INVESTIGATION OF THE CAUSE OF QUAIL CREEK DIKE FAILURE

Report of Independent Review Team

March 7, 1989

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The Honorable Norman H. Bangerter
Governor of Utah
210 State Capitol
Salt Lake City, Utah 84114

Dear Governor Bangerter:

After failure of Quail Creek Dike on January 1, 1989, State Engineer Robert L. Morgan, at your direction, convened an Independent Review Team to investigate the failure. The Team was given four objectives, the first was to "Determine the mechanism causing the failure of the dike, including how the design or remedial work undertaken may have contributed to the failure." The Team has completed its investigation of this first objective and herewith submits its report titled "Investigation of the Cause of Quail Creek Dike Failure."

The investigation included a comprehensive review of the predesign investigations, design, construction, operation, and remedial work for the dike and reservoir. Field studies of the remaining dike and foundation were carried out at the request of the Team, and laboratory testing and data analysis were also accomplished.

Contributions forthcoming from an investigation of this type include "lessons learned" such that safer dams can be constructed in the future. While much is to be learned from this investigation, the Team believes that most of the lessons from the Quail Creek Dike failure should be more appropriately termed "relearned and reinforced." Within this context, the conclusions of the investigation are summarized as follows:

1. The primary conclusion is that failure resulted because embankment materials placed on the foundation, including overburden left in place, were <u>not protected from seepage erosion</u>.

2. Geologic conditions at the site with thinly bedded, highly gypsiferous sediments striking up and downstream and with a shallow dip toward the left abutment (southeast) were extremely challenging and deserved special consideration in design.

3. Fractures in the form of three major near vertical joint sets were present in the foundation and permitted significant seepage flow; foundation exploration was not designed or complete enough to fully detect seepage problems associated with these joints.

4. The early assumption that there would be little or no seepage through the dike foundation below the shallow cutoff was not valid and had a profound effect on design of seepage erosion protection.

5. Highly fractured, pervious rock and erodible overburden was left in place upstream and downstream of the cutoff permitting seepage along the foundation contact.

6. Upstream-downstream trending hogback ridges were left in place with intervening valleys filled from upstream toe to downstream toe with unprotected and erodible Zone I material in intimate contact with open conduits in the fractured, pervious rock foundation.

7. The presence of considerable gypsum in the foundation was not the primary cause of failure; however, as time passed, the solutioning of gypsum allowed increased volume and velocity of seepage near the contact, thus hastening the erosion process.

8. Remedial grouting was not a long term solution for seepage control of this foundation as demonstrated by the shifting locations of seepage emergence during grouting and sporadic outbreaks of new seepage after completion of each episode of remedial grouting.

9. There is piezometric and field evidence that remedial grouting restricted downstream drainage channels in the rock foundation increasing hydraulic pressure against the embankment/foundation contact, enhancing conditions for piping at the contact.

10. Filter criteria was not met in the downstream toe drain which was invaded by eroded fine-grained material; the toe drain may have accelerated (but did not cause) failure by providing a closer uncontrolled exit for eroded materials than the original uncontrolled seepage exits.

11. There is no indication that seepage through the dike embankment or the quality of its construction contributed to the failure.

The Independent Review Team would like to take this opportunity to commend and thank the State Engineer's staff and the Washington County Water Conservancy District for their assistance in expeditiously carrying out the exploration and testing phases of this investigation. The Team also thanks all of the individuals interviewed and appreciates the frank responses to many probing questions.

Respectfully submitted,

Richard B. Catanach

Robert L. James, Chairman

Alan L. O'Neill

J. Lawrence Von Thun

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INVESTIGATION OF THE CAUSE OF QUAIL CREEK DIKE FAILURE

1. OBJECTIVE

As a result of the breach failure of the Quail Creek Dike on January 1, 1989, the State Engineer, at the request of the Governor of Utah, appointed an Independent Review Team to investigate the failure. At the initial Team meeting in St. George, Utah, on January 11, 1989, the Team was given four objectives:

A. Determine the mechanism causing the failure of the dike, including how the design or remedial work undertaken may have contributed to the failure.

B. Given the current technology, can a safe structure be rebuilt at the site, and if so, what conceptual design features should be incorporated into the dike and foundation to prevent a similar failure or other possible failures.

C. Is the main dam, in its present state, safe, or are additional studies, remedial action, or additional monitoring needed.

D. If the dike can be rebuilt, are the proposed plans and specifications adequate to prevent a similar failure and resolve any other latent deficiencies.

This report specifically deals with objective A, determining the cause of failure, and is the result of investigation and deliberation by Messrs. Richard B. Catanach, Robert L. James, Alan L. O'Neill, and J. Lawrence Von Thun. The Team's objective is to determine and present the technical facts concerning design and construction of the dike, remedial actions taken, and field investigations and testing (to the extent these facts can be determined), and from this review, establish the mechanism of failure.

2. BACKGROUND

2.1 Project Description

Quail Creek Reservoir is an offstream water storage project located in Washington County, Utah, and is owned and operated by the Washington County Water Conservancy District (WCWCD). Project location is shown on Figure 2-1 and the reservoir basin on Figure 2-2. The project provides for storage of approximately 40,000 acre-feet of irrigation and municipal and industrial water. Water is supplied to the offstream reservoir by a 66-inch pipeline from a diversion dam on the Virgin River about 2 miles upstream from Hurricane, Utah. The project includes the diversion dam, pipeline, and two small hydroelectric generating plants. The reservoir is formed by Quail Creek Dam which is a zoned embankment with a maximum height of 200 feet and length of 900 feet, and Quail Creek Dike which is a zoned embankment with a maximum height of 78 feet and length of 1,980 feet. Crest elevation for dam and dike is 2995. Other features include a tunnel intake and outlet facility near the left abutment of the dam and an uncontrolled chute spillway, crest elevation 2985, near the left abutment of the dike.

Project design was started in 1982 and completed in 1983. The firm of Creamer and Noble planned and designed the overall project while the firm of Rollins, Brown, and Gunnell designed the dam, dike, outlet, and spillway. A contract for construction of the dam, dike, outlet, and spillway was awarded to S. J. Groves and Sons in October 1983. The dike was completed in April 1984 and the dam in January 1985. Reservoir filling was started in April 1985.

