (which was later revised) by stating "The first condition can be treated by sealing and guniting in selected areas as the fill is placed."

BUREAU OF RECLAMATION

TETON PROJECT OFFICE
P.O. BOX 88
NEWDALE, IDAHO 83436

IN REPLY
REFER TO:

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 questionnable; 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:

Work or Material	Quantity	Unit	Price	Amount
1. Mobilization and demobilization of drill equipment	For the lump sum of			1,000.00
2. Air-trac pilot holes	300	lin.ft.,	5.00	1,500.00
3. Ten-inch diameter holes with 8-inch casing	200	lin.ft.,	35.00	7,000.00
4. Backfill concrete	350	cu.yds.,	30.00	10,500.00
5. Block cavern entrances	For the lump sum of			1,000.00
Total estimated cost				$ 21,000.00

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.

Robert R. Robison

Enclosures

cc: Regional Director, Boise, Idaho
Attn: 200 w/Enc.

<u>Note to Regional Engineer:</u>

We would appreciate having a member of your staff present during the visit of the Denver Office representatives to the Teton Project.

Denver, Colorado

JaK 4/10

TRAVEL REPORT

Code: 1300, 220, 230 Date: March 29, 1974

To: Director of Design and Construction

From: L. R. Gebhart, W. G. Harber, Engineers, J. D. Gilbert, Geologist

Subject: Examination of right abutment key trench - Teton Dam and Power and Pumping Plant - Specifications No. DC-6910 - Morrison-Knudsen Company, Inc., and Peter Kiewit Sons' Co. - Teton Basin Project, Idaho

1. Travel period (dates): March 20 to March 21, 1974.

2. Places or offices visited: Teton Project Office and damsite, Newdale, Idaho. We met with Project Construction Engineer Robison, Field Engineer Pillis, Engineer Aberle, and Geologist Sweeney from the project, and Engineer Kolterman, Regional Geologist Farina, and Geologist Magleby from the Regional Office.

3. Purpose of trip (include reference to correspondence prompting travel): To examine voids exposed in the right abutment key trench between Stations 3+45 and 4+44. Travel prompted by correspondence from Project Construction Engineer dated March 14, 1974.

4. Synopsis of trip: On Wednesday p.m. March 20, we met with the project and regional people noted above, and examined the right abutment key trench. On Thursday a.m. March 21, we examined the auxiliary outlet tunnel, the progress of work on the powerplant, and re-examined the right abutment cutoff trench. Engineer Harber examined the proposed riprap source, borrow pits, and embankment during this time. Thursday p.m. we met with key project personnel and discussed our observations.

Construction status: Spillway excavation was continuing between Stations 32+80 and 33+60. The excavated material covers the downstream portal of the auxiliary outlet works tunnel. Forming for the initial concrete placement for the auxiliary outlet works intake structure has begun. The contractor plans to line the first 60 feet of tunnel from the upstream portal prior to high water this spring. Whereas the auxiliary outlet works tunnel was dry when excavated, there is evidence that surface water has since seeped into the tunnel in several areas. One area between approximate Stations 12+25 to 13+75 had numerous icicles hanging from the tunnel roof. The shaft, which had been raise bore drilled to a diameter of 11 feet, had not been inspected all the way but the quality of the rock looked good from the bottom and at the top.

Dewatering at the damsite continued from most of the 14 wells located upstream and downstream of the cutoff trench. When weather permits, the contractor will be placing Zone 1 embankment from Borrow Area A, where a Barber-Greene wheel excavator will be loading 100-ton belly dumps to haul to a conveyor and drop hopper system on the right abutment. The conveyor system is designed for delivering 2,700 tons of Zone 1 material per hour. An effective method of adding water to obtain the required moisture content remains to be determined.

Various phases of work were being stepped up in the power and pumping plant area, with concrete placements being resumed. Visqueen covers and a central heating furnace assist in protecting the concrete placements. The concrete walls in the plant have been sack-rubbed. The forming and placement of the concrete were of excellent quality. Minor shrinkage cracks running transverse to the long dimensions were evident in some of the gallery room ceilings. The project expects completion by the target date of July 1974.

The project was waiting for laboratory test results of the proposed new riprap source. Subsequent to our visit, the test results appear favorable. Negotiations with the contractor for new higher prices because of the increased haul distance will follow official approval.

The drilling and grouting subcontractor has not returned with plant and equipment from his home base at Idaho Falls to continue foundation grouting operations.

Geologic observations:

a. Two intensely fractured zones containing extensive voids and fissures are present near final grade in the right abutment key trench. These zones, which cross the dam axis at about Station 3+55 and Station 4+30, strike approximately N 80° W, dip steeply to the north, and range from about 1 foot to 5 feet wide. Observed voids associated with these zones generally range from 1 foot to 3 feet in width, between 3 feet and 50 feet or more in height, and extend up to 70 feet into the abutment. The intensely fractured zones are overlain by a later welded tuff deposit, which has not been extensively fractured or altered. Generally, only a single joint can be observed extending from the intensely fractured zones through this upper deposit. The intensely fractured zones consist of:

(1) Intensely fractured, in-place rock separated by variably spaced joints whose surfaces have been altered by the action of hot gasses and water vapor. The alteration products make up less than 5 percent of the observed rock.

(2) Voids and small fissures, locally quite extensive. The wall rock in these voids has been altered to depths of 1/2 inch and is generally friable. Calcite deposits are common.

(3) Collapse rubble. Various size fragments from the roof and walls of the fissures.

b. The intensely fractured zones and fissures are not the result of faulting or other tectonic movements. No evidence of fault displacement or other fault-related features, such as drag folds and slickensides, was observed. No folding or buckling of the welded tuff was observed.

c. The fissures are believed to have originated as tensional cooling cracks that have been modified by ascending hot gasses and water vapor. Progressive alteration of the wall rock bounding these fractures reduced the welded tuff in portions of the zone to a fine sand-like material. This fine material was probably subsequently removed by circulating ground waters, resulting in the collapse of the zone along part of its extent, and the formation of extensive voids.

d. Preconstruction investigations indicated that intensely fractured zones with high permeability existed in this area, but no voids were encountered during drilling, nor were any observed during television logging. Preconstruction drill holes indicate, however, that conditions similar to those exposed in this portion of the keyway trench should be expected below final grade in this area, and also beyond the end of the right abutment. Previous and present searches of the canyon walls have produced no evidence that the fissures extend to the canyon for direct contact with the reservoir.

3. Conclusions:

a. Treatment should proceed as described in the project's memorandum dated March 14, 1974.

b. Angled, 3-inch-diameter, noncored exploratory drill holes should be drilled from the key trench floor to probe for suspected voids beneath the trench floor.

c. The project will document the nonfaulted nature of these fissures in the final construction report for the key trench.

6. Recommendations: None.

L. R. Gebhart
WGHarber

Copy to: Regional Director, Boise, Idaho
Project Construction Engineer, Newdale, Idaho

Blind to: 210
222
230
1300
1305

LRGebhart/WGHarber/JDGilbert:cmd

NOTED: APR 9 1974

H. G. Arthur
DIRECTOR
DESIGN AND CONSTRUCTION

DFC-40A (8-70)
Bureau of Reclamation

BUREAU OF RECLAMATION
Engineering and Research Center
Denver, Colorado

File

TRAVEL REPORT

Code : 230, 222

Date: November 23, 1973

To : Director of Design and Construction

From : Ira E. Klein and R. W. Bock

Subject: Review of Construction - Teton Dam and Power and Pumping Plant - Teton Basin Project, Idaho

1. Travel period (dates): October 16 - 17, 1973

2. Places or offices visited: Teton Project Office and damsite, Newdale, Idaho

3. Purpose of trip (include reference to correspondence prompting travel): Telephone call from Project Construction Engineer, Teton Dam, October 12, 1973, requesting a review of the cutoff trench excavation prior to placement of Zone 1 embankment and to observe other portions of the work.

4. Synopsis of trip: In the morning of October 16, after preliminary office discussion with the Project Construction Engineer, we examined excavations in the channel and both abutment sections, the auxiliary outlet works tunnel, and the spillway. On October 17, we visited the original and prospective new riprap quarry sites (including inspection of the drill core from the proposed site), inspected the powerplant and outlet works areas with special attention to the drainage and slope stability conditions on the left side of the valley, revisited the channel cutoff, and met with Project Construction Engineer and Field Engineer to review the overall situation. In the course of travel on both days, with the assistance of Project Geologist Sweeney and Acting Regional Geologist Magleby, observations were made related to the structural geology and physiography of the site with regards to any evidence of geologically recent fault activity and on potential reservoir seepage.

Channel Section

The grouting had been completed prior to our visit and the final cleanup was in progress, so that both the rock foundation and the alluvial deposits could be seen. As anticipated from the preconstruction investigations, basalt formed the left portion (about one-third) of the channel section and rhyolite the right portion. On the right side, natural irregularities in the rhyolite, which included some overhangs, had been shaped by presplit blasting to stable steep slopes. On the left side, the basalt had been eroded to remarkably smooth-curved clean slopes of hard fresh dense rock which required minimal cleanup. The flatness of the channel bottom over a 300-foot length in both the hard basalt and softer rhyolite was outstanding. There were no erosional slots, crevices, or potholes. The bottom elevation ranged from elevation 4931 to 4934 feet. The physical condition of the rhyolite, as well as the basalt, was good. There were no faults or shear zones. Both the low-dipping, flow-bedding joints and steep tensional cooling joints were predominantly tight, so that grouting and removal of loose joint blocks were readily accomplished.

Drainage of water not already intercepted by well points in the gravelly channel alluvium was being effectively handled by pumping from shallow toe drains. The lowermost cut slopes, which were excavated in a 15-foot-thick layer of finer-textured alluvium, were variably moist to wet and locally seeping. Although a little rock had been placed on the upstream right corner because of some signs of incipient instability in this finer-textured alluvium, slope stability was good at the time of this inspection and the outlook favorable. In the preconstruction investigations the occurrence of some clayey and silty sediments in the lowermost part of the channel fill had been indicated by DH-101 and DH-102. When the channel excavation had gone through about 60 feet of gravel, the fine-textured basal member of the alluvial fill was reached. Nine auger holes were then drilled to depth of 15 to 28 feet to determine its configuration and characteristics more completely than was practicable in the preconstruction investigations. The basal fine-textured channel fill deposit is predominantly a borderline ML-CL soil with an appreciable content of fine sand. Stringers and lenses of sandy and gravelly soil are present throughout the deposit, as indicated in the cutoff slopes by local seepages. These stringers and lenses are most obvious toward the top and bottom of the unit. The contact with the overlying gravel was irregular with several feet of relief. The distribution of the various alluvial soils was recorded as the excavation progressed by systematic geologic mapping supported by laboratory classification of samples. These data will be compiled in detailed maps and sections.

Left Abutment Area

Excavation and grouting in the cutoff trench are nearly completed. The work in the diversion tunnel and outlet works facilities is in an advanced state. The grouting has been recently reported on by Mr. Gebhart based on his inspection on October 8-11. The following remarks pertain to some localized conditions requiring further attention.

These are: (1) The large mass of loosely jointed, blocky rock on the downstream side of the left abutment trench, (2) high grout takes and incomplete closure in the fan holes at the end of the abutment, and (3) cavities in the uppermost part of the left abutment trench between Station 33+10 and Station 34+00. The first condition can be treated by sealing and guniting in selected areas as the fill is placed. The second and third are believed to be geologically related. The cavities are believed to be fossil fumarole vents along which steam rose through the rhyolite tuff from water trapped on the erosional surface buried by the volcanic ash flow. In this part of the foundation, irregular chimney-like patterns of caverns following the steep cooling joints appear to extend from the base to the top of the tuff. Fortunately this is the only part of the dam where this geologic phenomenon has been found. Plugging of these voids from the surface and more grouting in intermediate holes to obtain closure along the centerline curtain is planned; some investigations to determine if the cavernous condition persists to a significant extent beyond the end of the dam is needed. Another noteworthy aspect of the foundation geology was local large grout takes, associated with strong back pressures, in the "lakebed" formation just beneath the rhyolite. Where this formation was encountered in the initial grouting, these conditions appeared to represent a problem especially if the condition was a general one. Fortunately it has proved to be highly localized in the left abutment and channel section. Typically the buried sedimentary formation and the contact zone with the rhyolite, confirming the preconstruction investigations, have been fairly tight in the grout holes that have gone this deep.

Right Abutment Area

Excavation in the cutoff trench and auxiliary outlet works tunnel was quite advanced and considerable work had been done on the spillway. Scaling was in progress on the downstream side of the upper part of the cutoff trench (north of the spillway). In this area, which was near the natural top of the rhyolite, the rock is notably different than in the lower part of the abutment. Beneath the soft ashy top layer, which was treated as overburden, there is a lightweight "clinkery" layer about 10 feet thick. This is underlain by a distinctive thin-bedded zone. Both zones appear fairly loose jointed and extend into the upper part of the spillway chute, which was roughly excavated to a few feet above grade. The occurrence of these materials could affect the placement of anchor bolts in parts of the chute bottom and possibly increase grout takes in the crest area. However, concerning the latter, a pilot core hole for the auxiliary outlet works shaft indicated promisingly favorable conditions from the surface down. Large water losses were only found in a few points and they were below depth of about 140 feet.

Rock exposed in the auxiliary outlet tunnel was inspected at both portals and in the tunnel downstream of the gate chamber where excavation of the adit to the shaft was in progress. The portals were in stable 1/4:1 presplit cuts, excavated in sound, widely jointed rhyolite supported with some rock bolts. The tunnel, which was dry and still unlined, was generally unsupported except for steel sets for short intervals at each portal and rock bolts in selected places such as intervals where the flow bedding-type of joints were present in the arch. The most prominent joints were in a near-vertical set that trended obliquely across the tunnel. These joints generally had 1/8-inch to 1/4-inch brownish clay filling, but occasionally had open unfilled portions. Detailed data on these and other structural features are being recorded in the geologic tunnel logs. This sort of partially clay-filled jointing is evidently more frequent in this tunnel and presumably also elsewhere at depth in the right abutment, than generally was found in the diversion tunnel and left abutment area. This could be a significant factor in the relative amounts of grouting that will be required.

No cavernous-type voids such as occur in the end of the left abutment cutoff were encountered. It is also noteworthy that no faults, or even evidence of minor slippage on any of the prominent high-angle joints, (which apparently had a tensional cooling origin) were seen.

Possible Faults and Potential Reservoir Leakage

Special attention was given during the inspection of the dam and the surrounding terrane to evidence of possible faults near the damsite and potential reservoir leakage. We found that the categorically negative picture on faulting, pointed out in the remarks on the auxiliary outlet tunnel, applies equally to the several thousand feet of well-exposed rock in the channel section and abutment cutoff trenches, spillway cut, and portal cuts which we inspected, and also the diversion tunnel, powerplant foundation, etc., areas which had been mapped in detail by Geologist Sweeney. Furthermore, evidence of geologic structures and minor discontinuities (e.g. shear zones or sheared joints and tectonic flexures) are also not seen. This confirms our extensive preconstruction investigations by core holes and surface mapping. The foregoing is important because ongoing regional environmental studies by the USGS have led parties in that agency to express opinions based on preliminary studies (made available to USBR by letter of July 20, 1973, transmitting a draft - Preliminary Report on Geologic Investigations, Eastern Snake River Plain and Adjoining Mountains - June 1973) that geologically recent faulting with current seismic implications are common in the Rexburg Bench, and furthermore a fault with possible large lateral extent is postulated to lie on the north side of the reservoir within a few hundred feet of the end of the right abutment cutoff trench. Evidently, this fault is inferred in the dam vicinity from a northeast-trending, low, smooth escarpment in the aeolian, silt-covered plateau. This topographic feature lies between preconstruction core holes DH-5 and DH-6, at the north end of the area covered by subsurface investigations for potential reservoir leakage. Neither of these deep inclined core holes, which are now part of the permanent ground-water observation well net, found evidence of faulting. An additional intermediately located core hole is proposed to resolve the question of this inferred fault. It would also be useful to check on grouting at this end of the dam.

Concerning the related matter of seismicity, the USGS in the preliminary report referred to above, in our opinion, places too much emphasis on meager data. The Bureau's microseismic survey of the entire reservoir vicinity, in cooperation with the National Center of Earthquake Research (USGS) and the Geology Department of Ricks College, which should be in operation in 1974, will provide basic data that can be used in coordination with local and regional geologic mapping in refinement of our evaluation of earthquake risks related to the project.

The specifications riprap source was stripped and quarry operations started by the contractor in May 1973. There were 15,000 cubic yards produced to provide riprap for the river outlet works. Due to the high percentage of waste produced and inadequate space between the river and the access road alternate sites are being investigated. The most favorable is the Hobbs Site No. 2 located about 3-1/2 miles north of the Teton River. Six core drill holes indicate 28 to 48 feet of good, massive, locally slightly fractured vesicular to dense basalt in one flow. The Project Construction Engineer was authorized to negotiate with the contractor to perform a blast test at this site to determine the size and quality of the rock which can be produced.

5. Conclusions:

a. The foundation geology in the bottom of the channel section cutoff was highly favorable for efficient earthfill placement. The ground-water drainage and slope stability conditions were also satisfactory.

b. The deep cutoff trench design in the abutments is proving successful in eliminating the bulk of the open-jointed rock mass in which grouting would have been very difficult.

c. In the upper part of the left abutment cutoff trench plugging of exposed cavities and additional grouting in the fan hole pattern at the end of the abutment are required. The surface cavities and pervious conditions at depth in this part of the foundation appear to be geologically related. To better understand the geologic conditions for both final construction and future operational needs, one or two deep core holes which can be made part of the permanent project ground-water observation net, should be planned a short distance past the present end of the cutoff trench.

d. In the right abutment area geologic features which were observed that are of some significance to construction are the loose-jointed, thin-bedded structure of the rhyolite in the upper part of the partially excavated spillway chute and the open character of part of the steep joints at depth as indicated in the auxiliary outlet works tunnel.

e. There is no evidence of faults in the several thousand feet of excavations in any part of the dam and appurtenant works. The existence of a fault, inferred from preliminary environmental geology studies of the USGS to be located a short distance north of the dam, is questionable. To resolve this it is proposed that a core hole, which can be incorporated in the permanent ground-water observation net, be located near the right end of the dam.

f. The alternate Hobbs No. 2 basalt riprap quarry site is being investigated. The completed geologic exploration indicates that under the local terrain conditions, which are conducive to efficient quarrying operations, sufficient rock is available for the dam construction. A blast test to determine the size and quality of the rock is being performed.

6. Recommendations: None

Isa. E. Klein

R. W. Bock

Copy to: Regional Director, Boise, Idaho
Project Construction Engineer, Newdale, Idaho

Blind to: 210
222
230
1300

IEKlein/RWBock:jdm

NOTED: DEC 3 1973

D. J. DUCK

DEPUTY DIRECTOR
DESIGN AND CONSTRUCTION

Question No. 10

Did Design participate in deciding to treat the rock defects under zone 1? Did they take part in the decision not to treat the rock above approximately el 5200?

Because there were construction photographs which show cracks above elevation 5200 which would appear to accept gravity grout and because some of the field personnel thought that surface grouting of some voids might have been continued above the point where the grouting was terminated, why did the Bureau not grout these cracks?

Furnish copies of trip reports, memorandums, or other documentary evidence that is available on design visits and/or design modifications during construction based on field conditions which justify not grouting specific cracks.

What is considered minimal voids and open cracks in rock? Does the Bureau consider any open crack beneath a highly erosive silty fill material acceptable?