2.2 Dike Description

The plan along centerline of the Quail Creek Dike is shown on Figure 2-3 and typical sections are shown on Figure 2-4. The dike has a maximum height of 78 feet, a crest width of 20 feet, and a length of 1,980 feet. Foundation treatment consisted of a 30-foot-wide centerline cutoff trench through weathered rock to an average depth of 10 feet and nominal stripping to a depth of 1 foot. Exposed rock contacts were not treated, and grouting was limited to a sandstone section in the left abutment between Stations 0+00 and 2+00. As shown on Figure 2-4, the embankment consists of a centrally located Zone I of generally low-plasticity silty and clayey sands with a 1-foot blanket of medium-plasticity weathered shale (Zone II) on the bottom of the cutoff trench and a 10-foot-thick layer of Zone II along the upstream face of the Zone I. A 4-foot-wide vertical processed sand filter is located downstream of the Zone I. The upstream shell is a pit-run sandy gravel (Zone III), and the downstream shell is random fill (Zone IV) enveloped by Zone III gravels. Upstream slope protection consists of 18 inches of dumped basalt riprap. Dike instrumentation consisted of six open standpipe piezometers and six crest bench marks located as shown on Figure 2-3. During construction, Zone I material was used to level the stripped foundation, and since the bedrock topography consists of a series of hogback ridges and valleys oriented perpendicular to the dike axis, at some locations more than 10 feet of Zone I was placed over the foundation from the upstream toe to the downstream toe (see Photograph 8). This is illustrated by the as-constructed section at Sta. 6+50 shown on Figure 2-5.

2.2.1 Dike Performance: As the reservoir rose to elevation 2935, seepage appeared along the downstream toe. Initial actions in response to the seepage flows (March 1986) included placing a weighted filter blanket of concrete sand, pea gravel, and coarse gravel from Sta. 4+50 to Sta. 8+00 and drilling 18 inclined drain holes along the downstream toe from Sta. 6+40 to Sta. 10+00. In April 1986, a contract was initiated to grout the dike foundation from the top of the dike through the embankment. In May 1986, with the reservoir at elevation 2969, the estimated seepage flow was 6.3 cubic feet per second (cfs) with major seepage concentrations downstream of the toe at Sta.s 3+00 and 6+50. Grouting was completed in September 1986, and seepage flow was estimated at 0.3 cfs at reservoir elevation 2949. In October 1986, a contract was let to install an upstream cutoff trench and partially blanket the reservoir for 500 feet upstream of the upstream toe. Location and configuration of the upstream cutoff are shown on Figure 2-6. In June 1987, seepage started to increase downstream near Sta. 6+00. Grouting was resumed in July 1987 and continued to April 1988 when seepage was reduced to approximately 1 cfs at reservoir elevation 2985. During the summer of 1988 seepage increased to about 5 cfs primarily at Sta. 4+30. Grouting was resumed in September 1988 and stopped in November 1988, with seepage reduced to 0.1 cfs at reservoir elevation 2976.

2.3 Failure Description

Based on eyewitness accounts, the first indication of the potential failure was on December 31, 1988, with the observed upward flow of discolored water around an observation pipe located at about Sta. 5+90 (see Figure 2-6). Equipment and materials were mobilized to place a gravel filter over the seep area. Despite continued efforts to control the flow, the volume increased to an estimated 70 cfs at about 10:30 p.m. when the flow changed from vertical to horizontal from a rapidly growing cavity at the toe. At this point, personnel and equipment were removed and an emergency downstream evacuation ordered. At about 11:30 p.m., a wedge of the downstream slope about 50 feet wide and extending about one-third of the way up the slope dropped down. Continuing embankment caving toward the reservoir crossed the dike crest and breached the dike, releasing the reservoir at about 12:30 a.m. on January 1, 1989. By 1:00 p.m. on January 1, about 25,000 acre-feet of water had drained from the reservoir, resulting in a breach about 300 feet wide and 80 to 90 feet deep.

2.4 Chronology of Major Events

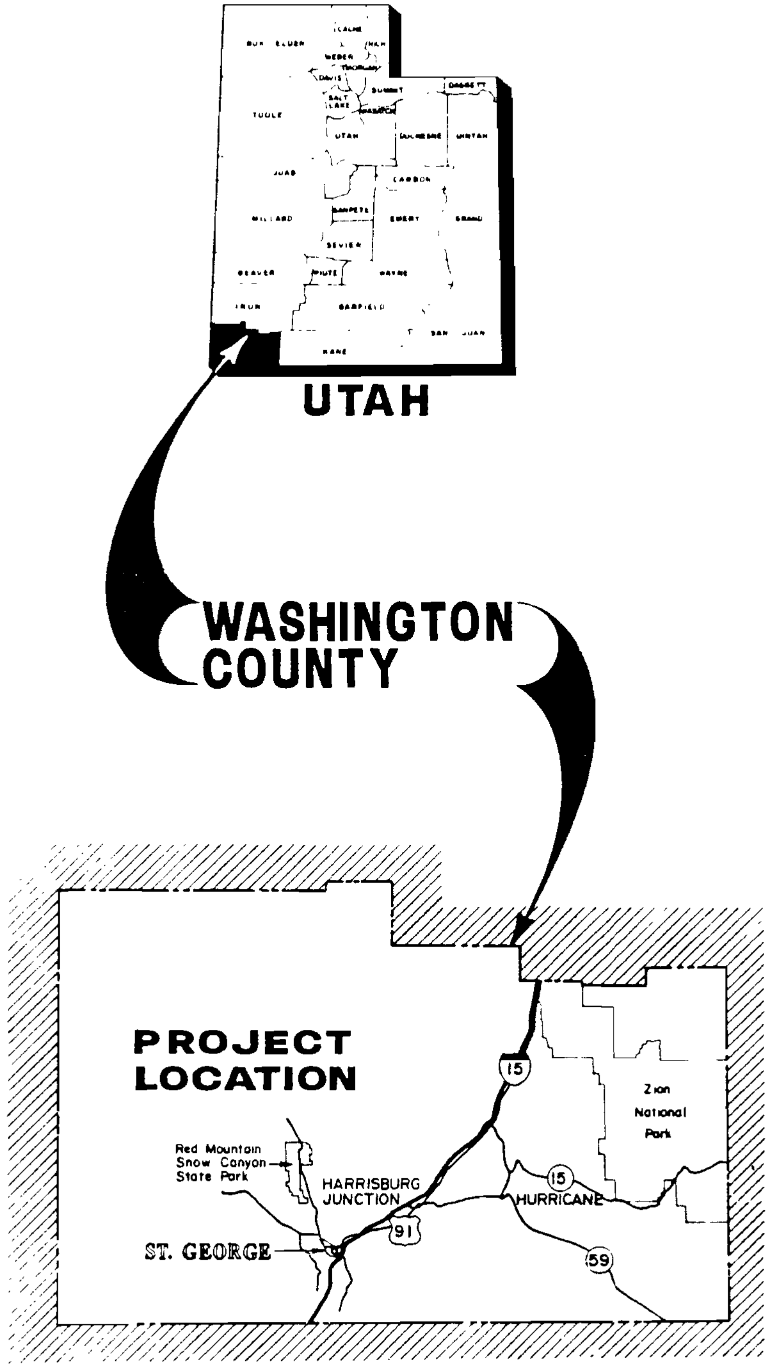
A brief chronology of major events concerning Quail Creek Dike based on Reference 17[1] is as follows:

1982	Preliminary design was completed.
1983	Final design was completed.
Oct. 5, 1983	Bids were opened.
Nov. 1983	S. J. Groves and Sons began construction.
April 1984	Dike construction was completed.
Jan. 1985	Dam construction was completed.
April 1985	Reservoir filling began.
Feb. 1986	St. George City made a cut to install pipelines through dike at approximately Sta. 12+00.
March 1986	Aardvark installed 18 drain wells (4-1/2-inch outside diameter with slotted polyvinyl chloride pipe) between Sta.s 6+40 and 10+00.
April 1986	Plans and specifications were prepared for dike foundation, pressure grouting and dam blanket drain, and berm.
April 22, 1986	Boyles Bros. were awarded dike grouting contract.
April 29, 1986	Interstate Rock was awarded dam blanket drain and berm contract.
May 1986	Work began on dike and dam. Reservoir was at approximate elevation 2969. Estimated total flow from dike was 6.3 cfs. Major seepage areas appeared at approximately Sta.s 6+50 and 3+00 downstream of toe.
July 2, 1986	Dam blanket drain and berm were completed.
Sept. 11, 1986	Grouting was completed on dike. Flow was estimated at 0.3 cfs with reservoir level at 2949.
Oct. 21, 1986	Decision was made to install upstream cutoff at dike.
Oct.-Dec. 1986	Interstate Rock completed upstream cutoff and partial blanketing of area from upstream toe 500 feet upstream.
June-July 1987	Substantial seepage developed in vicinity of Sta. 6+00.
July 1987	Boyles Bros. were mobilized for grouting operation. Reservoir level was approximately 2970. Flow was difficult to locate, then very difficult to close off. Main seepage was near contact between Sta.s 6+15 and 6+35. Decision was made to grout left abutment.
April 1988	Grouting work was completed. Approximately 1 cfs was being collected in 12-inch flume and 2-1/2 cfs in 36-inch flume with reservoir full (El. 2985).
Summer 1988	Seepage increased to near 5 cfs in 12-inch flume by end of August. Color starts showing.
Aug. 30, 1988	Boyles mobile to site. Main seepage source was found on Sept. 21, 1988, 130 feet down from crest (approximately 40 ft below contact). Seepage was extremely difficult to stop; finally stopped it using

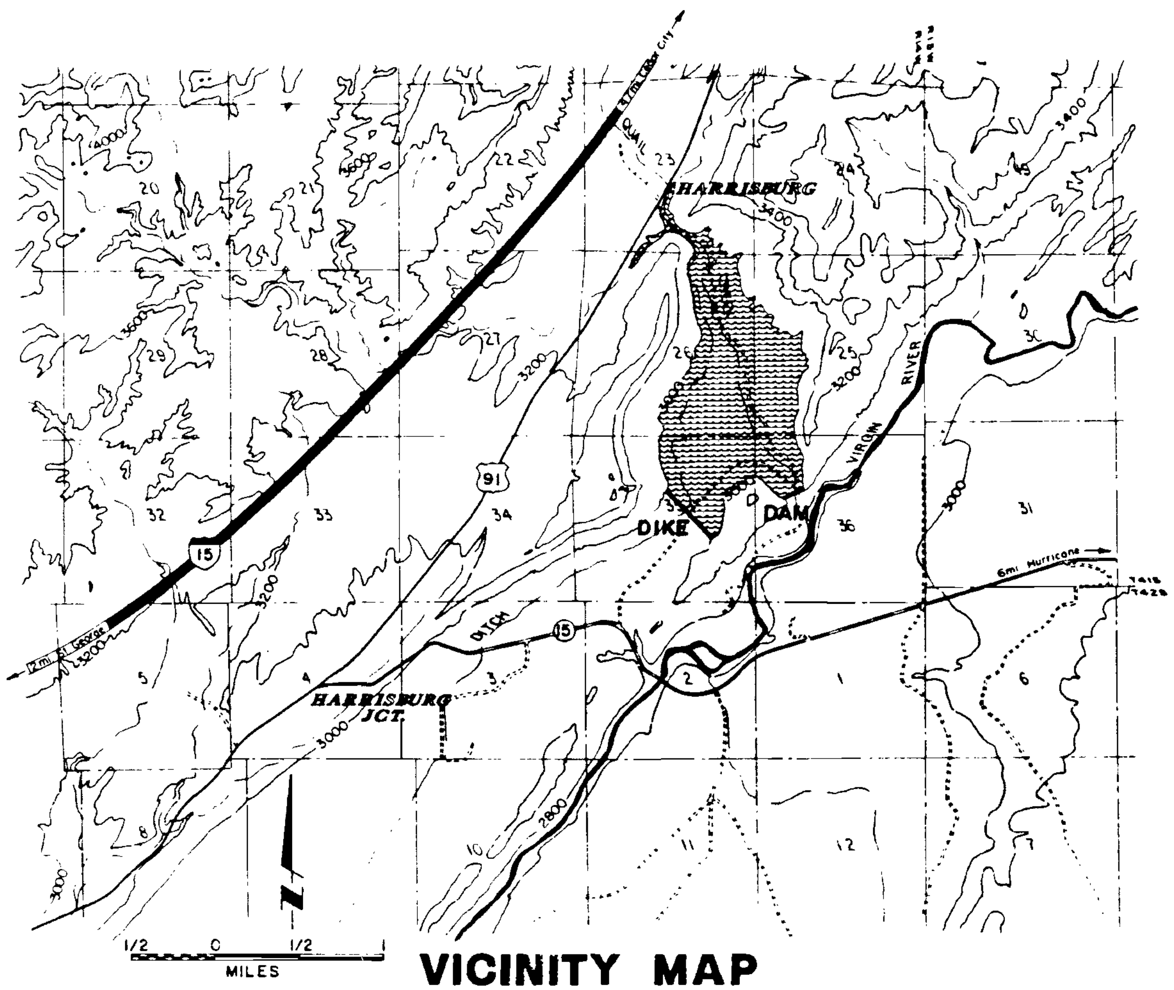
[1]References are given in the list of provided materials in Appendix A of this report.

	several Redi-mix trucks pumping concrete through 125 feet of casing.
Nov. 18, 1988	Grouting was stopped. With reservoir at elevation 2976, 12-inch flume read 0.1 cfs and 36-inch flume 0.7 cfs.
Dec. 31, 1988	Increased discolored seepage was observed at observation well near downstream toe at Sta. 5+90.
Jan. 1, 1989	Quail Creek Dike breached at about 12:30 a.m.

QUAIL CREEK DAM AND DIKE



QUAIL CREEK DAM AND DIKE SITE IS LOCATED IN WASHINGTON COUNTY, SECTIONS 23, 25, 26, 35 AND 36, TOWNSHIP 41 SOUTH, RANGE 14 WEST OF THE SALT LAKE BASE AND MERIDIAN.