Provide the purpose, reasoning, and criteria used in determining the need for surface grouting as a part of abutment surface preparation prior to placement of the earth fill.

a. What was the purpose intended during the design consideration of need?

b. What purpose was intended by the project construction staff? By the inspectors?

c. Did the designers concur in the methods, procedures, and criteria used in placing the gravity grout as determined by the project staff? Were they aware in detail of the procedures used?

d. What was the specific criteria on determining when a crack would be grouted? Who made the decision on which crack to grout or not to grout?

ANSWER

Answer to Question in Sentence No. 1

The designers did participate in the decision to treat the rock which underlay zone 1 embankment.

Answer to Question in Sentence No. 2

The designers did not make the decision not to treat the rock above approximately elevation 5200. The decision not to treat the rock above this elevation was dictated by the rock conditions. Above this elevation, the openings were minimal, horizontally oriented, and rubble and silt infilled. This type of rock and the joints were not conducive to gravity grouting.

Answer to Comment and Question in Sentence No. 3

Bureau design, construction, and geology personnel were in agreement that the variability in the fracture and jointing patterns in the formation could be treated more adequately by someone examining the joints in the field than by an arbitrary set of rules devised in the Denver Office.

Crack treatment was based on the following items: width of the crack, whether the crack was infilled, the type and degree of compaction of the infilling, and whether the crack was open to depth. In the case referred to by the question, it may well be that the crack was open at the surface and closed or infilled at a shallow depth.

Answer to Question in Sentence No. 4

There are, to the best of our knowledge, no travel reports or memorandums which justify "not grouting specific cracks." As an indication that the Bureau was conscientious in treating any condition which was deemed by project personnel or E&R Center engineers to be a potential hazard, the travel report and project letter dealing with the treatment of the right abutment fissures are submitted. (These reports are appended to question No. 9.)

Answer to Question in Sentence No. 5

There are no "minimal voids" or "minimal open cracks"; there are only cracks or joints which, in the opinion of the projecc personnel inspecting them, should or should not be treated based on site inspection. In the Bureau's response to the August 18, 1976 Independent Panel request for information (also transmitted to the Review Group), the width of joints which were less likely to receive treatment was listed as "approximately 1/2 inch" whereas in the project personnel interviews taken by the Review Group grouting subcommittee (dated August 17-19, 1976), Mr. Ken Hoyt referred to "1/4-inch" widths.

Answer to Question in Sentence No. 6

The Bureau does not consider any open crack beneath a highly erosive, silty fill material which will accept neat cement as "acceptable." The Bureau considers that any open crack in the formation under the core zone of the embankment should be treated. This treatment may take the form of grouting, dental concrete, slush grouting, or special compaction of zone 1 material, depending on the judgment of the field personnel involved. As an indication that the Bureau does consider open crack treatment important, a tabulation of the locations and amounts of slurry grout used to fill cracks and fissures in the right abutment is attached.

Answer to Question in Sentence No 7

The purpose of surface grouting was to prevent the migration of fines from the zone 1 core into the foundation. The reasoning used in "determining the need for surface grouting" was that any joint which appeared by visual inspection of on-site personnel to require treatment was to be treated. The criteria for surface crack treatment is given below:

a. The purpose of surface grouting as intended by the designers was to prevent any migration of zone 1 material through or into the foundation. Paragraph IV.I of the Design Considerations, p. 16, clearly expressed this intent and is quoted below:

"I. Open Joints, Cracks, and Springs

"Open joints or cracks found in the bottom of the foundation key trench and cutoff trench are 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. Springs may be treated in a similar manner. However, if considerable water is involved, it may be necessary to extend grout pipe or dewatering pipes through the fill from gravel drains until the embankment level permits sufficient grouting pressure to seal the spring. This would usually be part of the contractor's dewatering obligation."

b. The purpose of surface grouting as intended by both the construction staff and inspectors was essentially similar to that of the designers in Denver, namely, to treat all cracks that were open so that no zone 1 material could migrate into or through the foundation.

c. The designers did participate in the selection of and concurred with the general method of surface treatment. They also concurred with the general procedures and criteria used in placing the gravity grout. It was understood that specific procedures and criteria for surface treatment would be developed in the field.

The designers did not, however, make regularly scheduled site investigations to check the continuing adequacy of the field procedures. They did receive monthly reports from the field office as well as the travel reports by E&R Center personnel and were in verbal and written contact with construction liaison and project personnel.

d. The specific criteria used in determining how a crack should be treated are as follows: where an inspector examined the cleaned foundation and, in his opinion, judged that a crack should be grouted, that crack was grouted. This decision was influenced by the width of the crack, whether that crack was infilled, the type and degree of compaction of the infilling, and whether the joint was open.

The decision to grout or not to grout at Teton was made on the site by the inspector who was examining the specific crack.

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

	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

,c	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

Question No. 11

Stripping specifications for the abutments did not require removal of any in situ impervious soils prior to placement of zone 2 blanket drain. Roughly 50 percent of the rock in the abutments was covered by silt when the blanket drain material was placed. What design considerations were given to:

a. Partially blocked access to blanket drain for seepage exiting from the rock abutments? Isn't there a good possibility that these impervious silts could have been eroded by flow from joints? At places where open joints come in contact with the silts it seems unlikely that all of the joint flow would have to be confined to the joints themselves.

b. Partial blockage creating concentrations of seepage flows to impinge directly onto the blanket drain materials in localized areas?

c. Localized and channelized flow of seepage exiting from the rock abutments which might result in washout of the silt beneath the blanket drain or migrating of the silt into rock openings? In this very pervious abutment, flows large enough to result in washing of silt beneath the gravel blanket into rock openings probably should have been expected.

ANSWER

Answer to Part a, Sentence No. 1

The Bureau designers considered it possible that a partially blocked access to the blanket drain might occur for seepage exiting from the abutment rock; however, it was assumed the large surface area contact of the downstream zone 2 blanket and the rock jointing would result in redistribution of this seepage to other parts of the blanket with no harmful pressure buildup in the abutments.

Answer to Part a, Sentence No. 2

There is a possibility that the silt along the abutments could have been eroded into the blanket. However, it is extremely difficult to visualize the erosion of the silt into the zone 2 material since, as noted in the response to question No. 7, zone 2 satisfied the Terzaghi filter criteria. As the Review Group has pointed out, the abutments were fractured and jointed and this would have made it difficult for pressure to have built up within the abutments other than by massive seepage losses. Additionally, even if the silts beneath the zone 2 had eroded it should only have led to a localized problem of slumping.

Answer to Part a, Sentence No. 3

The Bureau agrees with the general nature of the statement.

Answer to Part b

Designers considered that if concentrations of seepage flows occurred locally due to partial blockage, these concentrated flows would quickly be dissipated either in the joint system or by the zone 2 blanket.

Answer to Part c, Sentence No. 1

The possibility of washout of silt beneath the blanket zone 2 material and excessive migration of the silt into rock openings were expected to be a minor consideration since (1) the hydraulic exit gradients at the contact zone were expected to be very small, (2) the zone 2 material was considered to have sufficient fines to prevent significant migration of silt particles, and (3) seepage pressure would be expected to dissipate rapidly in the jointed rock formations.

Answer to Part c, Sentence No. 2

The possibility of large flows in the abutments eroding the silt beneath the zone 2 blanket hinged on the magnitude of the flow. The entire reservoir loss was estimated at 75 ft^3/s and most of this was assumed to be lost to the regional water table from the reservoir area, thus having no effect on the embankment.

Pump-in tests on the right abutment demonstrated the capacity of the upper portion of the abutment to carry very large flows and thus it was felt that any flow which might go beneath the grout curtain would be quickly dissipated.

The Bureau anticipated that any abutment seepage would exit at such distances down the canyon walls that it would in no way impair the functioning of the dam. Even if it is assumed that the seepage from a specific joint did erode the silt beneath the zone 2 blanket, it should have resulted in a localized sink hole or depression which could have been easily treated.

Question No. 12

During construction of the dam, why were undisturbed block (cube) samples of compacted zone 1 fill and of fine-grained zone 3 fill not obtained for laboratory testing to verify design parameters?

ANSWER

Undisturbed block samples of an embankment are usually not required to verify design parameters for zone 1 when laboratory data on soil from a uniform borrow area are available as was the case at Teton. Denison samples were taken in the zone 1 fill at Teton in conjunction with preparations for the dynamic analysis of the dam. The parameters determined from testing these samples did verify design parameters. In addition, construction control tests were also available to verify zone 1, zone 2, and zone 3 design parameters.

Question No. 13

What investigations and studies were made during design to determine the significance of the older alluvium beneath the intracanyon basalt as related to the integrity of the dam?

ANSWER

The older alluvium beneath the intracanyon basalt was investigated by water testing and grouting drill holes in the alluvium during the 1969 test grouting program.

The foundation investigation in the area of the intracanyon basalt indicated that the underlying alluvium consisted of silt, sand, and gravel. Core recovery was minimal in the drill holes which indicated a possibility of some minor settlement under loading of the embankment. The test grouting in the area indicated that the alluvium could be consolidated by grout, and since it was confined between the basalt and rhyolite, settlement was not considered a problem. Analytical studies were not made during the design phase to examine the magnitude of possible settlement. After failure of the dam, a stress analysis of the foundation, including the deep lake bed sediments, was made using the finite element method. The results indicated a uniform compressive stress pattern and a smoothly varying deformation pattern of small magnitude.

Question No. 14

Why were embankment and foundation instruments, such as piezometers, deflectometers, and devices for measuring internal movements considered unnecessary? In other USBR dams of similar material is there information from instrumentation programs that indicates cracking of the core material may be a matter of concern? There are no two foundations similar in all respects. It is highly doubtful that grout curtain performance, movement of water through the foundation, etc., would be predictable at Teton Dam from observations at other dams. Explain the justification for not installing foundation piezometers at Teton Dam.

ANSWER

Answer to Question in Sentence No. 1

Instrumentation is not always used for structures constructed of materials previously instrumented at other dams and which have satisfactory performance records or in structures in which no problems are anticipated.

Answer to Question in Sentence No. 2

No. At the time of the design of Teton Dam (and to the present) review of the instrumentation data and past safety inspections in other Bureau dams with similar material did not indicate that cracking of the core material would be a matter of concern.

Answer to Question in Sentence No. 5

The justification for not installing foundation piezometers is as follows:

> The upper portions of the abutments consisted of highly fractured rock and it was felt that insufficient pressure would be developed in this region to warrant the placement of foundation piezometers. In the lower portion of the foundations the major high-angle joint set would provide a directional flow that would make the detection of any specific pore pressure buildup improbable.

Question No. 15

What design criteria were used to establish the width of the abutment key trenches as related to reservoir head? What was the gradient considered across the key trench and what is the Bureau normal standard?

ANSWER

Answer to Question in Sentence No. 1

The bottom width of 30 feet was selected to allow room for construction equipment and the three grout curtains. The width of the key trench selected was not related to the hydraulic gradient.

Answer to Question in Sentence No. 2

The gradient across the key trench will vary with the assumed head and the thickness of zone 1 material. The Bureau considered that at a normal water surface elevation of 4320.0, the maximum hydraulic gradient across the 30-foot-wide trench bottom at elevation 5100 would be 220/30 = 7.3. This gradient would have occurred in massive rock.

There is no written Bureau criteria for a limiting gradient across a zone 1 foundation contact. More generally, a gradient of 1:1 between the toe points of the zone 1 material is considered desirable.

Question No. 16

What design considerations were given to possible differential settlements and subsequent cracking of the low plasticity zone 1 fill due to geometric configuration of the supporting steep rock abutments and deep key trenches? What projects are similar to Teton in the use of ML materials and geometric configuration of abutments and key trenches?

ANSWER

Answer to Question in Sentence No. 1

The main design considerations given to possible differential settlements and subsequent cracking were based on an examination of past experience and the fact that no previous problems of this type had been known to occur. It was also assumed that the close construction control of zone 1 and zone 2 and the Bureau's conservative method of zone 1 placement would eliminate any potential settlement or cracking problems. At the time of the design of Teton Dam, the state-of-the-art had not developed so that a practicable mathematical solution could be performed to determine tension zones in the embankment due to differential settlement.

Answer to Question in Sentence No. 2

The Bureau has constructed the following dams which used an ML material:

Name	Height above streambed
Medicine Creek	102
Enders	100
Tiber	196
Palisades	260
Glendo	170
Red Willow	126
Bully Creek	104
North Coulee	90
Soldier Creek	251
Teton	305

The Bureau had not previously constructed a dam with deep key trenches such as were used at Teton Dam; however, Palisades Dam had abutment slopes which were approximately 60 percent for the left abutment and 50 percent for the right abutment and these slopes approximate closely those at Teton Dam. No significant problems have been observed in the 20-year history of Palisades Dam.

Question No. 17

Considering the degree of fracturing and permeability of the foundation rock, explain why the upstream and downstream grout curtains were not split-spaced to "closure." Regarding split spacing of upstream and downstream grout curtains, information is needed on how the probable volume of rock voids was estimated and how the probable direction of grout travel was estimated in order to determine the final 20-foot spacing in the upstream and downstream grout rows. Since the purpose of the other rows was to limit grout travel and provide an upstream and downstream barrier to allow the centerline holes to be grouted effectively, it appears that the term "3-row grout curtain" is somewhat misleading.

ANSWER

Answer to Question in Sentence No. 1

The rationale for not closing out the upstream and downstream rows of grout holes is given on pp. 9-10 of the Design Considerations For Teton Dam and is quoted below:

> "* * * Foundation investigations at the Teton damsite indicate that large grout quantities will be required to produce a tight curtain, and that special procedures will be required to prevent travel of the grout beyond the limits of the impervious barrier. In this connection, it is believed that even in the most pervious parts of the formation a barrier a few hundred feet wide will be ample for our design; consequently, when it appears that grout is traveling beyond these limits, steps should be taken to restrict its flow. To facilitate control of the grout, our design has adopted some of the methods and philosophies associated with overburden grouting. Briefly, these include drilling and grouting three staggered rows of grout holes through the critical areas. The outer rows of holes are drilled at a specified spacing and injected with a limited volume of grout based on the probable volume of voids in the zone being grouted. When this quantity has been pumped, grouting in the stage should be discontinued. Intermediate closeout holes at a spacing less than that shown on the drawings are not required in the outer rows.
>
> "After grouting at the outer rows is complete, the center grout cap row is drilled and closed out in the conventional manner."

Answer to Question in Sentence No. 2

There was no theoretical method used to estimate either the probable volume of rock voids or the probable direction of grout travel. The spacing of 20 feet was selected by the designers. The 3-row grid pattern shown in Detail A of specifications drawing No. 549-D-9 illustrates that a minimum spacing at 5 feet occurs between the holes of the three rows when they are projected onto a single plane passing through the centerline curtain. This concept as well as the fact that the 20-foot spacing of the downstream holes is offset 10 feet from the upstream holes and 5 feet from the centerline holes influenced the selected hole spacing and locations.

Answer to Comment in Sentence No. 3

The comments in response to the question in sentence No. 1 make it clear that the designers considered that a single barrier was being created. In creating the barrier, three individual sets of drill holes were being used. Thus, it was felt that the term "3-row grout curtain" would more clearly relate to others the reality of what was being done.

Question No. 18

Examination of the grouting records indicates that on the centerline curtain often only one closure hole was drilled between two scheduled taking holes rather than bracketing each scheduled taking hole with two closure holes. In a number of areas, no splitting appears to have been done when only one scheduled hole was a taker. Explain.

Specific questionable locations with respect to the splitting are:

a. Secondary hole at station 9+17 took 20 cf in a stage from 170-200 feet. The adjacent tertiary holes were not drilled below 150 and 160 feet.

b. There seems to be some confusion in the changing of the "closure" curtain from the centerline to the upstream row under the spillway. An upstream tertiary hole at station 11+37 took 1003 cf in a stage from 220 to 245 feet. Quanternary and fifth order holes were drilled in the upstream row at stations 11+26 and 11+31. When transitioning to the centerline row, no hole was drilled below 180 feet until station 11+66.

c. There seems to be some confusion between sheet 186 and sheet 175. The last primary hole on sheet 186 appears to be at station 13+10. The first primary on sheet 175 appears to be at station 13+46 but is too shallow (160 feet).

d. The secondary hole at station 17+03 is short with respect to the adjacent taking primary hole at station 13+46 (49 cf in a stage from 200 to 260 feet).

e. Quanternary holes 24+83 and 25+01 took 28 and 23 cf from 0 to 20 feet. There should be fifth order holes at 24+79, 24+87, 24+96, and 25+07.

f. Quanternary hole 33+86 took 360 cf in a stage from 230 to 250 feet. Fifth order bracketing holes were not drilled.

g. High takes were experienced in the last two fan holes at the end of the left abutment centerline row. Extraordinary, continued grouting at this location was probably not necessary.

ANSWER

Answer to Question in Sentence No. 1

An answer to this question is not required as a result of the following statement received from the Review Group. (Letter dated October 6, 1976.)

"Regarding centerline row closure holes, reexamination of the grouting drawings indicates that bracketing of the final scheduled taking hole was performed."

Answer to Question in Sentence No. 2

Normally all grout takes larger than 20 cubic feet of cement per stage on 10-foot spacing closure holes were drilled to 5-foot centers on both sides of the stages that took grout. In several areas, 5-foot center holes were drilled adjacent to 10-foot center holes even though the 10-foot center holes were tight. We are aware of no cases where splitting was not done when only one scheduled holes was a taker other than where additional grouting was used as in part "b" below:

Answer to Part "a"

Take for the primary hole at station 9+17 on the centerline curtain was checked on the right by the secondary hole 8+70 drilled to a depth of 220 feet. As the take was a marginal 20 cubic feet in a 30-foot stage and at depth (170'-200'), the stage was not checked by a secondary hole on the left.

Answer to Part "b"

Grout hole 11+37, stage 220 to 245, accepted 1,003 cubic feet and was split on one side as noted. However, the take was near the gate chamber adit and the area was well checked with grout holes from within the adit and access shaft.

Answer to part "c"

The first primary hole on sheet 175 is at station 13+78, 260 feet deep, some 68 feet down station from station 13+10. The hole at station 13+46, which was indicated as primary, is a secondary hole.

Answer to part "d"

The secondary hole at station 17+03 was short with respect to the adjacent primary hole at station 13+46 due to a drilling problem at depth at Station 17+03.

Answer to part "e"

At station 24+83, a large amount of the 28 cubic feet leaked at the surface. (See plan view on drawing No. 549-147-162.) Therefore, closures to 5-foot centers were not made. The details of why closure holes adjacent to station 25+01 were omitted for a take of 23 cubic feet for stage 0 to 20 feet are not known. It was probably due to loss from surface leaks or some other judgment factor.

Answer to part "f"

Station 33+86 was a tight hole; however, station 33+77 accepted grout at depth and was not closed to 5-foot centers. Most of the grout injected in the lake sediment area on the left abutment indicated back pressure and splitting to 5-foot centers at such depths was not deemed necessary.

Answer to part "g"

At the extreme ends of grout curtains on abutments, where the curtains could easily be extended in the future if necessary, strict closure was not deemed necessary.

Question No. 19

Why was water pressure testing routinely performed above and immediately after grouting a lower part of a grout hole?

Regarding water pressure testing on a freshly grouted interval of a rock, examination of field inspector's grouting reports indicates that a stage in a hole would be grouted and then successively higher stages in the hole would be water tested before the grout had a chance to set. What is the desirability of this procedure?

ANSWER

Answer to Question in Sentence No. 1

Water testing each stage prior to grouting was done to assure the packer was seated. Also, a water test gives an inspector a feel for the quantity of take and the starting water-cement ratio is often determined by the water test. Pressures for stages decrease as the packer is raised and a water test should not interfere with the grouted stages below. Stages with negligible water takes are not grouted.

Answer to Comments in Sentence No. 2 and Question in Sentence No. 3

In grouting ascending stages of a hole in firm rock, it is normal Bureau of Reclamation practice to water test each stage prior to grouting for the reasons given previously. As pressures decrease for higher stages, the grouted stages below should not be affected unless the grouted stages leaked to the surface in which case the grout should be allowed to attain its initial set prior to water testing the stage above. If the grouted stage be disturbed below the stage being water tested, grouting of the stage above will repair the disturbed area.

Question No. 20

Why was $CaCl_2$ used to restrict grout travel in the curtain area rather than using low pressure, heavy grout, and close hole spacing? Were attempts made to achieve design injection pressures regardless of rate of take?

ANSWER

Answer to Question in Sentence No. 1

$CaCl_2$ was only used where it was clearly evident that very large grout takes would be required and very little $CaCl_2$ was used in the centerline grout holes. $CaCl_2$ was used in lieu of close hole, low pressure heavy grout for three reasons.

1. Using $CaCl_2$ and standard grouting pressures would spread the grout sufficiently to achieve the sealing effect desired by the designers in the outer rows of holes.

2. $CaCl_2$ would require smaller grout quantities.

3. The center row of grout holes would be closed out with closely spaced holes using standard grouting pressures.

Answer to Question in Sentence No. 2

No.

Question No. 21

What grout quality laboratory tests were performed to verify the use of $CaCl_2$ percentages as high as 8 percent?

Laboratory tests of grout mixes were limited and did not emulate field conditions. What knowledge was available to verify the use of $CaCl_2$ percentages as high as 8 percent (the grouting contractor says 10 percent was used in the beginning)? Grout with 90° temperature was injected into a rock mass with temperatures as low as 50° at depth and perhaps much lower near the surface during the winter. What knowledge of grout behavior was available to justify the high percentages of $CaCl_2$ under these conditions?

ANSWER

Answer to Question in Sentence No. 1

No laboratory tests of the grout using $CaCl_2$ percentages as high as 8 percent were performed. $CaCl_2$ was used based on its generally accepted characteristic of accelerating the set of concrete.

Extensive tests of grout mixes containing varied quantities of water, cement, sand, bentonite, and $CaCl_2$ were performed in 1974 to determine yield, cost per cubic foot, and strength. However, maximum percentages of $CaCl_2$ by weight of cement were approximately 4.5 percent. Results of the tests, included in the report titled "Teton Basin Project Foundation Grouting Report - DC-6910," dated May 1974, are as follows:

> "YIELD AND COMPRESSIVE STRENGTHS OF GROUT MIXES
>
> "Laboratory tests to determine yield for various grout mixes used on the Teton Project were performed during the past months. Figure No. 1 shows the yield and strengths achieved from the various mixes tested.
>
> "The ingredients for the individual mixes were predetermined to fill a standard 6" X 12" concrete cylinder mold and were wet-cured for 6 days prior to breaking.
>
> "A cost per cubic foot of yield was calculated using the bid price for cost of cement and cost of injection and also includes the cost of calcium chloride in those mixes where calcium chloride was used. The cost of bentonite was not included as the addition of bentonite to a sanded mix is included in the cost of the sand.

"It can be noted from Figure No. 2 (attached) that the compressive strength and yield are related for mixes not containing calcium chloride. Mix No. 1 shows a high yield and low compressive strength while Mix No. 2 and 14 show a higher compressive strength and lower yield.

"Calcium chloride was added to Mixes No. 3, 4, and 5 using mix water at a temperature of 67° F. A higher compressive strength with lower yield was attained in Mix No. 3 by using a higher concentration of calcium chloride than was used in Mix No. 4 and 5.

"Calcium chloride was also added to mixes 10 through 13. A constant yield with increase in compressive strength was attained by increasing the mix water temperature. For Mixes No. 10, 11, and 12, water temperatures used were 44°, 54°, and 60° F respectively. Initial set time was 34, 23, and 16 minutes at temperatures of 70°, 75°, and 76° F respectively.

"Mix Nos. 6 through 9 are grout mixes containing various amounts of sand per bag of cement. Compressive strengths for those mixes containing no calcium chloride varied between 2,292 psi and 2,502 psi."

Answer to Question in Sentence No. 3

There are, to the best of our knowledge, no studies available concerning the specific effect of using $CaCl_2$ percentages as high as 8 percent in grouting operations.

Very little research has been done by others in this field. In regard to tests by others, Traetteberg and Sereda in their paper entitled, "Strength of C_3A Paste Containing Gypsum and $CaCl_2$ (Cement and Concrete Research, Vol 6, 1976)" noted that when using 8 and 16 percent $CaCl_2$ with C_3A, the early strength producer of portland cement, produced a microstructure expected to result in improved interparticle bonding. If so, one might hypothesize that this would improve resistance to erosion or dissolution by water.

Following are two references regarding erosion of concrete containing $CaCl_2$:

a. Proceedings of the American Concrete Institute, Vol 60, 1963, page 1489. Report of ACI Committee 212, Admixtures for Concrete,

> "It (calcium chloride) significantly increases the resistance of concrete to erosive and abrasive action especially at early ages." (1-2 percent additions of $CaCl_2$)

b. Shideler, J. J., "Calcium Chloride in Concrete" Proceedings of the American Concrete Institute, Vol 48, 1952, pp 557.

> "Data obtained in abrasion and cavitation tests conducted in the Bureau laboratories strongly indicate that concretes containing 2 percent $CaCl_2$ are more resistive to erosive forces than plain concrete under either moist curing or drying conditions."

Although these references are of limited applicability they do imply that no serious defects occur as the percentage of $CACl_2$ increases.

<u>Answer to the Comment in Sentence No. 4 and the Question in Sentence No. 5</u>

No specific test results were performed concerning the effect of using $CaCl_2$ percentages up to 8 percent in a grout that was to be injected into rock with temperatures as low as 50 degrees. However, the core recovered from the drill holes used to examine the results of the test grouting program indicated good bond between the grout and the foundation rock.

Mix No.	Ratio W/C/S	Weights of Materials Used (Pounds) Water	Cement	Sand	Bentonite	$CaCl_2$	Yield Ft^3/Bag	$Cost/Ft^3$	Strength 7 Days
1	1:1	6.656	10.027	0	0	0	1.332	$3.68	1,415 psi
2	0.8:1	6.144	11.569	0	0	0	1.293	3.79	2,440
3	0.8:1	6.144	11.569	0	0	0.42	1.178	4.33	3,336
4	0.8:1	6.144	11.569	0	0	0.35	1.244	4.10	3,135
5	0.8:1	6.144	11.569	0	0	0.36	1.274	4.00	3,043
6	1:1:1.08	4.99	7.52	6.72	0.13	0	1.950	4.72	2,292
7	1:1:1.08	4.99	7.52	6.72	0.13	0.23	1.938	4.87	2,603
8	1:1:1.3	4.37	6.59	7.06	0.14	0	2.133	4.73	2,502
9	1:1:1.5	3.74	5.67	7.06	0.14	0	2.232	4.88	2,457
10	0.8:1	5.99	11.28	0	0	0.40	1.293	4.01	2,760
11	0.8:1	5.99	11.28	0	0	0.34	1.293	3.95	2,972
12	0.8:1	5.99	11.28	0	0	0.34	1.293	3.96	3,114
13	0.8:1	5.99	11.28	0	0	0.51	1.293	4.02	2,583
14	0.8:1	5.99	11.28	0	0	0	1.259	3.89	2,813
15	1:1:1	5.62	8.46	7.02	0	0	1.393	4.70	
16	0.8:1:1	4.50	8.46	7.02	0	0	1.786	4.98	

Figure No. 2

Question No. 22

In a rock mass where the orientation of joint sets varies unpredictably, the grout holes in each grout line are often oriented in a different direction. Please explain why the upstream and ceterline holes were oriented in the same direction.

ANSWER

Initially, the grout curtain design required that the upstream and downstream holes were to be vertical and the centerline holes were to be angled at 30° into the abutments. After exposure of the right abutment key trench, it was decided that the upstream row of holes should also be angled into the abutment in order to intersect more joint sets. Both the upstream and centerline holes were oriented in the same direction. To the best of our knowledge there is no greater probability of ensuring complete grouting by orienting the holes of one row at say 45 degrees and the other row at 30 degrees. The important concept is simply that the holes are angled.

Question No. 23

Was the alluvium below the basalt sampled, and were appropriate tests performed to determine the relative merits of cement vs. chemical grouting for this material?

ANSWER

The results of the pilot grouting program of Teton damsite in 1969 conclusively demonstrated that the 5- to 20-foot thick zone of alluvium between the basalt and rhyolite was groutable with a cement grout. Thus, for economic reasons, a chemical grout was not tested or considered.

Question No. 24

Was the decision not to continue the key trench under the spillway made only on economics?

ANSWER

The decision not to continue the key trench under the spillway was not based on economics. The decision was based on the need to avoid differential settlement of the spillway crest structure in order to prevent cracking of the crest structure and possible malfunction of the radial gates. The formation under the spillway crest was considered to be much superior to compacted embankment for carrying the design loadings.

Postfailure Investigation of Right Abutment Embankment Remnant

Reproduced from pages 3-10 through 3-16 and Figure F-42 of Independent Panel Report "Failure of Teton Dam," December 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 accidently 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.

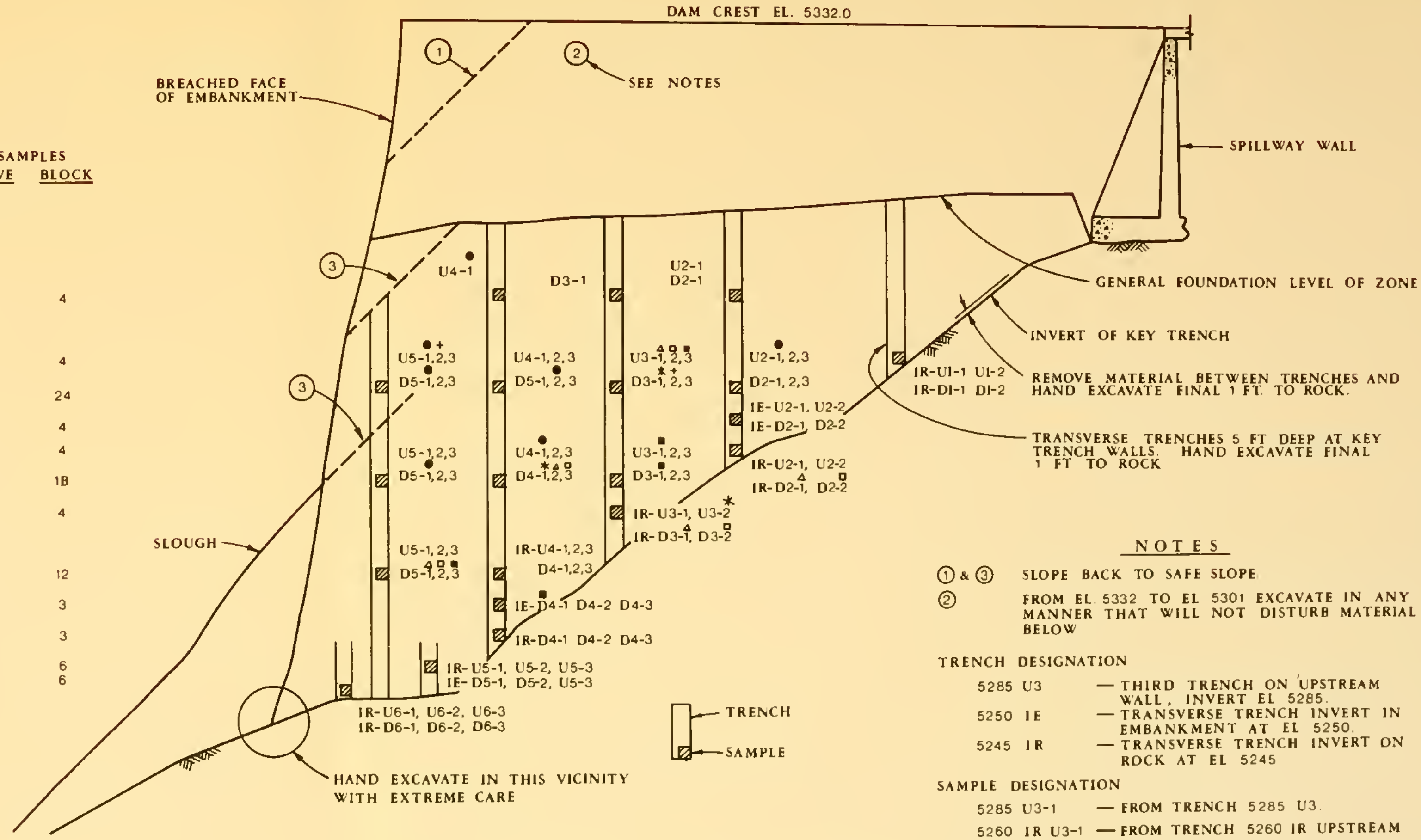
Selected block samples, representative of the range of materials and densities found, and spanning the mass of the embankment remnant on the right abutment, were sent to various laboratories for identification tests and tests of designated engineering properties. To the extent practicable, two laboratories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

ZONE-1 EL.	SAMPLES DRIVE	SAMPLES BLOCK
5301	4	
5296	4	
5290		
5285	5	4
5280	8	
5275		4
5270	8	24
5265		4
5260		4
5255	8	18
5250		4
5245		
5240	6	12
5235		3
5230		3
5227.5		
5225		6
5222.5	4	6
5220		
5210		
TOTAL SAMPLES	47	92



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

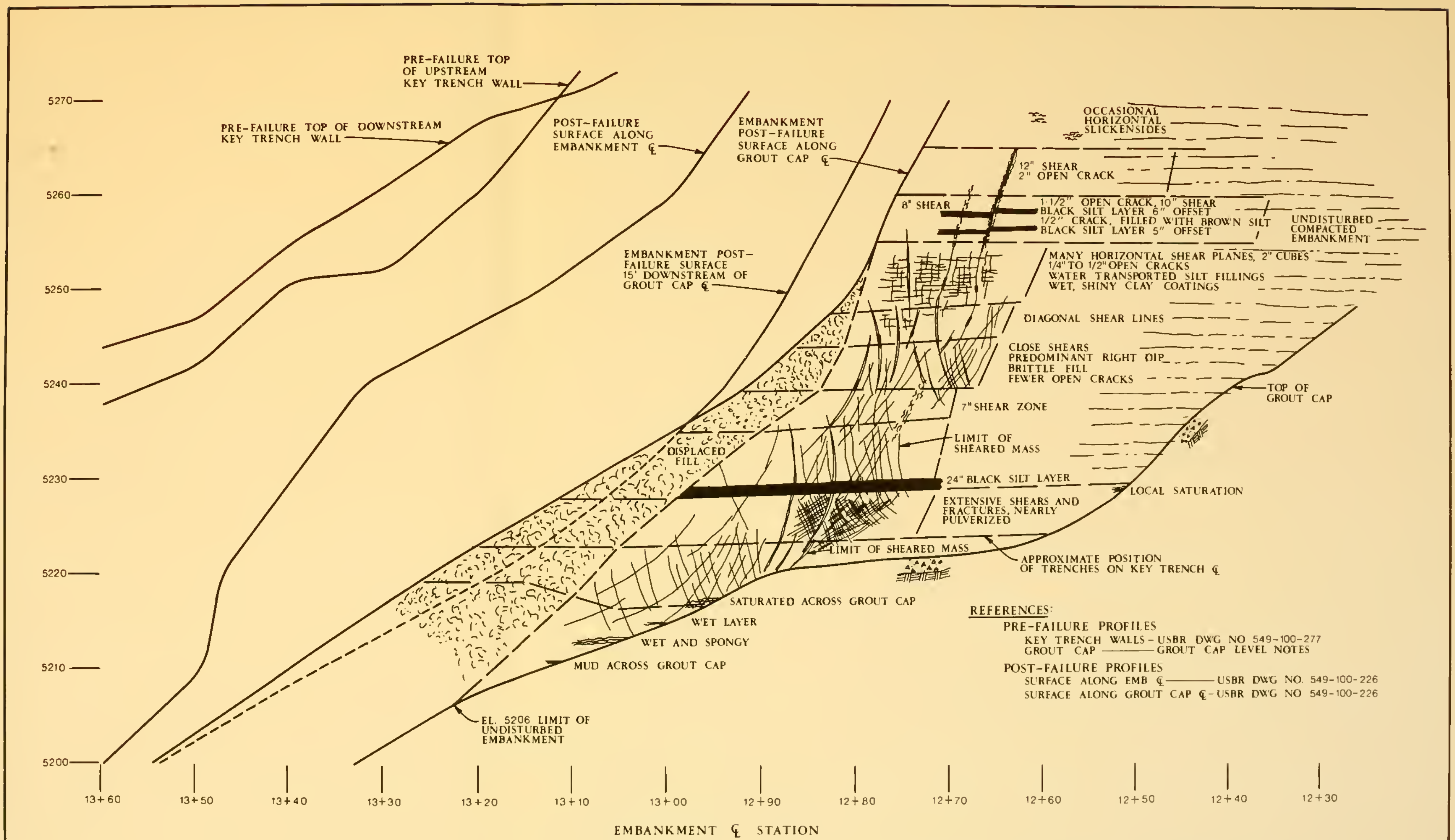
TESTING PROGRAM

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

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



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

In Situ Stress Investigations

Reproduced from Appendix D, "Hydraulic Fracturing and Its Possible Role in the Teton Dam Failure," and pages 3-17 through 3-21 of the Independent Panel Report on "Failure of Teton Dam," December 1976.

APPENDIX D

FINITE ELEMENT ANALYSES

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

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

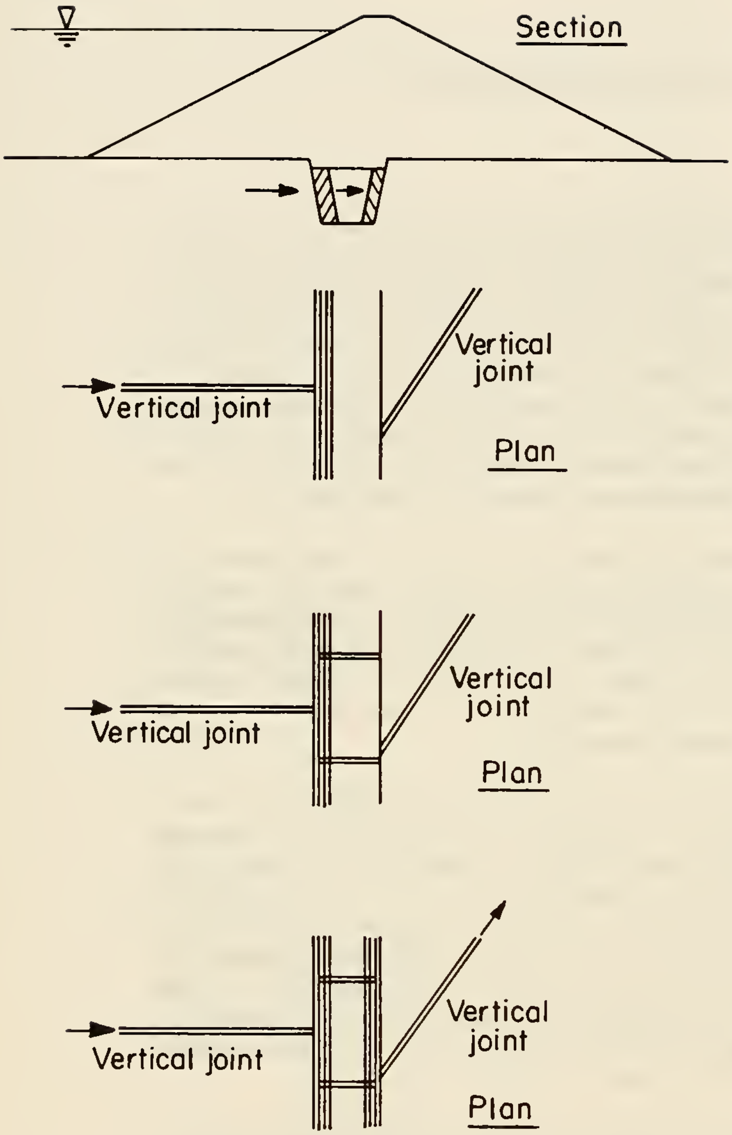
In recent years, cracking leading to excessive loss of drill water in the cores of a number of embankment dams has been attributed to the phenomenon of hydraulic fracturing; that is, a condition leading to the creation and propagation of a thin physical separation in a soil whenever the hydraulic pressure 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.