VICINITY MAP

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3	DAM PLAN VIEW
4	DAM CROSS SECTION
5	DAM FOUNDATION
6	FOUNDATION DETAILS
7	GROUT CURTAIN
8	DIKE PLAN VIEW
9	DIKE CROSS SECTION
10	OUTLET WORKS TUNNEL
11	OUTLET WORKS INTAKE STRUCTURE
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7A	GROUT CURTAIN
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8B	DIKE GROUT CURTAIN
8C	DIKE GROUT CURTAIN
8D	DIKE GROUT CURTAIN
12A	OUTLET PIPE LAYOUT AND DETAILS

OWNER'S ACCEPTANCE

Washington County Water Conservancy District certifies that the company accepts the plans and specifications for the construction of the Quail Creek Dam and Dike.

Date

ENGINEER'S CERTIFICATE

Rollins, Brown & Gunnell, Inc., certifies that the corporation was employed to prepare the construction drawings and specifications for the proposed Quail Creek Dam and Dike to be constructed near St. George, Utah and that these drawings consisting of 27 sheets and the accompanying specifications indicate the information to be submitted for approval.

Ralph L. Rollins 12/5/83 No. 2366
President, Rollins, Brown & Gunnell Date Professional Engineers license number

APPROVED

In accordance with Title 73, Chapter 10, Utah Code Annotated, 1953, as ammended, approval is hereby given for the construction of the Quail Creek Dam and Dike project near St George, Utah.

Daniel F. Lawrence 12 28 1983
Director, Utah Division of Water Resources Date

APPROVED

In accordance with S73-5-5, Utah Code annotated 1953, as amended, approval is hereby given for the construction of the Quail Creek Dam and Dike project near St. George, Utah.

Utah State Engineer Date

AS CONSTRUCTED

DESIGNED BY R.L. ROLLINS DATE 5/83 DRAWN BY P.L. Davis DATE 5/83 CHECKED BY DATE	ROLLINS, BROWN AND GUNNELL, INC. PROFESSIONAL ENGINEERS	WASHINGTON COUNTY WATER CONSERVANCY DISTRICT	QUAIL CREEK DAM AND DIKE	Figure 2-1

Figure 2-1

Quail Creek Project Location

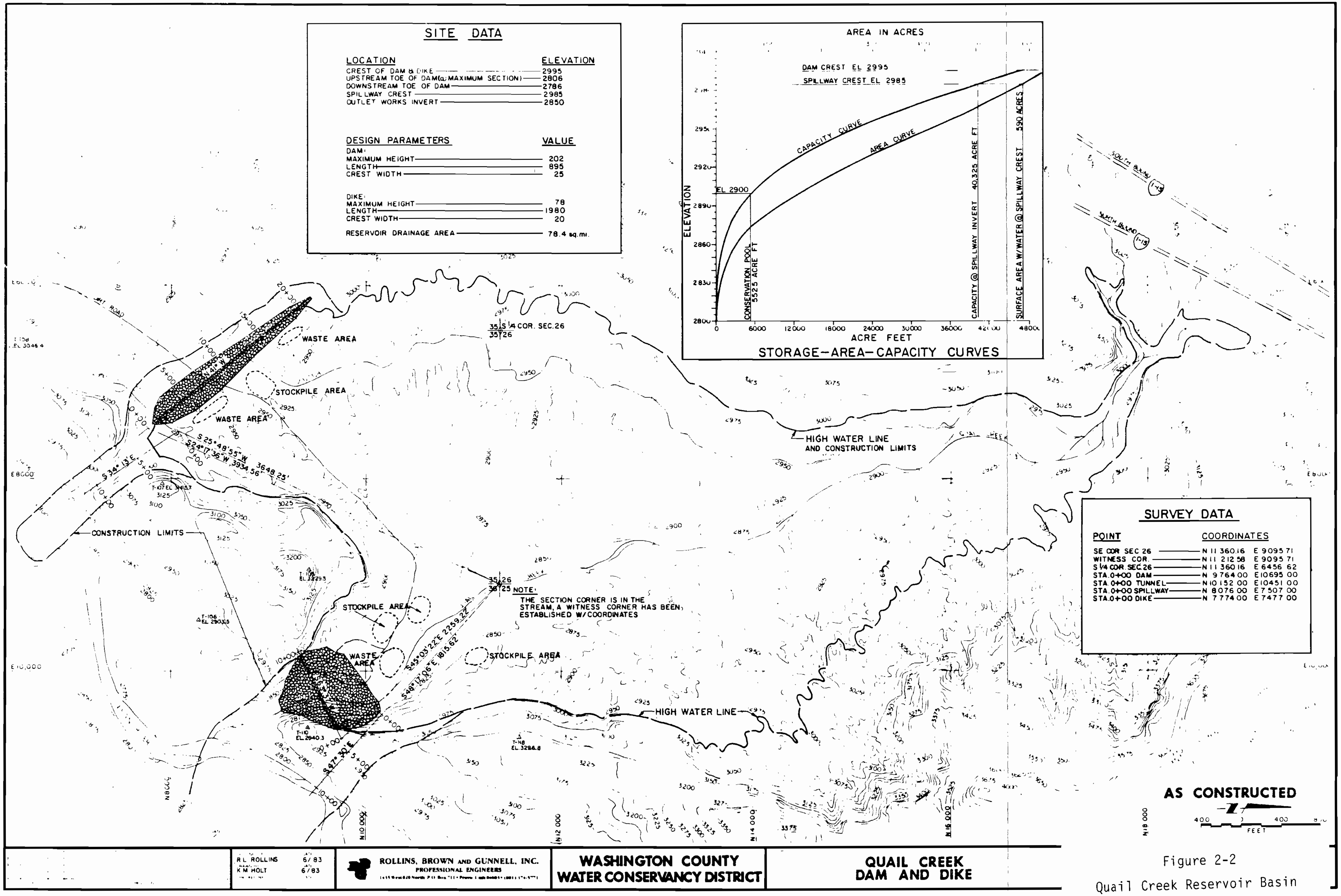
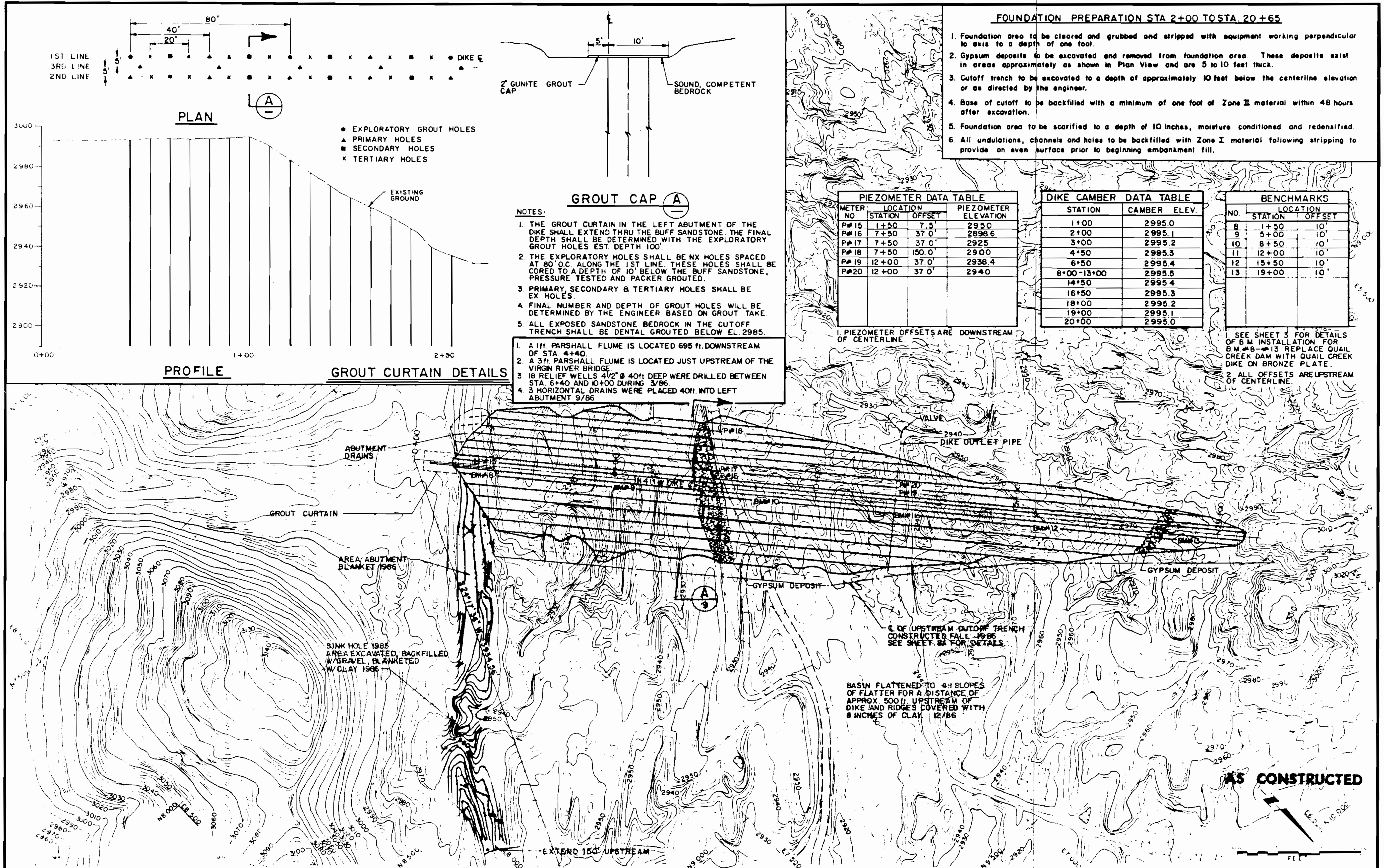