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

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)

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 SCHEMATIC 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 I, 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:

$$E_i = Kp_a \left(\frac{\sigma_3}{p_a}\right)^n$$

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 Remoulded ML, at Compaction Water Content	U-U Test Remoulded CL, at Compaction Water Content	Drained Test Remoulded ML-CL at Compaction Water Content	Drained Test Undisturbed at Field Water Content Boring 5255-D-4	Drained Test 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						

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. $G = 0.35$; $\gamma = 117$ lb/ft^3; $c = 1650$ psf, $\phi = 31^o$; 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:

$\gamma = 117$ lb/ft^3
$c = 1650$ psf
$\phi = 31^o$
$K = 470$
$n = 0.12$
$R_f = 0.79$
$F = 0.10$
$d = 4.0$

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

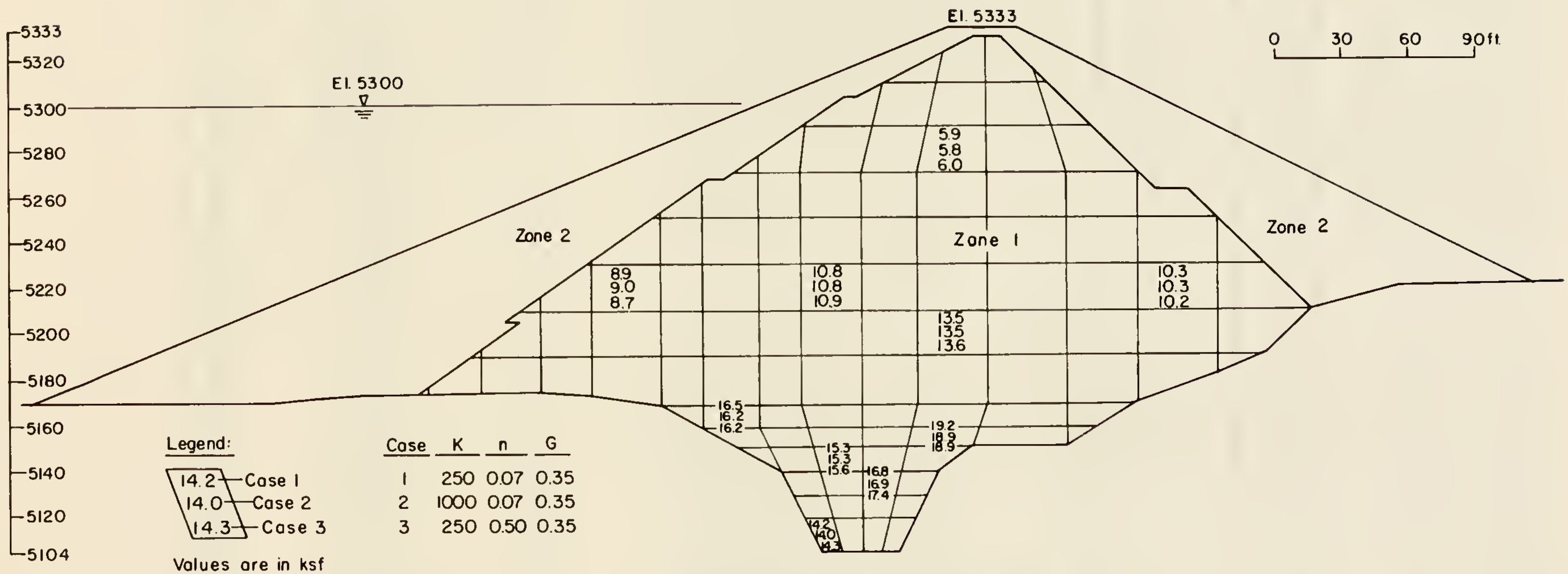


FIG. 2 EFFECT OF VARIATIONS IN PARAMETERS K AND n ON COMPUTED VALUES OF MAJOR PRINCIPAL STRESS AT STA. 15+00

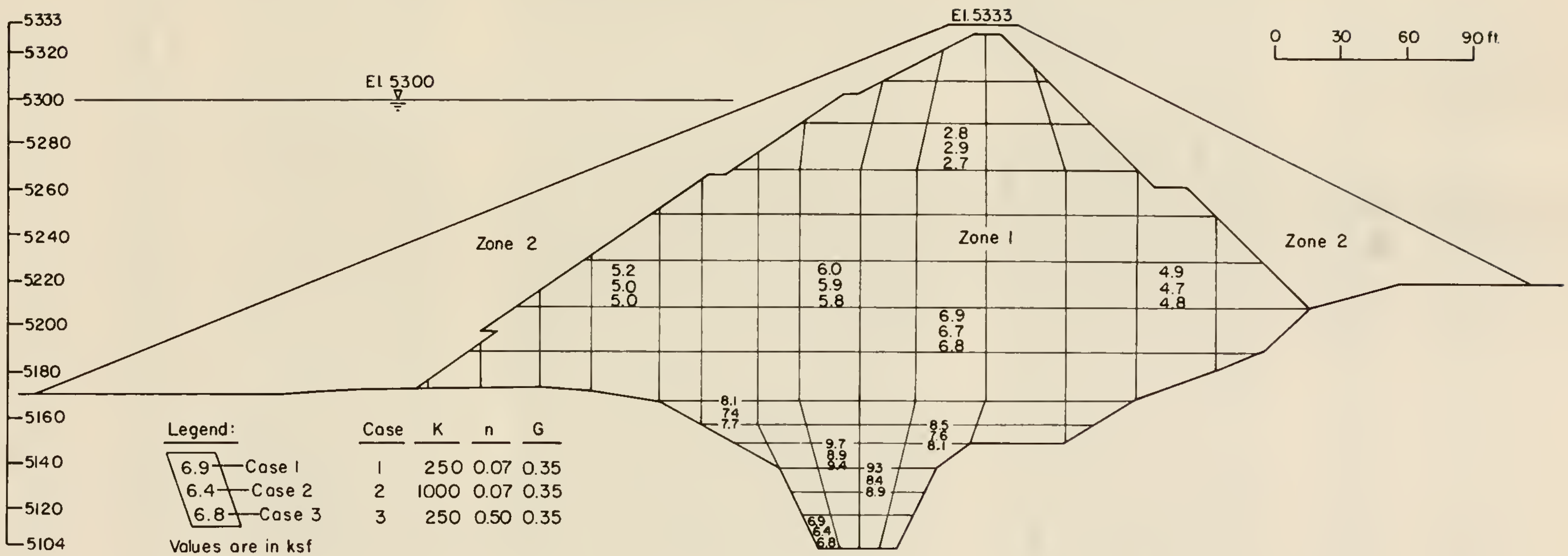


FIG.3 EFFECT OF VARIATIONS IN PARAMETERS K AND n ON COMPUTED VALUES OF MINOR PRINCIPAL STRESS IN TRANSVERSE PLANE AT STA. 15+00

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

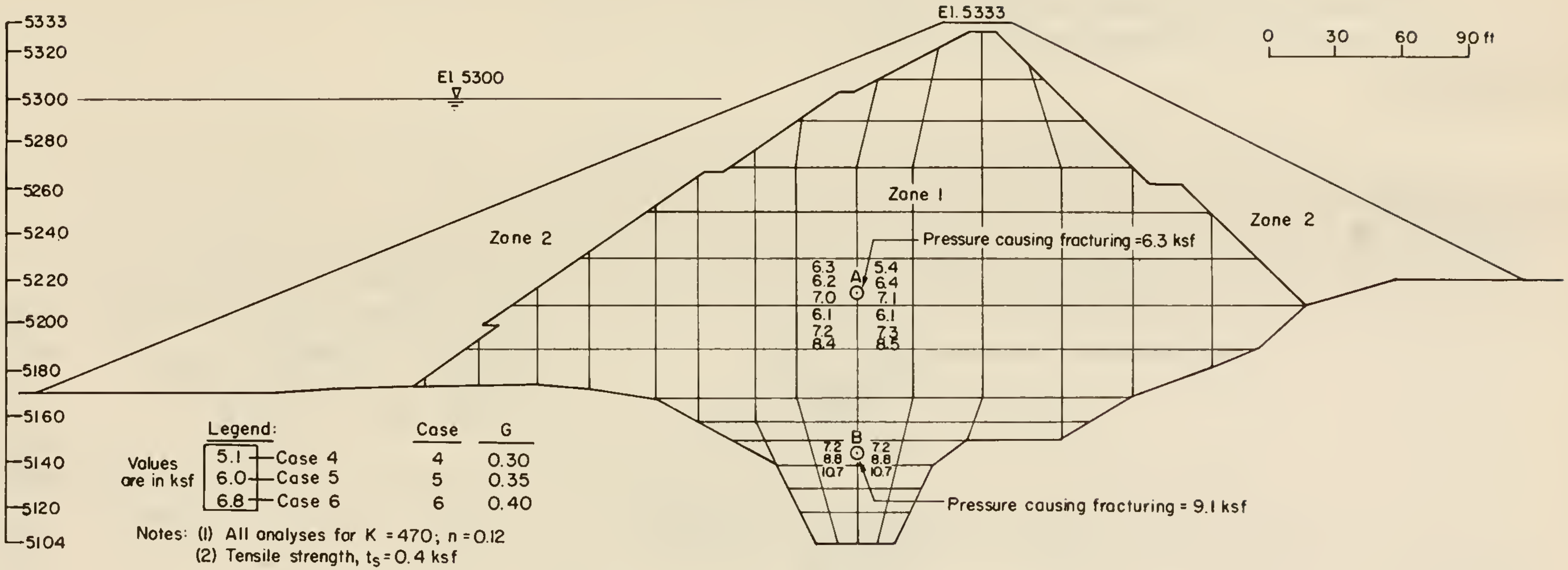


FIG. 4 EFFECT OF VARIATIONS IN PARAMETER G ON COMPUTED VALUES OF SUM OF MINOR PRINCIPAL STRESS AND TENSILE STRENGTH OF SOIL-STA. 15+00 KEY TRENCH ASSUMED TO BE UNWETTED

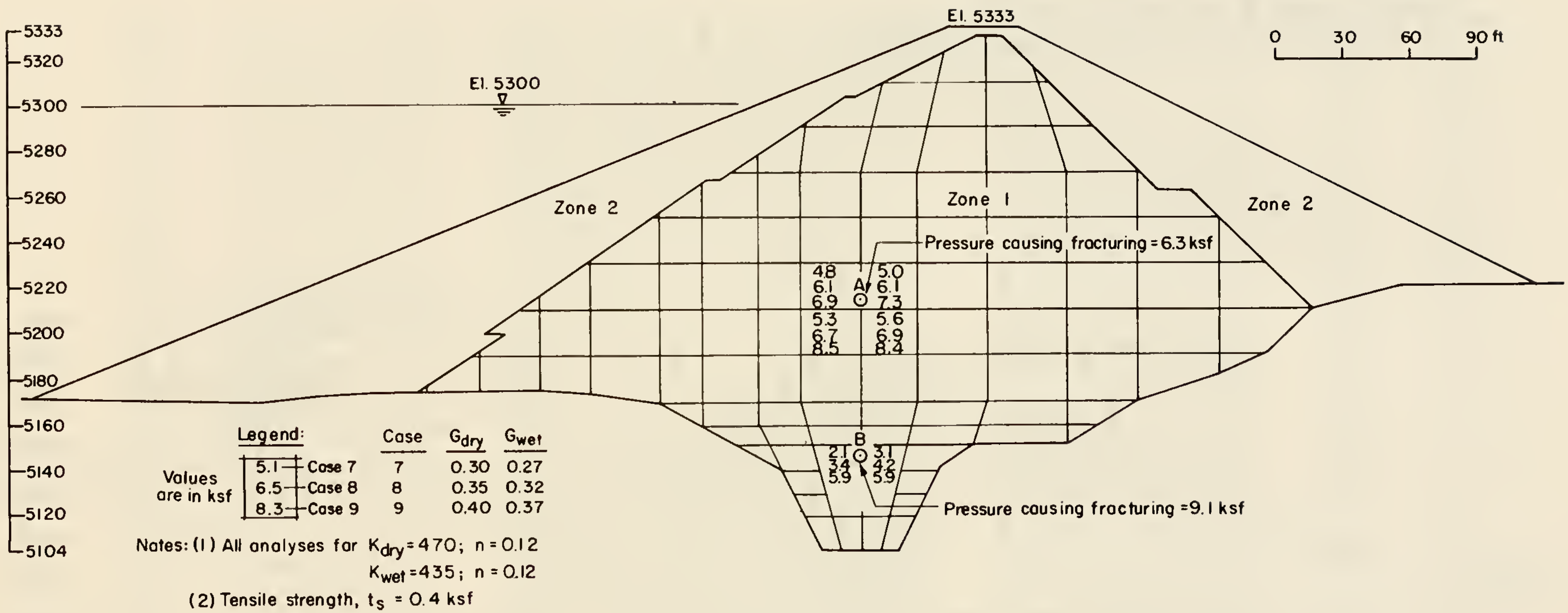
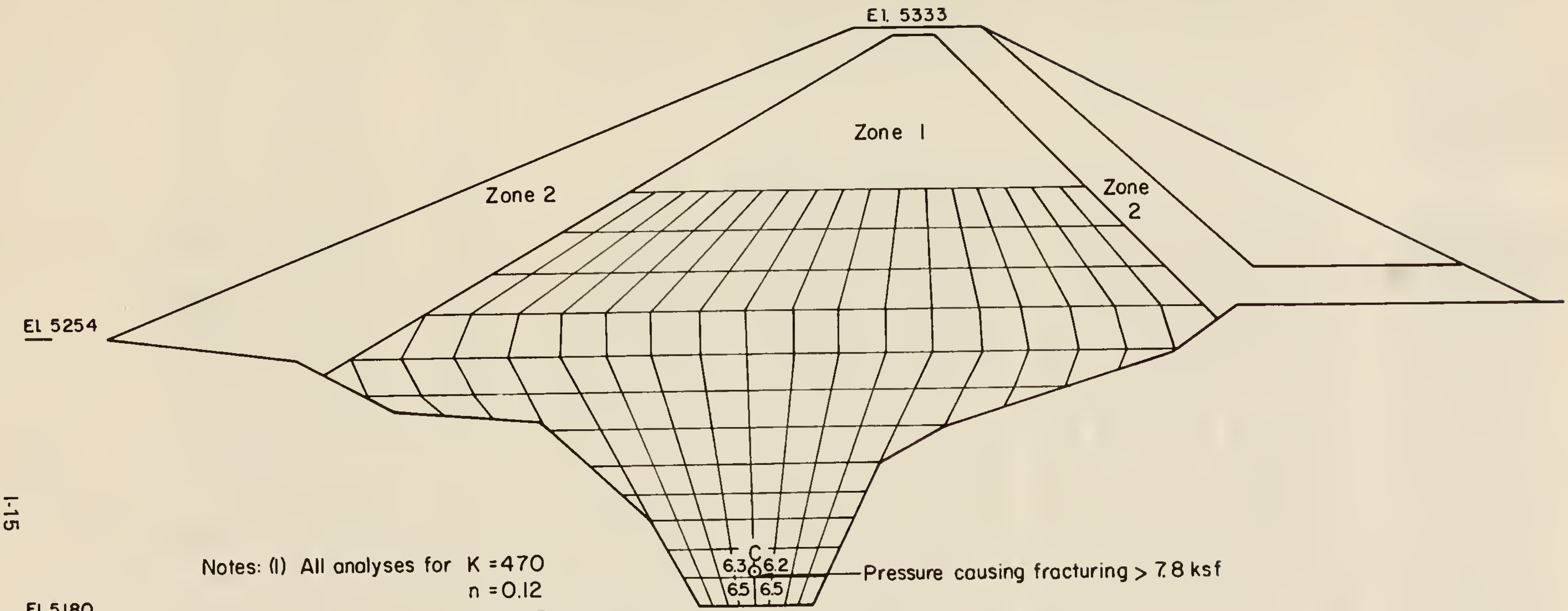


FIG. 5 EFFECT OF VARIATIONS IN PARAMETER G ON COMPUTED VALUES OF SUM OF MINOR PRINCIPAL STRESS AND TENSILE STRENGTH OF SOIL-STA. 15+00 KEY TRENCH ASSUMED TO BE WETTED



Notes: (1) All analyses for K = 470
n = 0.12
G = 0.35
(2) Tensile strength of soil, t_s = 0.4 ksf

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^{o} 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

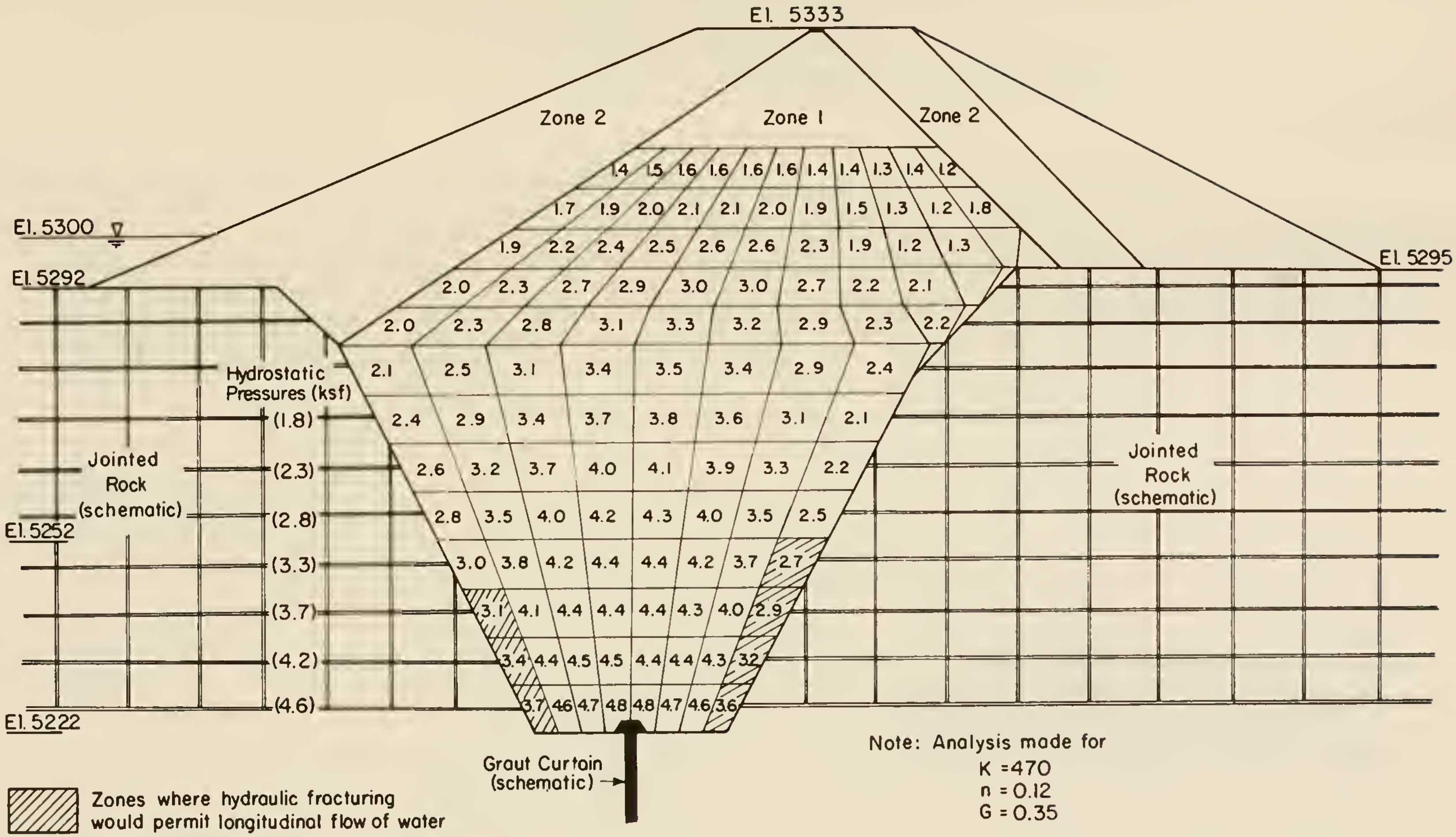


FIG. 7 COMPUTED VALUES OF MINOR PRINCIPAL STRESS IN PLANE OF SECTION ksf STA. 12+70

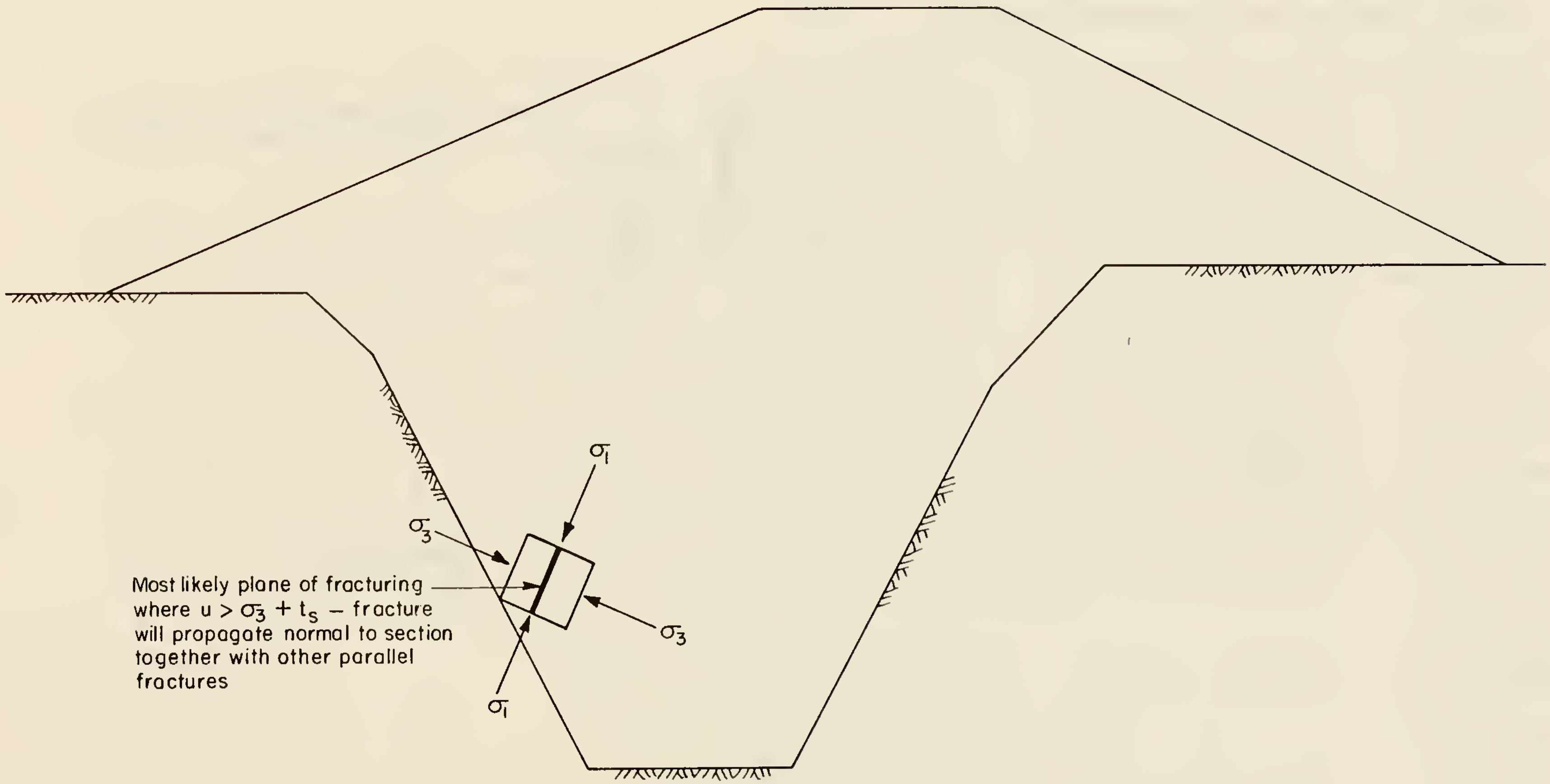


FIG. 8 DEVELOPMENT OF LONGITUDINAL FRACTURE PATTERN

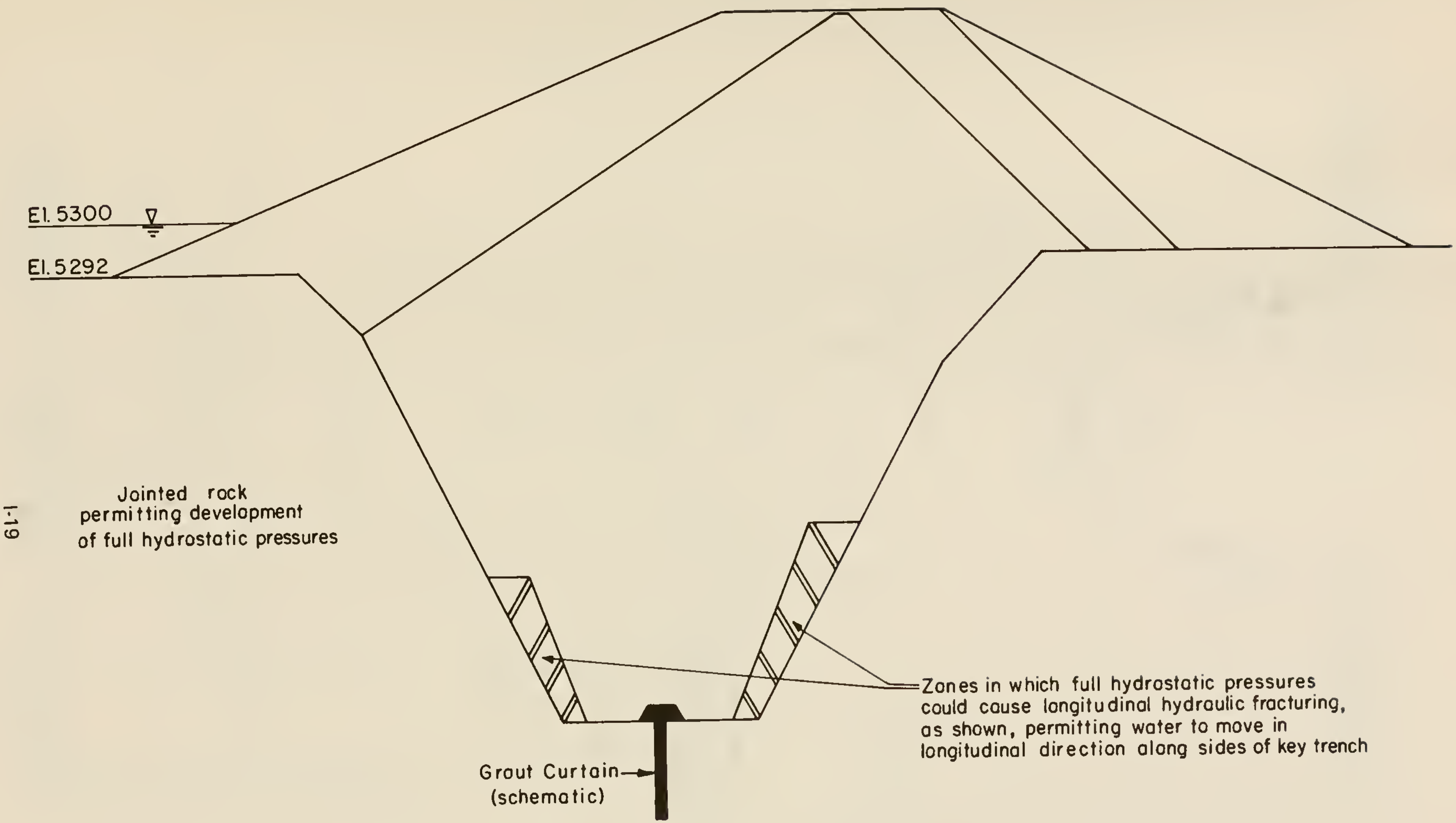
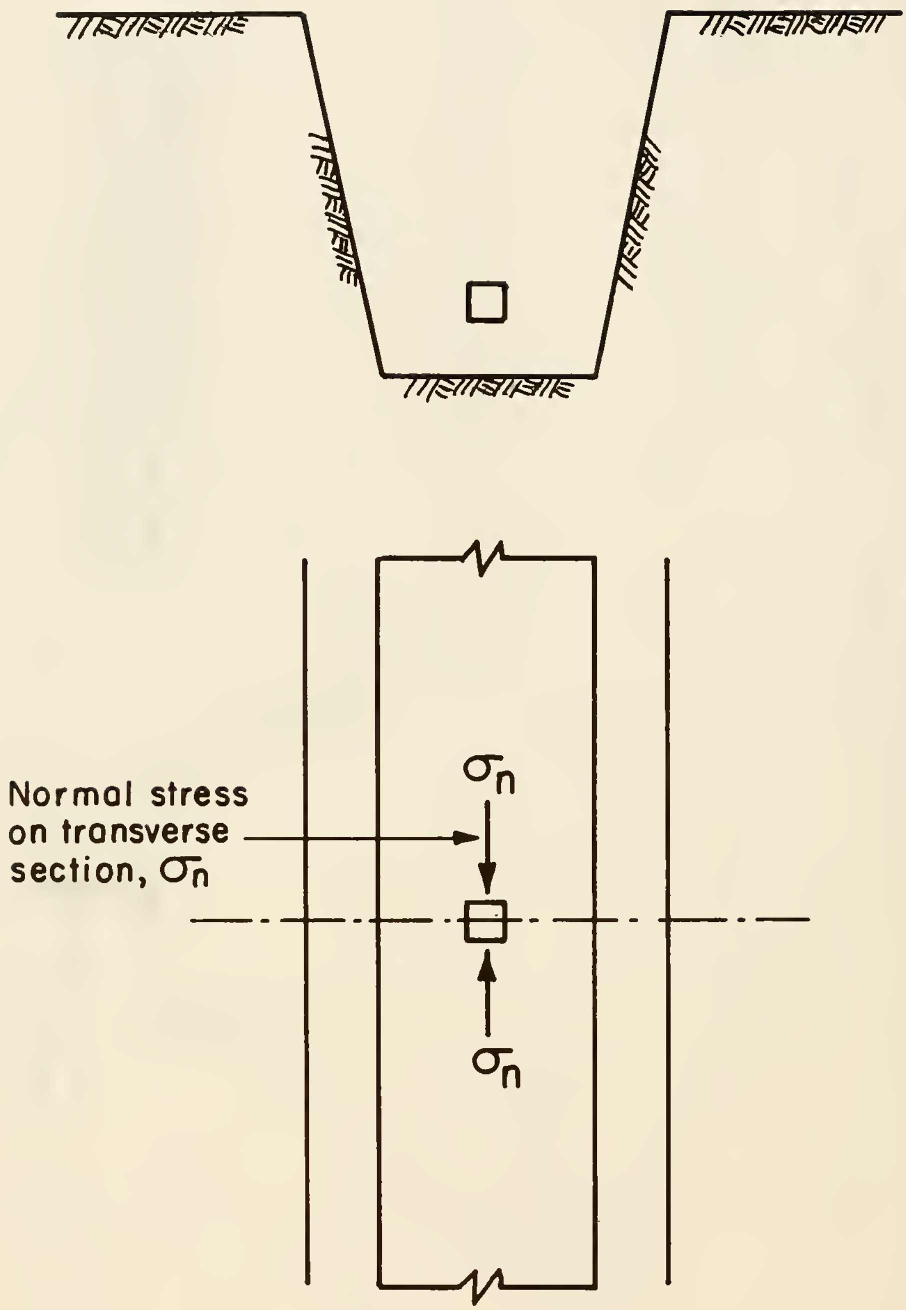