Figure 2-2
Quail Creek Reservoir Basin



PIEZOMETER DATA TABLE

METER NO.	LOCATION STATION	OFFSET	PIEZOMETER ELEVATION
P#15	1+50	7.5'	2950
P#16	7+50	37.0'	2898.6
P#17	7+50	37.0'	2925
P#18	7+50	150.0'	2900
P#19	12+00	37.0'	2938.4
P#20	12+00	37.0'	2940

DIKE CAMBER DATA TABLE

STATION	CAMBER ELEV.
1+00	2995.0
2+00	2995.1
3+00	2995.2
4+50	2995.3
6+50	2995.4
8+00-13+00	2995.5
14+50	2995.4
16+50	2995.3
18+00	2995.2
19+00	2995.1
20+00	2995.0

BENCHMARKS

NO	LOCATION STATION	OFFSET
8	1+50	10'
9	5+00	10'
10	8+50	10'
11	12+00	10'
12	15+50	10'
13	19+00	10'

R.L. ROLLINS 6/83
K.M. HOLT 6/83

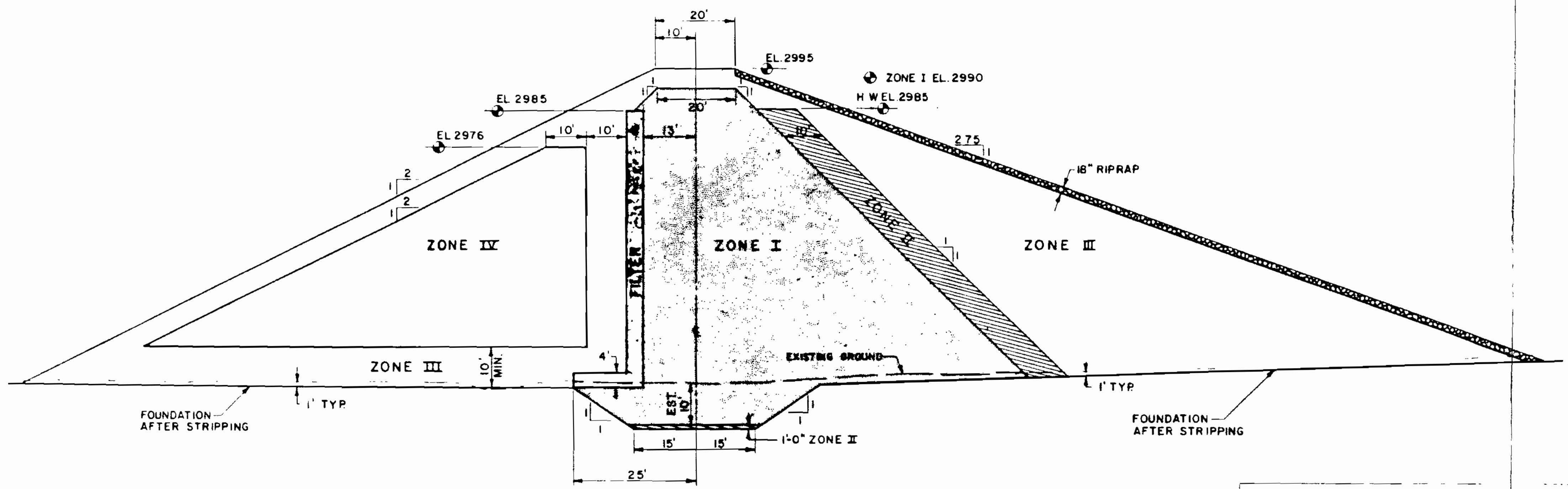
ROLLINS, BROWN AND GUNNELL, INC.
PROFESSIONAL ENGINEERS

WASHINGTON COUNTY
WATER CONSERVANCY DISTRICT

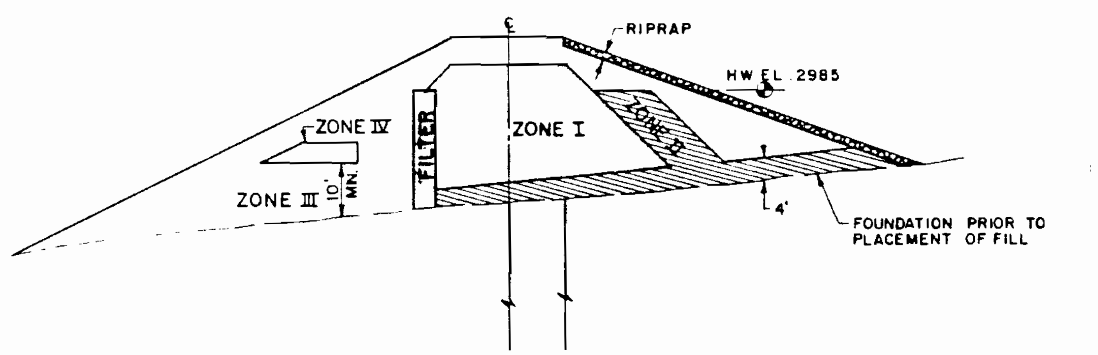
QUAIL CREEK
DAM AND DIKE

Figure 2-3

Quail Creek Dike - Plan



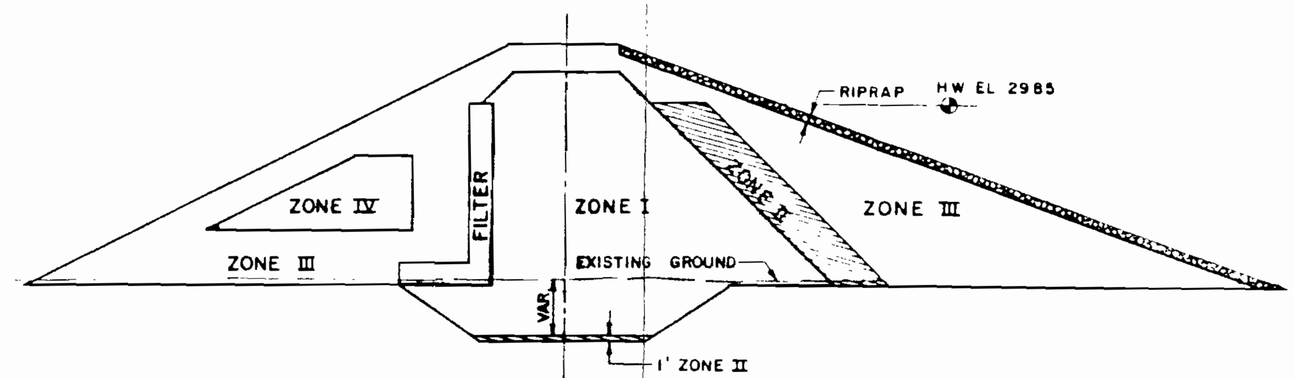
SECTION (A/8)



TYPICAL SECTION FOR FOUNDATION ON SANDSTONE
(APPROX. STA.1+00 TO STA.2+00)

EARTHWORK SUMMARY

ITEM	MATERIAL TYPE	SOURCE	ESTIMATED QUANTITY CU.YDS. (DAM)
COMMON EXCAVATION EMBANKMENT FOUNDATION	WEATHERED SHALE, SILT, SAND, AND GYPSUM	EMBANKMENT FOUNDATION	25,400
COMMON EXCAVATION CUTOFF TRENCH	WEATHERED SHALE	CUTOFF TRENCH	28,700
EARTHFILL ZONE I	SILTY SAND & SANDY SILT Min 15% Passing No 200 Sieve	BORROW AREA NO I RESERVOIR BASIN	196,400
EARTHFILL ZONE II	CLAY	BORROW AREAS 2A, 2B	31,900
EARTHFILL ZONE III	SANDY GRAVEL Less than 5% Passing No. 200 Sieve	BORROW AREA NO 3	287,900
EARTHFILL ZONE IV	RANDOM ZONE, SAND,	MATERIAL NOT ACCEPTABLE AS ZONE I or ZONE III from BORROW AREAS 1 & 3 & required excavations	88,000
FILTER	PROCESSED MATERIAL See Gradation requirements Sheet 4	BORROW AREA NO 3	14,500
RIPRAP	BASALT	BASALT ROCK BORROW AREA 3	13,800



TYPICAL SECTION FOR FOUNDATION ON WEATHERED SHALE
(APPROX. STA.2+00 TO 20+65)

AS CONSTRUCTED

R.L. ROLLINS 7/83 K.M. HOLT 7/83	ROLLINS, BROWN AND GUNNELL, INC. PROFESSIONAL ENGINEERS	WASHINGTON COUNTY WATER CONSERVANCY DISTRICT	QUAIL CREEK DAM AND DIKE	Figure 2-4