FIG. 9 ZONES OF POTENTIAL LONGITUDINAL FRACTURING AT STA. 12 + 70



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

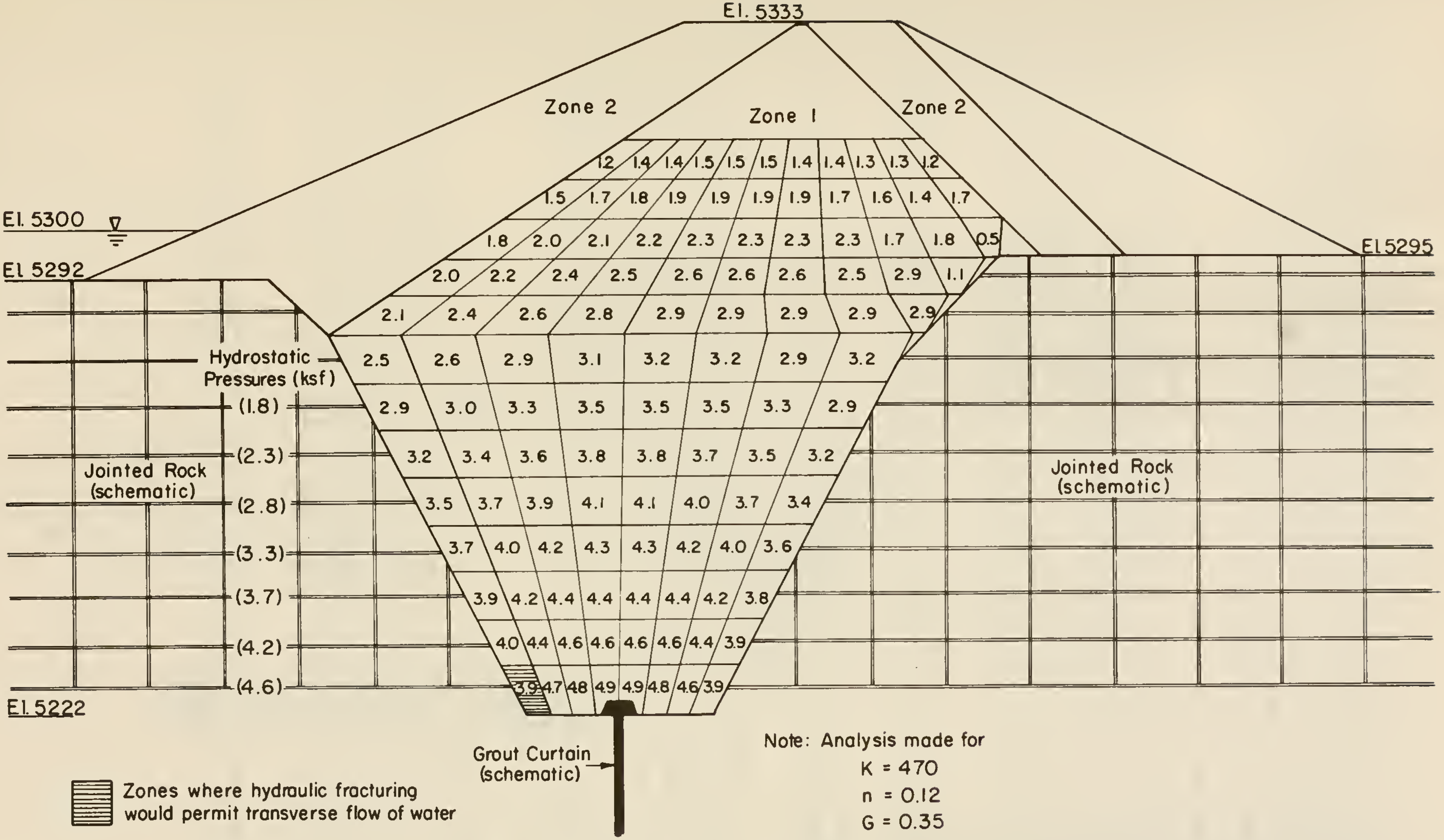


FIG. 11 COMPUTED VALUES OF NORMAL STRESS ON TRANSVERSE SECTION ksf STA. 12+70

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

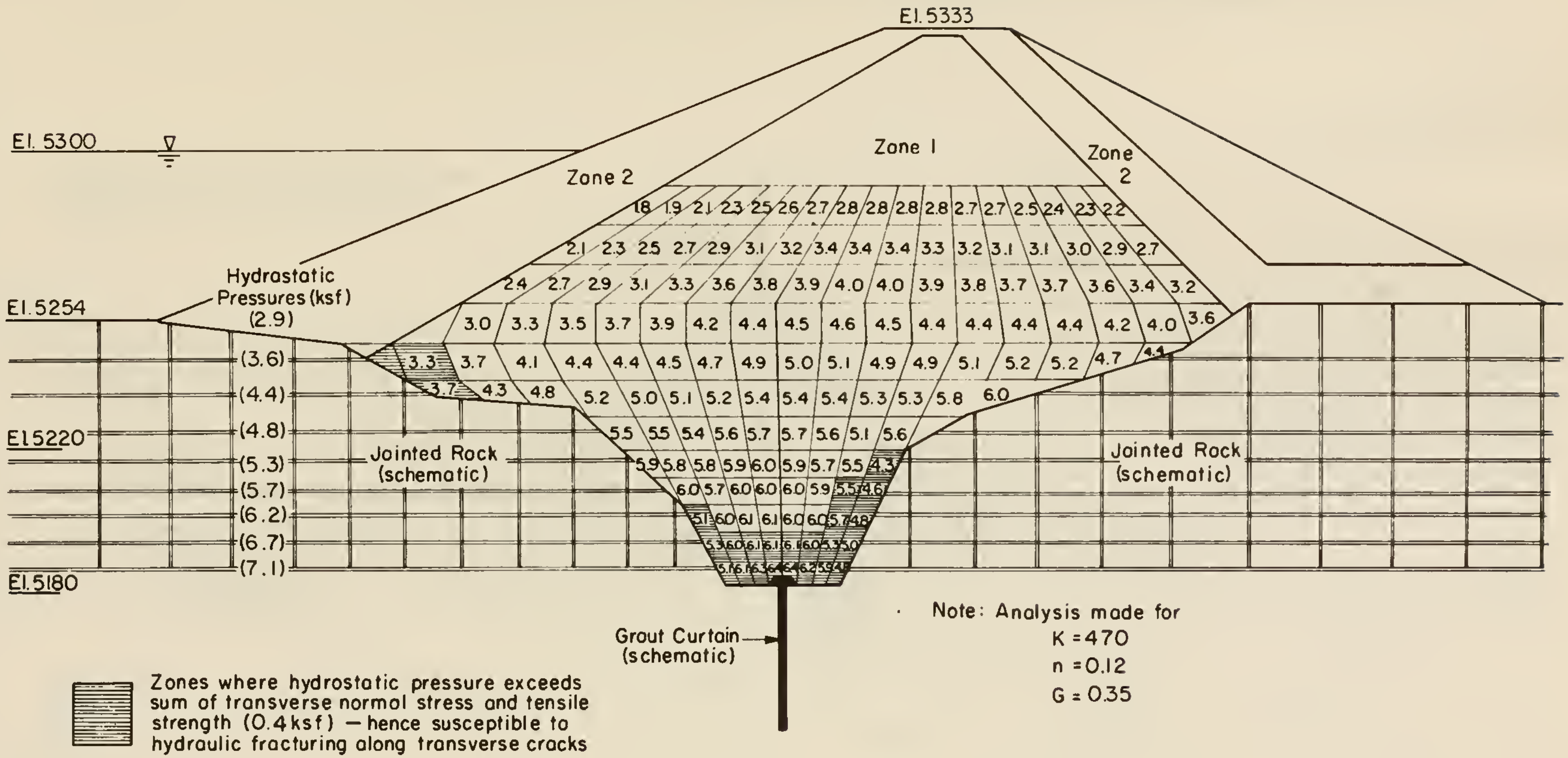


FIG. 12 COMPUTED VALUES OF MINOR PRINCIPAL STRESS IN PLANE OF SECTION STA. 13+70

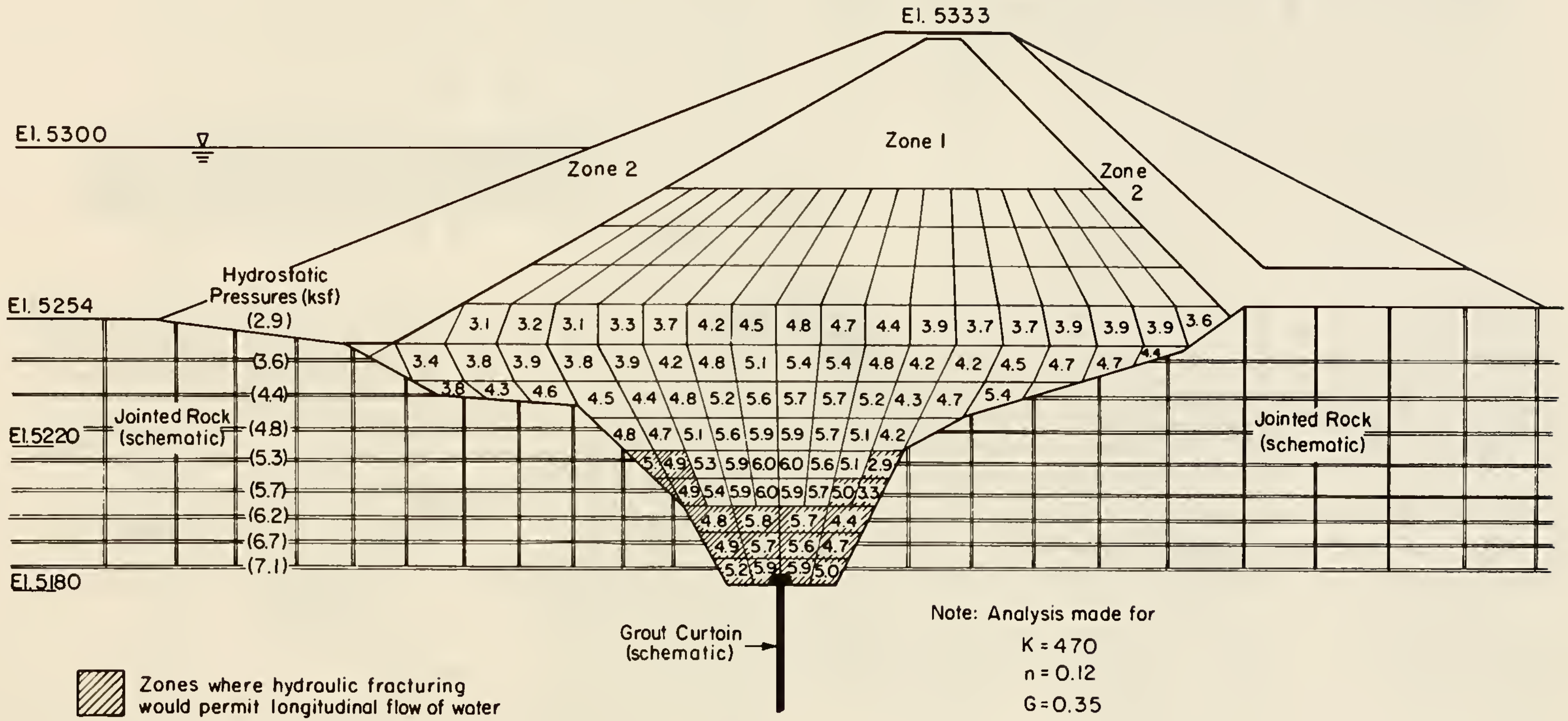


FIG. 13 COMPUTED VALUES OF NORMAL STRESS ON TRANSVERSE SECTION IN Ksf STA. 13+70

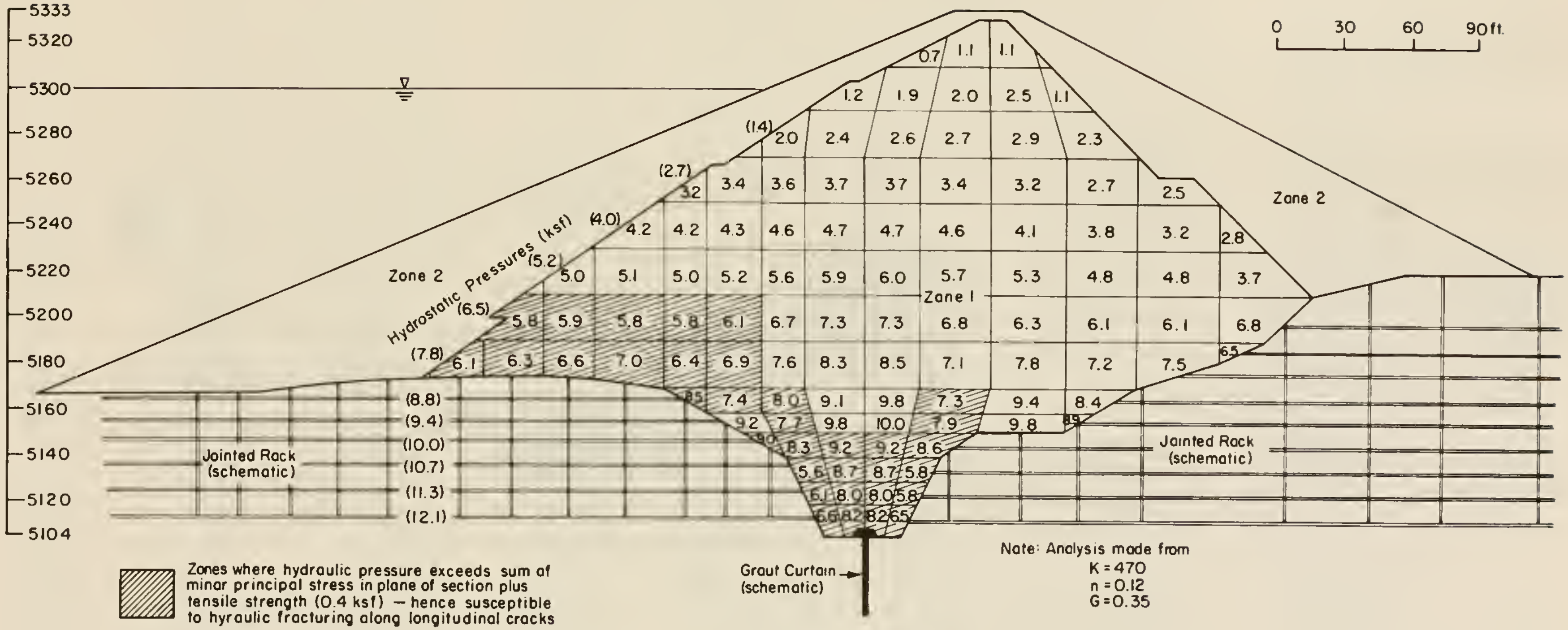


FIG. 14 COMPUTED VALUES OF MINOR PRINCIPAL STRESS IN PLANE OF SECTION ksf STA. 15+00

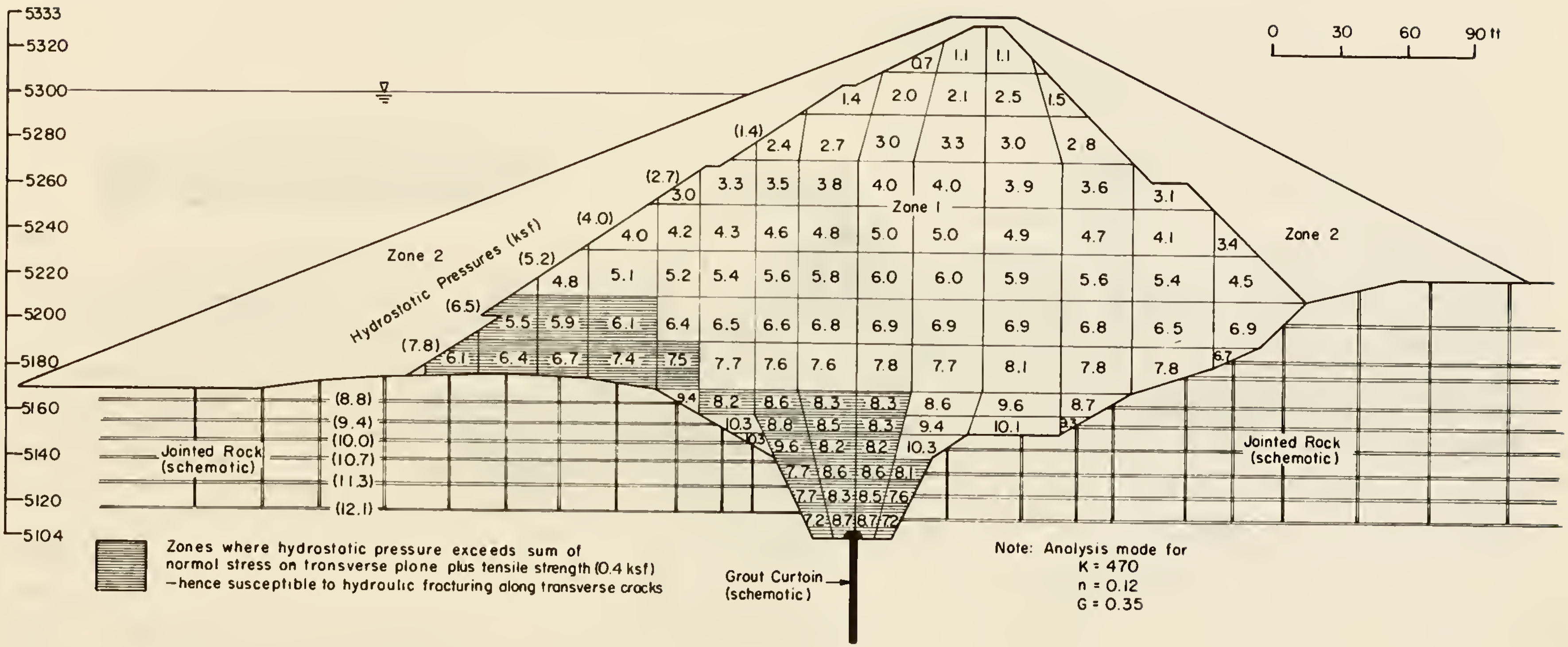


FIG. 15 COMPUTED VALUES OF NORMAL STRESS ON THE TRANSVERSE SECTION IN ksf STA. 15+00

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

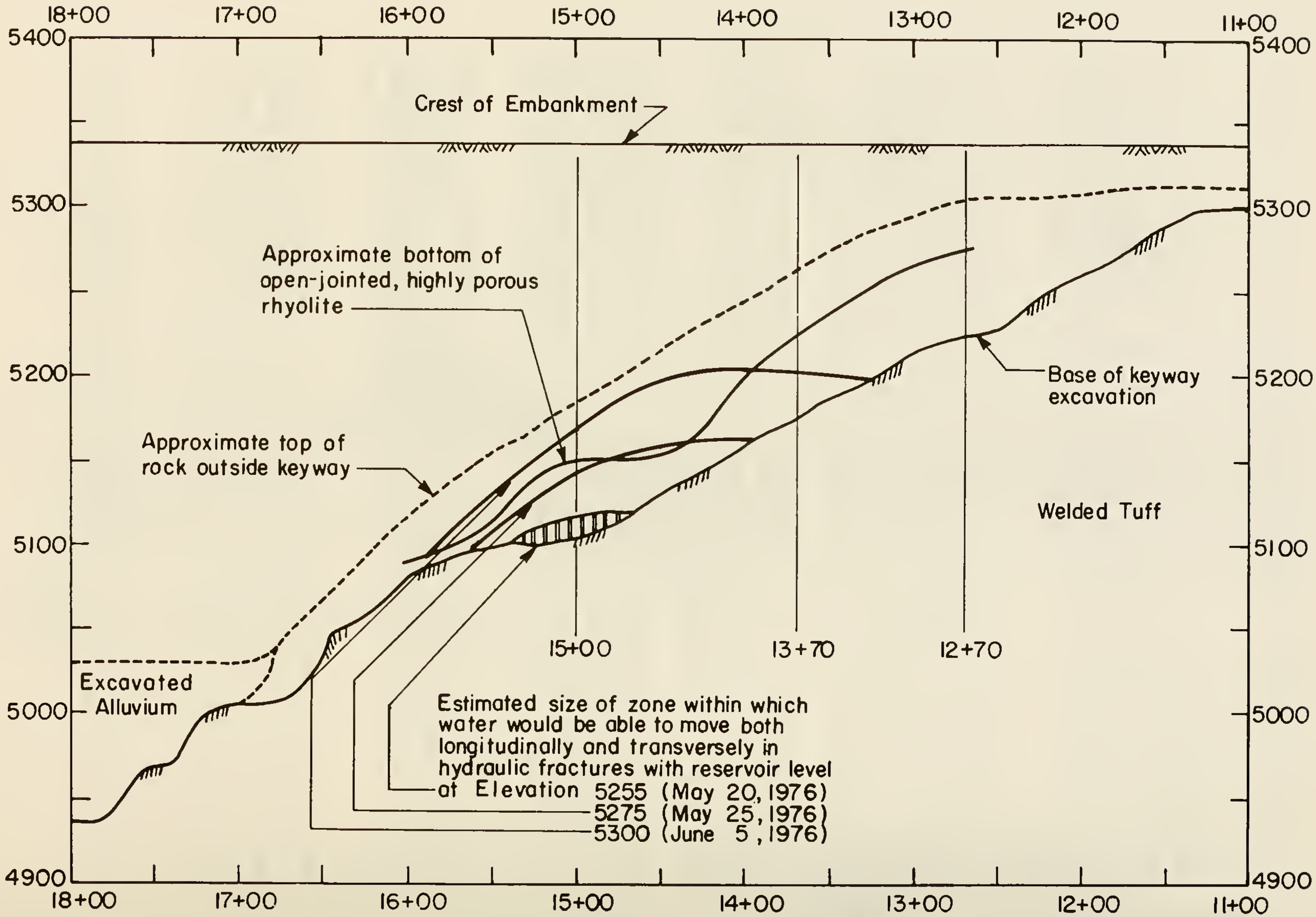


FIG. 16 LONGITUDINAL SECTION THROUGH CENTER LINE CREST AND GROUT CAP

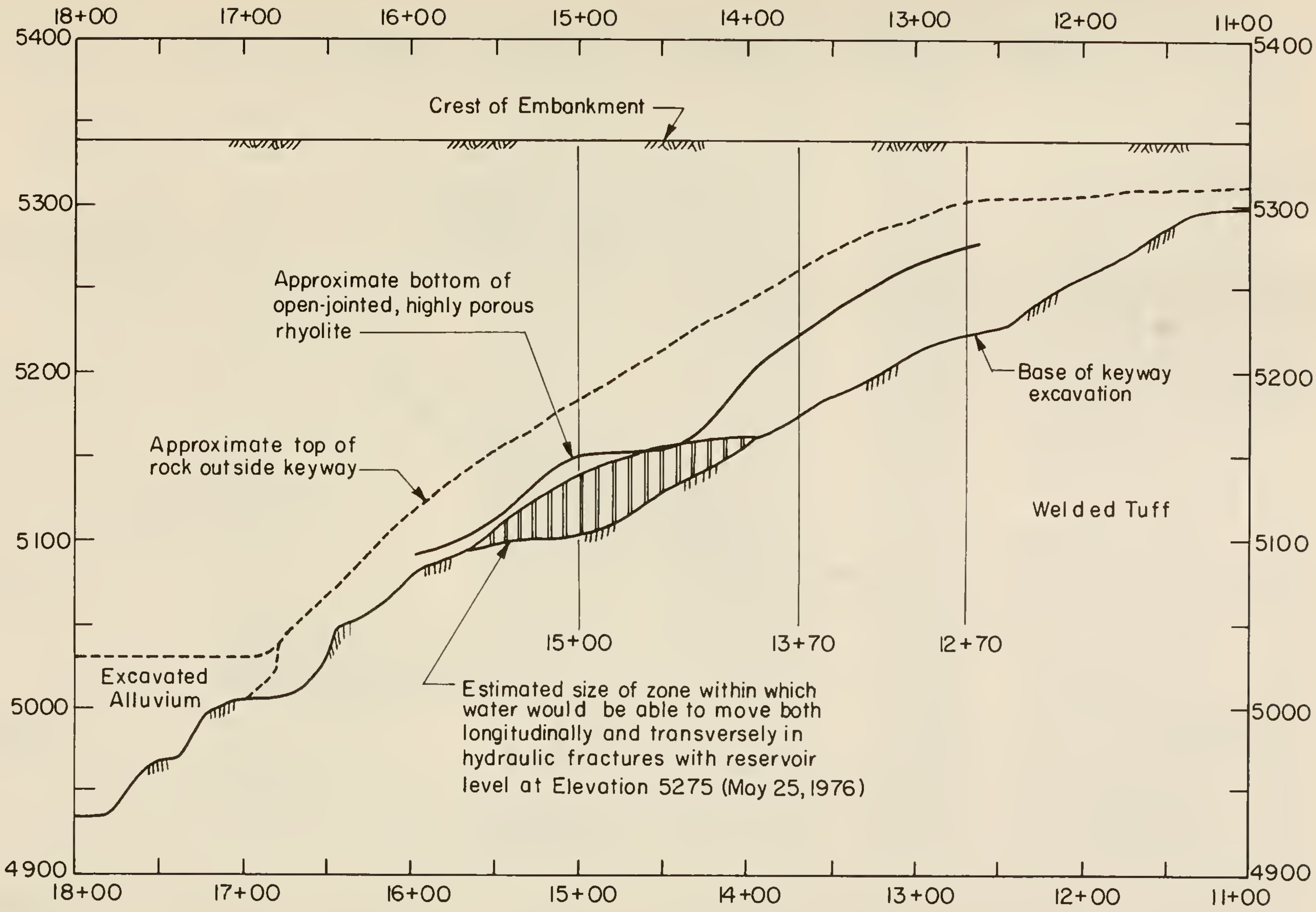


FIG. 17 LONGITUDINAL SECTION THROUGH CENTER LINE CREST AND GROUT CAP

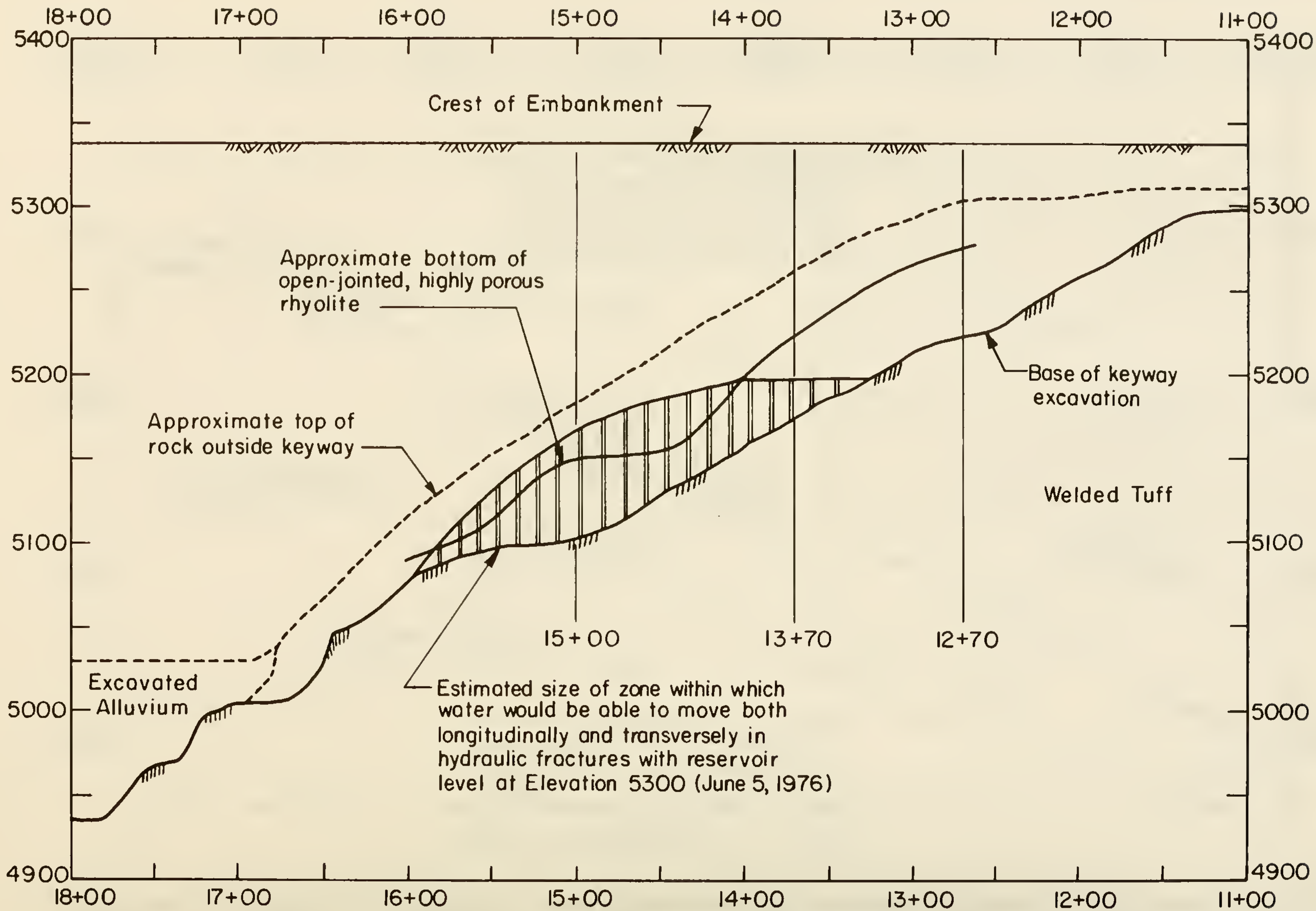