Quail Creek Dike
Typical Sections

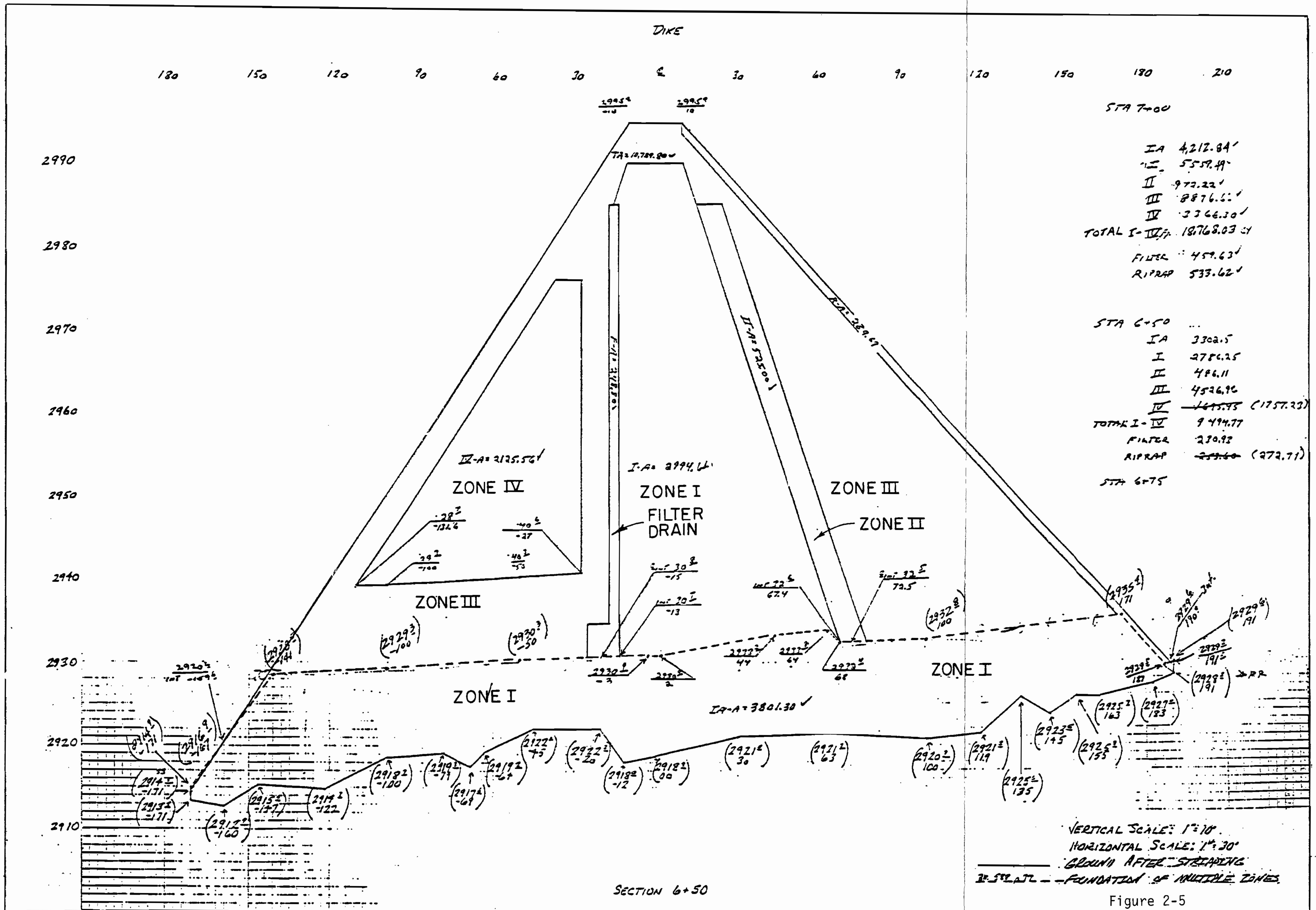


Figure 2-5

As Constructed Section 6+50
Final Embankment
Cross Sections and Quantities

NOTE: East T.P. Drain Is At Location Of Concentrated Seep Prior To Failure.

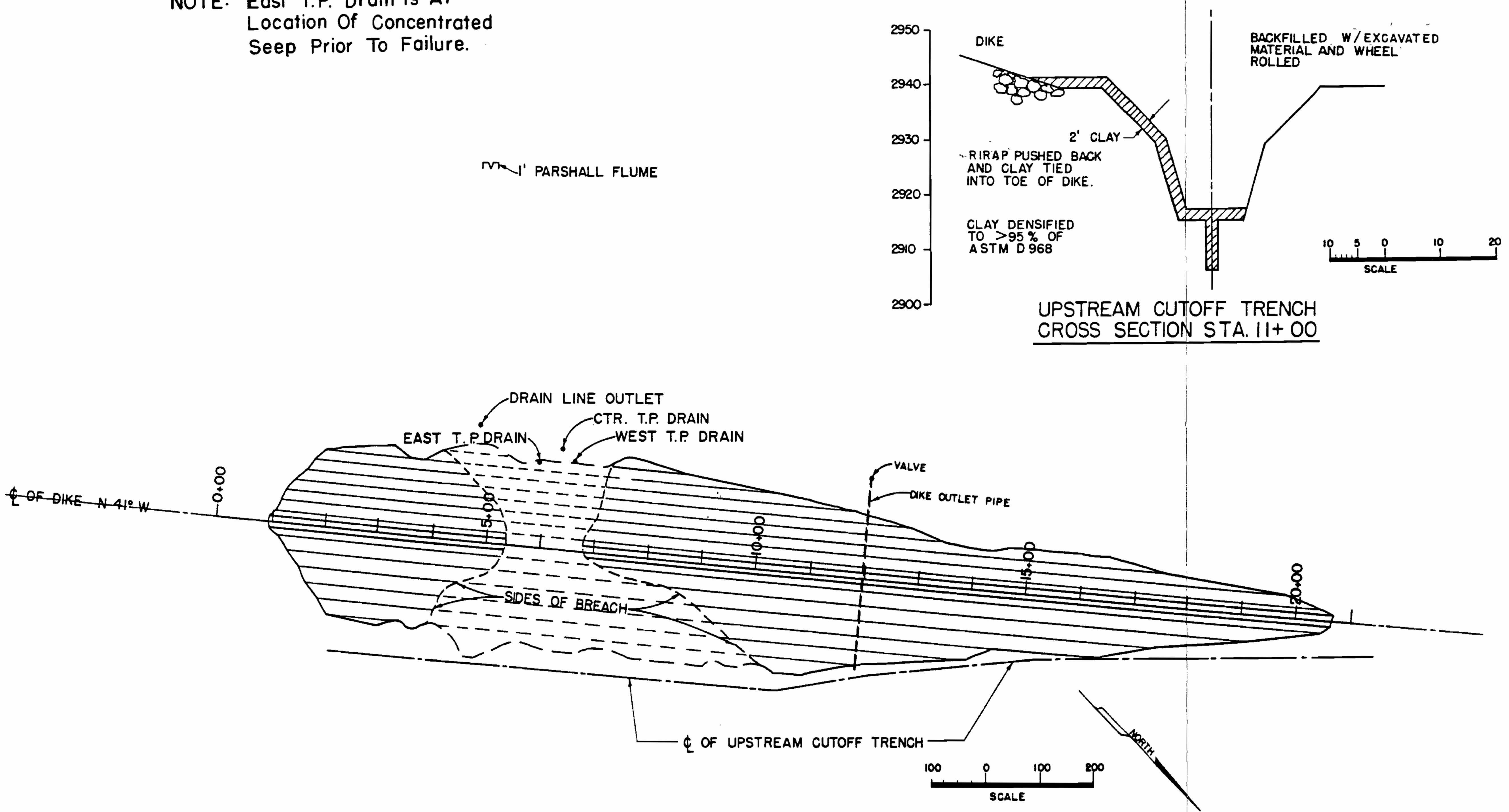


Figure 2-6

Upstream Cutoff Trench and Plan View of Area of Failure

3. SCOPE OF INVESTIGATIONS

The State Engineer's office provided the Team with documents concerning the design, construction, and performance of Quail Creek Dike; see Appendix A for a summary of these documents. In addition, the staff of the State Engineer's office provided engineering and geologic services to the Team in the form of soil and rock sampling and testing and geologic mapping. The Team also requested that explorations be undertaken in the foundation rock of the exposed breach section and in the embankment remnants immediately adjacent to each side of the breach section. Interviews were held with eye witnesses to the failure development. Discussions were held with geologists and engineers associated with the dike design, construction, performance monitoring, and remedial repairs.