FIG.18 LONGITUDINAL SECTION THROUGH CENTER LINE CREST AND GROUT CAP

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

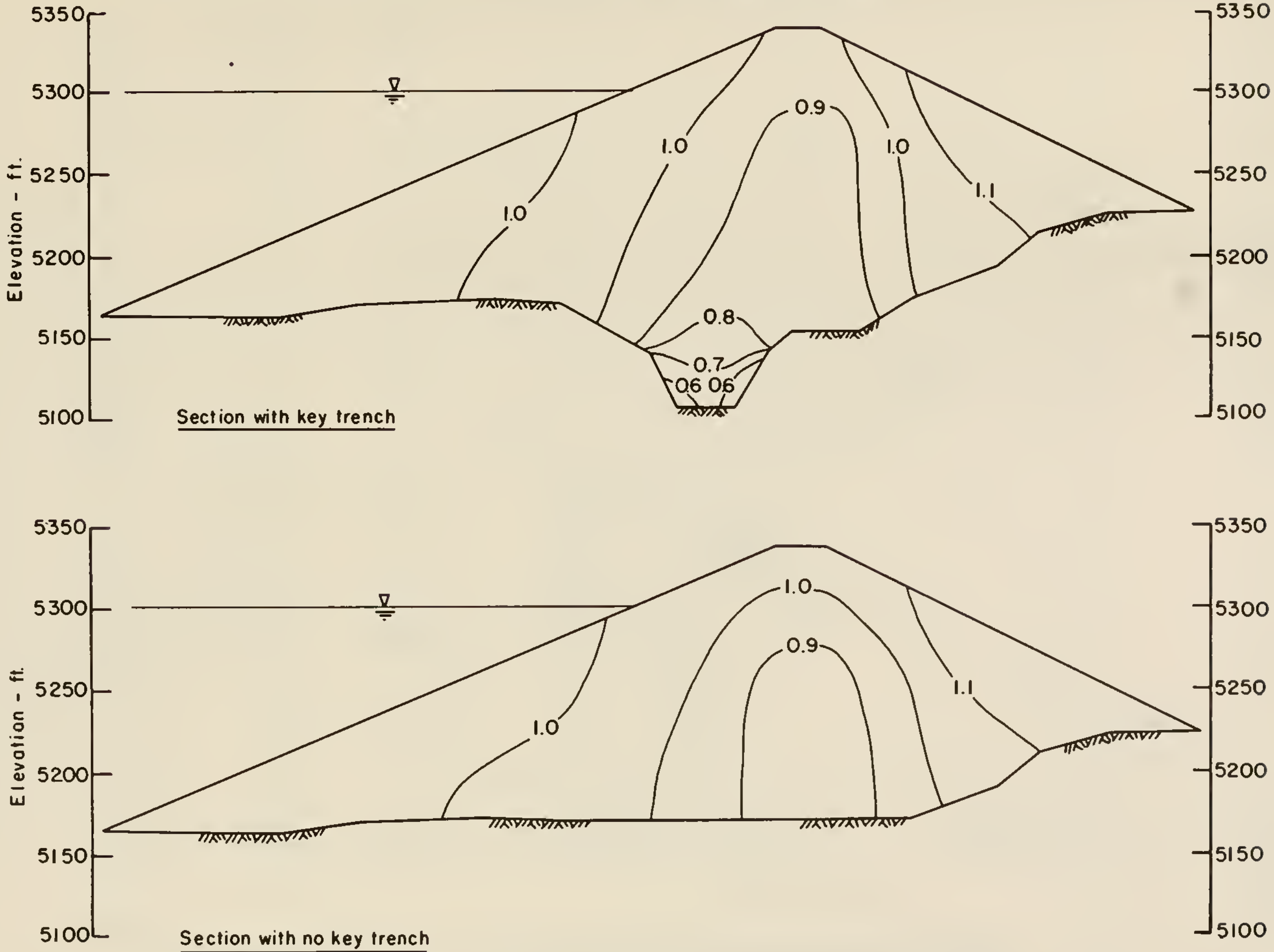


FIG. 19 CONTOURS SHOWING RATIO OF VERTICAL STRESS IN EMBANKMENT TO OVERBURDEN PRESSURE FOR CONDITIONS WITH AND WITHOUT KEY TRENCH AT STA. 15+00-BEFORE WETTING OF SOIL IN KEY TRENCH

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	1.50	435	650	0.12	0.75	0.32	0.10	4.0	0	0.003

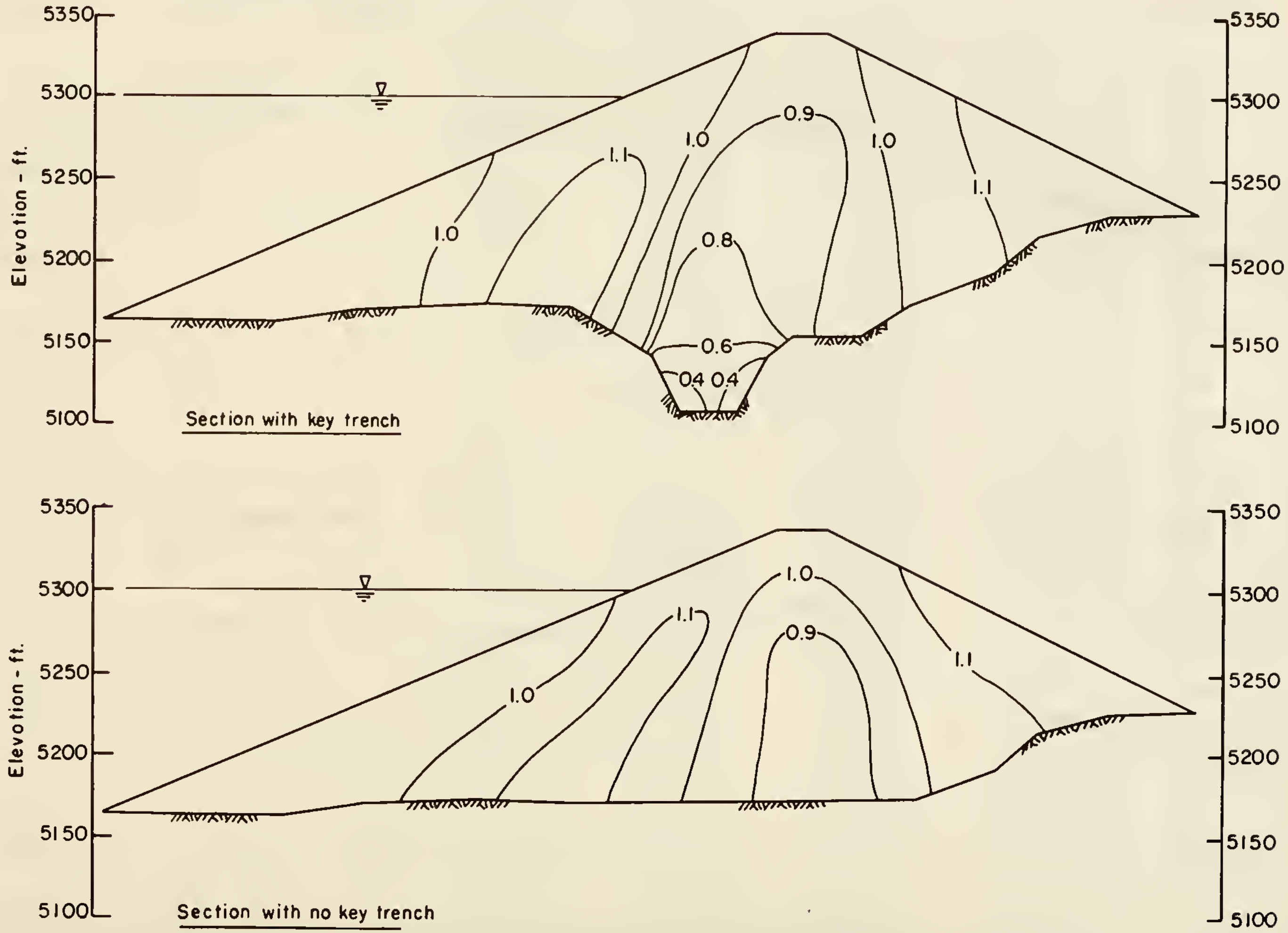


FIG.20 CONTOURS SHOWING RATIO OF VERTICAL STRESS IN EMBANKMENT TO OVERBURDEN PRESSURE FOR CONDITIONS WITH AND WITHOUT KEY TRENCH AT STA.15+00-AFTER WETTING OF SOIL IN KEY TRENCH

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).

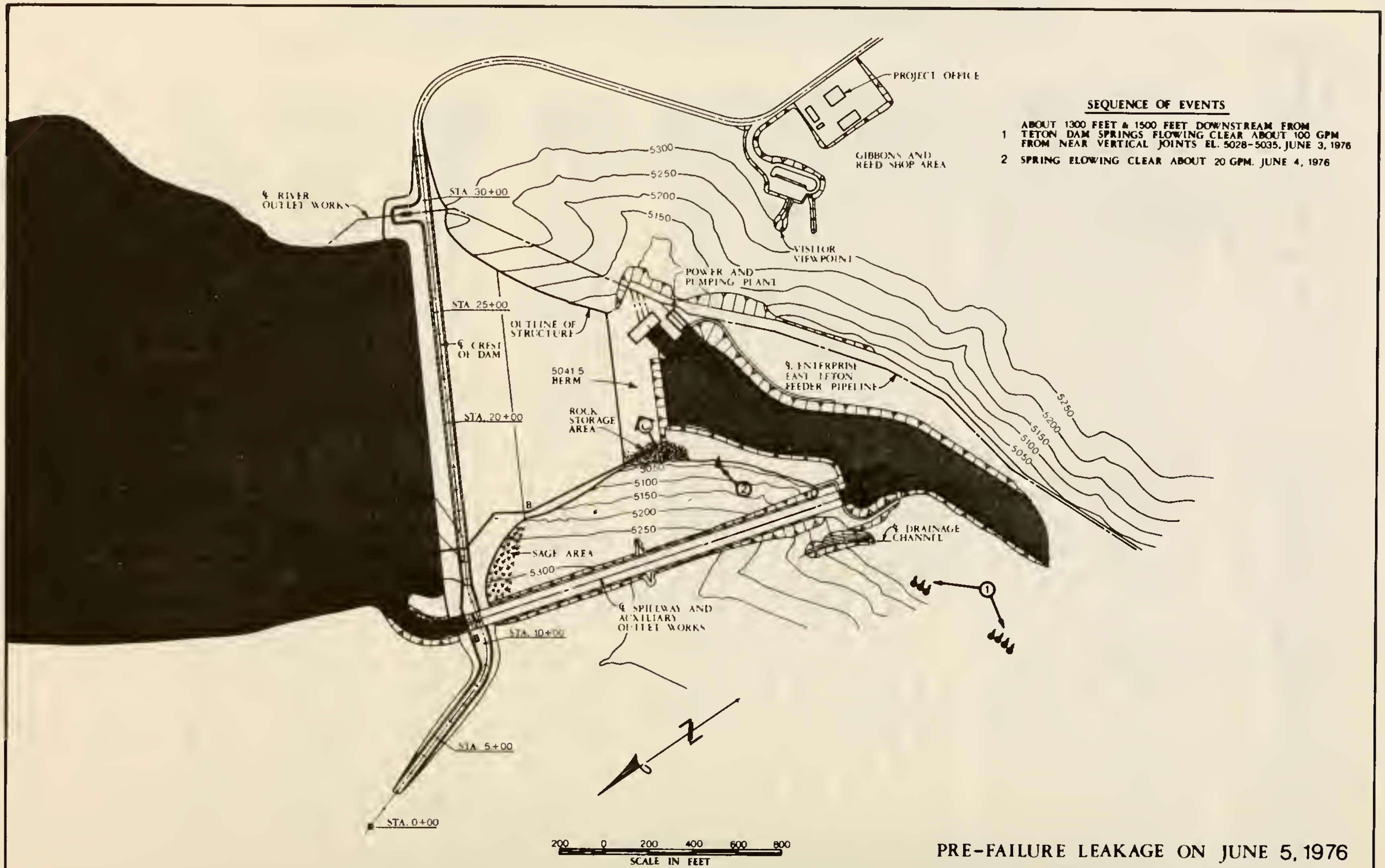
Thus the critical escape route for leakage was the multitude of partially filled void spaces in the loose slabby rock just beneath the Zone 1 fill downstream from the key trench. Significantly, materials partially filling void spaces in this zone of rock would be unaffected by overburden pressures from 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).



PRE-FAILURE LEAKAGE ON JUNE 5, 1976

FIG. 21. U. S. DEPARTMENT OF THE INTERIOR — STATE OF IDAHO INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE

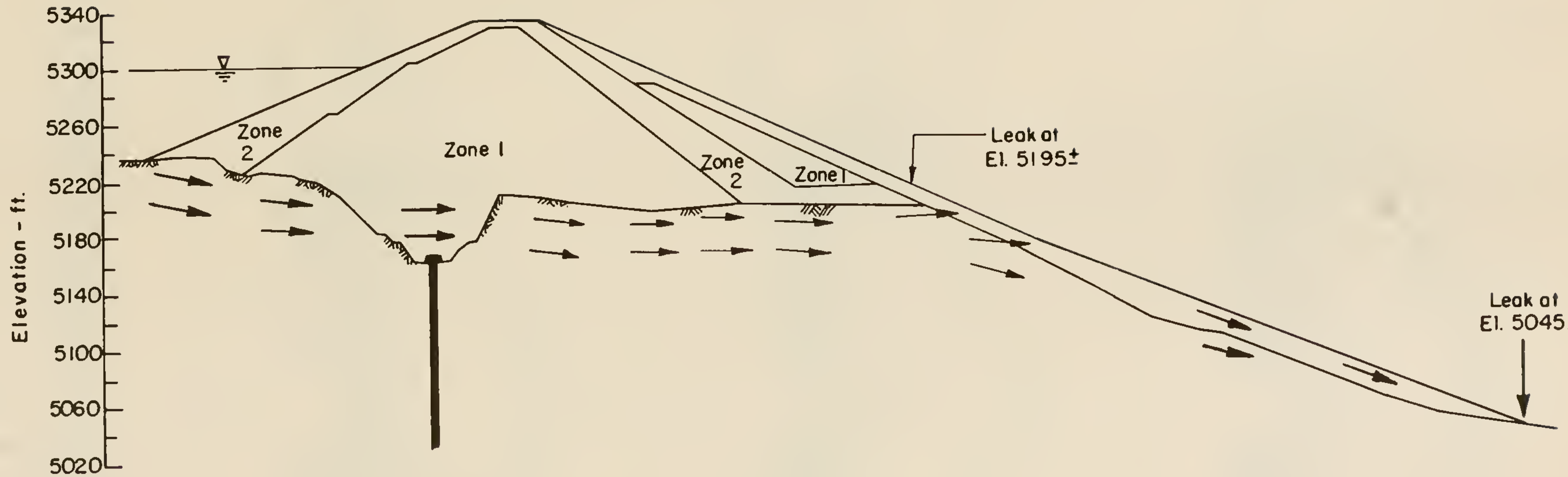
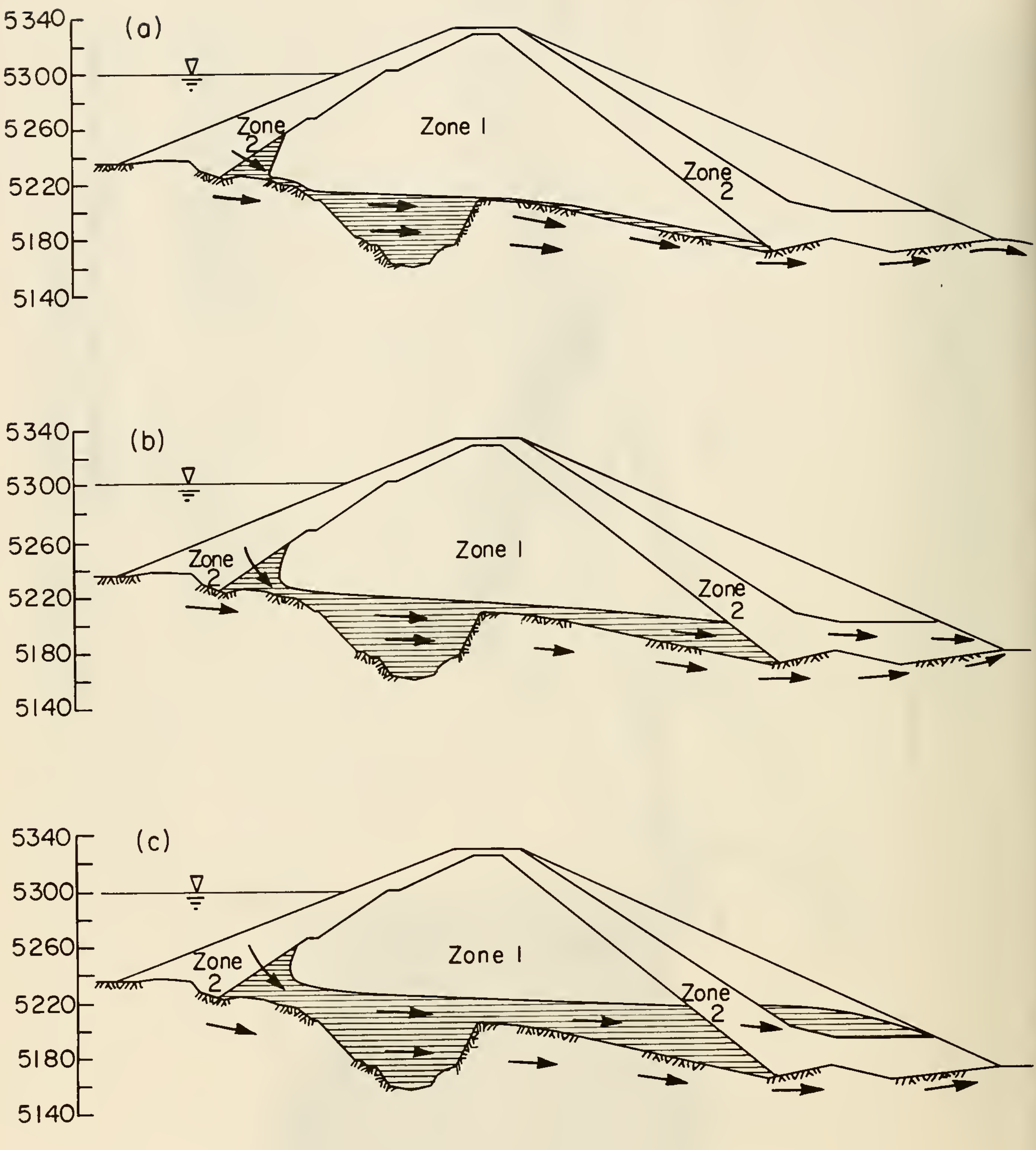
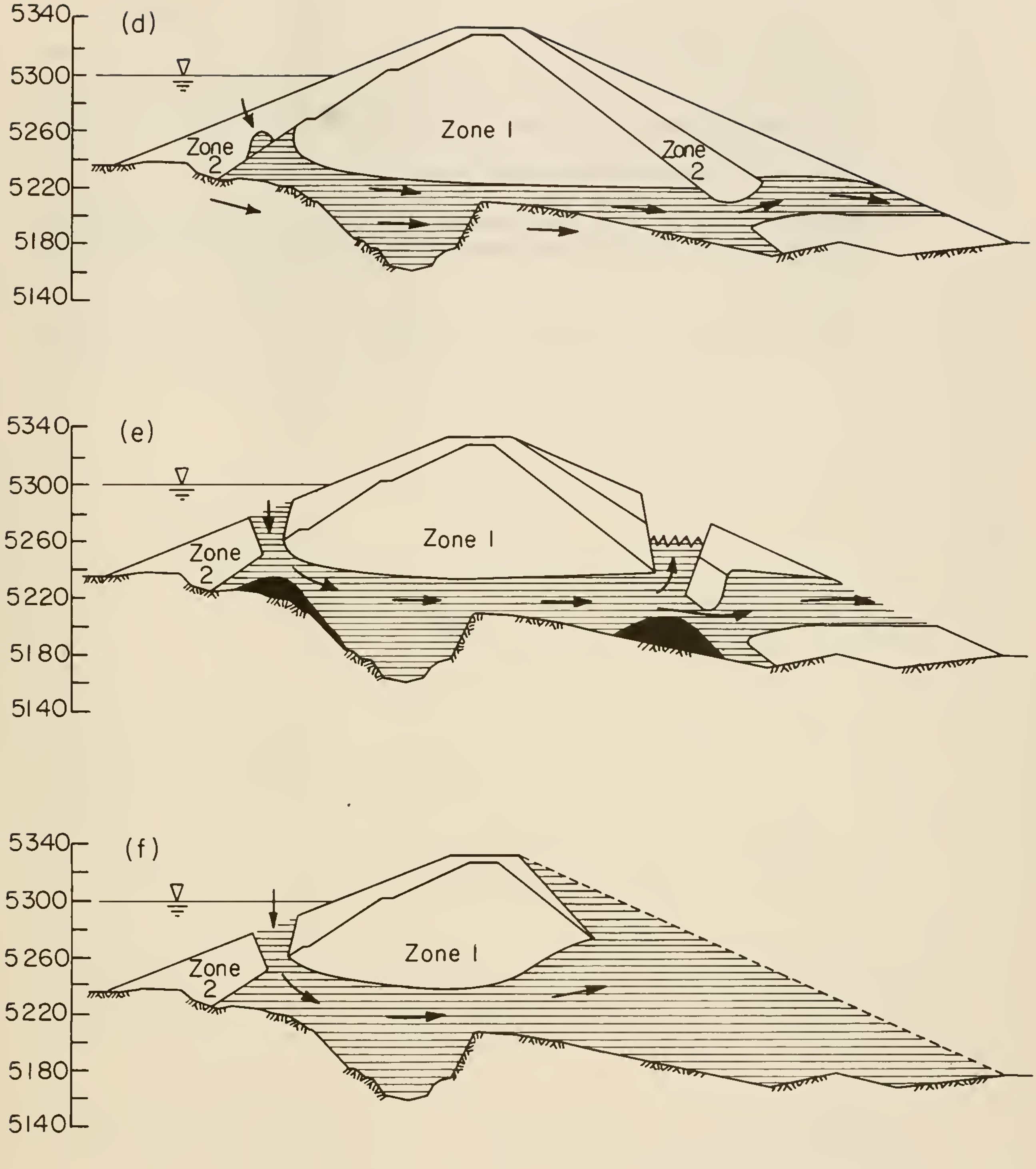


FIG. 22 PROBABLE PATH OF WATER IN EARLY STAGES OF LEAKAGE



(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.

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.

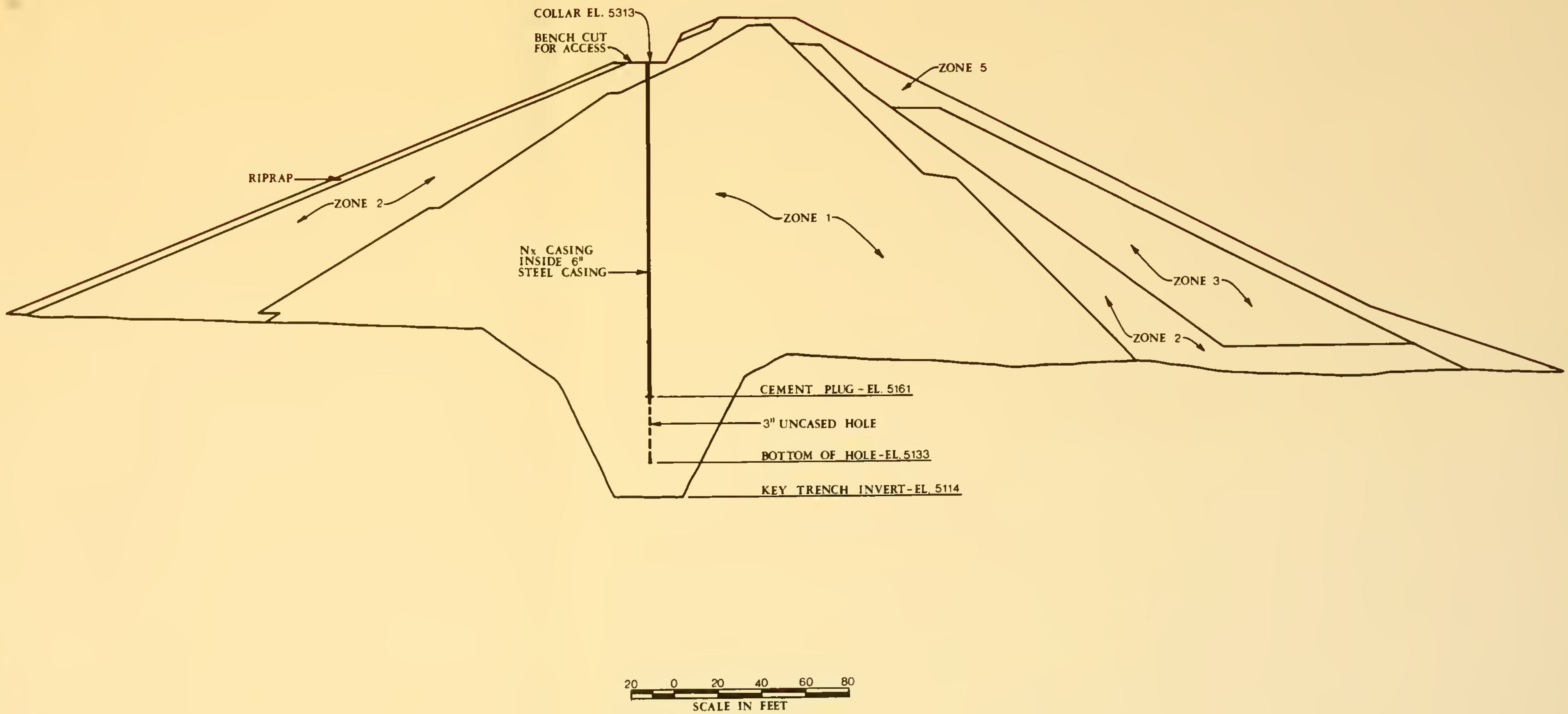
A geologic program was developed to investigate the following possible avenues of leakage through the foundation:

1. Around the right end of the dam.

2. Through the grout curtain.

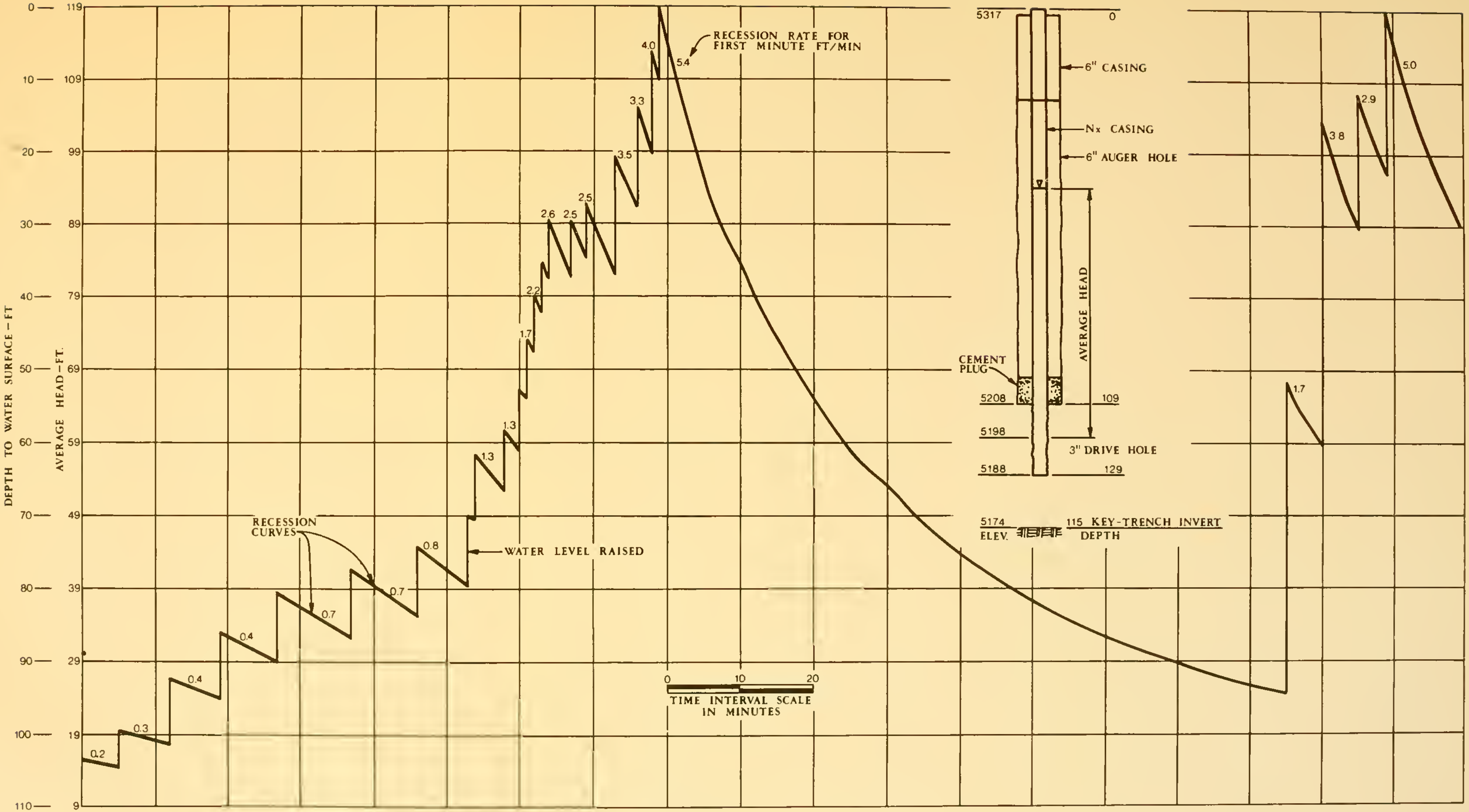
3. Through large cavities discovered near the right end of the dam during its construction.

4. Through sedimentary deposits underlying the volcanic rock foundation.



BORE HOLE STA. 26+00

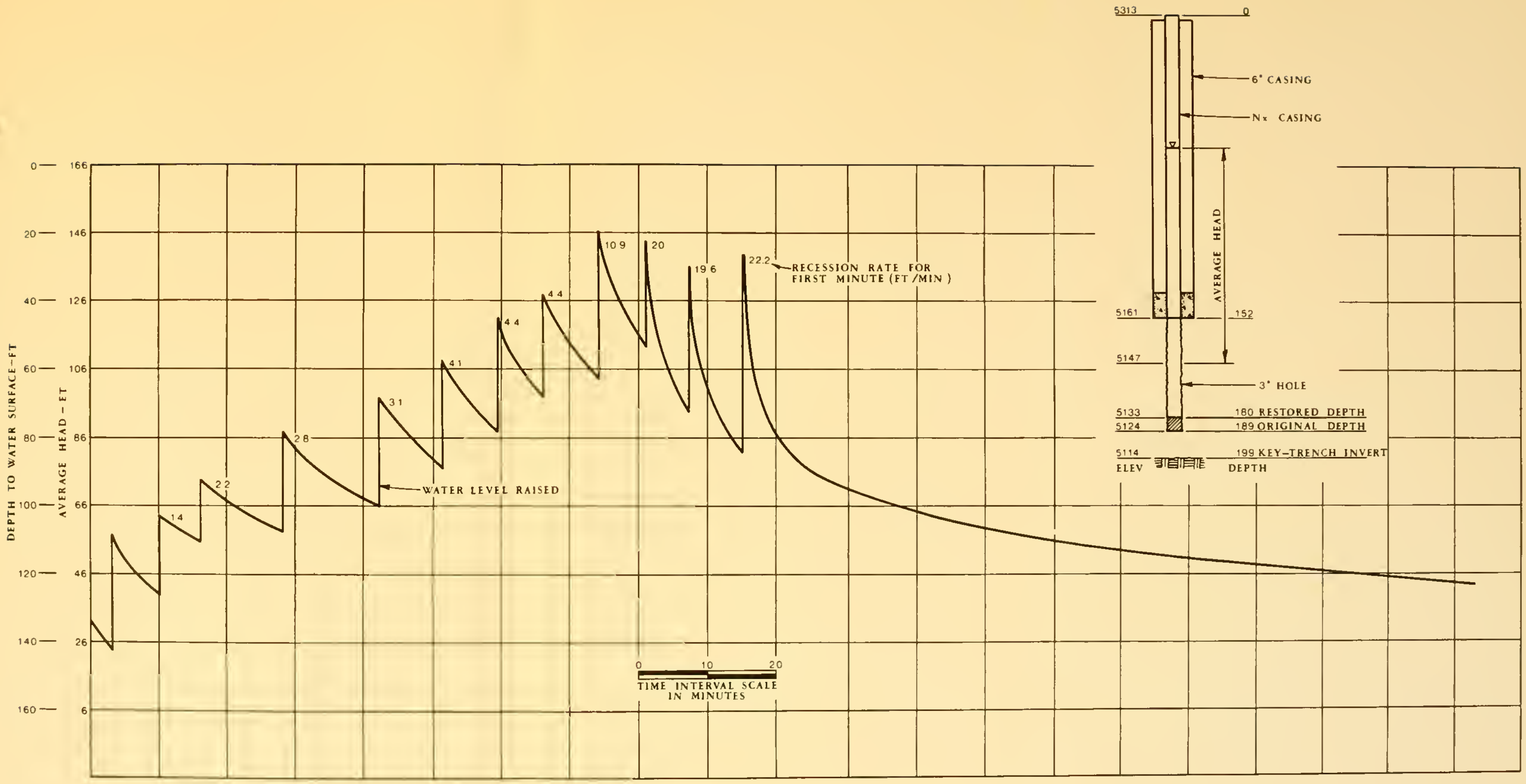
FIG. 3-6. U S DEPARTMENT OF THE INTERIOR — STATE OF IDAHO
INDEPENDENT PANEL TO REVIEW CAUSE OF TETON DAM FAILURE



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 RECESSION RATES

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