3.1 Approach to the Investigations

In order to determine the mechanism of failure and the influence of the design and remedial work on the failure, the Team reviewed the geologic conditions of the foundation, the design assumptions and considerations, the excavated foundation conditions, and the dam construction. The Team also examined the external and internal response of the dam and foundation to reservoir filling, the nature of the seepage episodes during operation, and the results of the remedial measures. The Team then carefully considered the details of the failure development. These reviews are reported herein factually, without detailed comment on their relationship to failure.

After considering the available data, the Team postulated potential modes of failure and carried out explorations and inquiries to determine whether evidence could be developed to lend credence to a specific failure mode. Specific findings are provided in this report. Based on the post-failure site observations and the specific findings, the Team established a most likely failure mechanism. Further, the Team was able to identify what influence they believed design assumptions, design considerations, design modifications during construction, and remedial measures had on permitting the failure and on the timing of the failure. Finally, the Team considered what lessons in dam siting, design, performance monitoring and evaluation, and remedial repair were reinforced by the failure.

4. REVIEW OF GEOLOGIC CONDITIONS

4.1 Areal Geology

Quail Creek Dike is located in a somewhat unique geologic setting. The rock formations of the area are sedimentary, ranging in age from Triassic to Recent. The dominant structural feature of the area is the Virgin Anticline which trends northeast-southwest. The anticline is an eroded, doubly plunging dome which exposes beds of the Triassic Moenkopi Formation in the area of the dike. The dike occupies the southeast flank of the anticline with the crest of the anticline being a short distance northwest of the upper right abutment.

4.2 Seismicity

Quail Creek Dam and Dike are situated in a moderately active area of the Intermountain Seismic Belt. The Washington and Hurricane faults, located about 6 miles from the dike, are the closest known regional structures. It has been reported that 15 minor earthquakes occurred along the Hurricane fault between 1888 and 1964. Between 1850 and 1986, four earthquakes with estimated magnitudes (M) greater than 5.5 occurred within about 50 miles (80 kilometers) of the site. The closest of this group to the dike was an M6.0 event in 1902 at a distance of about 18 miles (30 kilometers).

The Final Design Report (1)[2] indicates that seismic stability of the dam was checked by pseudostatic analyses of the upstream and downstream slopes using an average seismic coefficient of about 0.13. The factor of safety for the earthquake condition was computed to be slightly greater than unity for the downstream slope. Since the dam and dike embankment sections were considered to be similar, no separate seismic analysis was performed for the dike.

No evidence has been presented to the Independent Review Team to suggest that earthquakes and resulting seismic effects played a role in the failure of Quail Creek Dike.

4.3 Geology of the Dike Foundation

Sedimentary beds of the Moenkopi Formation are exposed by erosion of the anticline core. The dike was constructed in a geologic setting where arching of thin-bedded sediments, minor faulting and folding, and pervasive jointing of beds resulting from the anticlinal building forces could be anticipated. The thin-bedded sediments at the site were deposited in a tidal flat environment which allowed for deposition and formation of limestone-dolomite and gypsum beds and other salt-rich sediments. The combination of the structural effects caused by arching of the anticline and the presence of significant amounts of soluble

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[2]Numbers in parenthesis refer to items in the list of provided materials in Appendix A.

minerals presented a complex and challenging foundation on which to build the dike.

4.3.1 Stratigraphy: The dike was constructed entirely on beds of the Shnabkaib Member of Triassic-age Moenkopi Formation. While most of the foundation was concealed beneath colluvial and alluvial sediments in the valley, a buff-colored sandstone was exposed on the upper southeast (left) abutment and could be mapped as a separate bed. The buff sandstone was important because of its highly jointed and porous nature. Grouting of the sandstone bed was a design consideration for the dike foundation.

The remainder of the concealed Shnabkaib Member beds, which constituted the major portion of the foundation for the 1,980-foot-long dike, was described in the literature as alternating thin beds of gypsiferous siltstones, gypsum, and dolomite. As previously noted, the dike was constructed on the eastern limb of the anticline where beds were mapped to have a northwesterly strike with dips of 5° to 25° (locally up to 45°) to the northeast. With reference to the dike, this orientation resulted in an upstream-downstream strike of the beds with a gentle dip toward the left abutment (see Photograph 15). Varying competence of the rock foundation, which ranged from soft and easily erodible gypsiferous beds to resistant dolomitic beds, resulted in formation of many small hogbacks (elongated ridges) aligned with the strike of the beds and perpendicular to the dam axis. Although covered with soil, these hogbacks are a geomorphic form easily recognizable to an experienced geologist.

4.3.2 Faulting: During geologic mapping for design of the project, several small faults with little displacement were noted in the area of study. No faults were mapped through the dike foundation. Faults of the type mapped were suspected to be old features resulting from anticlinal-forming forces. Such faults would normally be expected in a long excavation, such as for the dike cutoff, but their exact location and frequency would be difficult to predict prior to exposing the rock formations. Properly treated, small faults of this nature do not affect the integrity of the foundation. Several such faults were recorded during geologic mapping of the core trench excavation.

There is no evidence that faults in the foundation contributed to the dike failure.

4.3.3 Jointing: The fact that the Moenkopi sediments are heavily jointed is clearly visible in the ledge-forming rocks above the dike. The Design Geology Report (1) pointed out the fractured nature of the rock and identified two prominent sets, N5°W to N5°E with a near-vertical dip, and N70-85°W also with near-vertical dips. A few joints were observed to have wide openings. The report indicated that the majority were observed to be tight and showed very little indication of water movement. The report concluded that the fractures would pose little problem at the dike site and in the reservoir area.

4.3.4 Gypsum: The type locality for the Shnabkaib Member in the vicinity of St. George contains a sequence of bedding that is reported as 65 percent siltstone, 25 percent gypsum, and 10 percent limestone and dolomite. The Design Geology Report (1) also indicated the presence of gypsiferous siltstones with some beds of thin-bedded to laminated gypsum. In a discussion regarding the reservoir, the report stated that the gypsum, except for the surface zone, exists in thin lenses, and since its solubility characteristic is low, it should present small, if any, problems. The report recommended that where thick beds of gypsum are encountered in foundations, they should be removed.

4.4 Design Investigations

During reconnaissance and design phases for Quail Creek Dam and Dike, it appears that there was minimum involvement of an experienced engineering geologist during subsurface explorations and in interpretation of the geologic conditions as they would affect the project. Included in the Final Design Report (1) is a geologic map of the project area and a short chapter on geology for the dam, dike, and reservoir. It appears that the consultant geologist's involvement was limited to review of data already gathered reporting basic areal geology and drawing broad conclusions therefrom. Subsurface explorations were apparently planned, conducted, and interpreted by others.

4.4.1 Borings: Seven test borings were initially drilled along the dike centerline during reconnaissance phases of investigation. Better topographic maps, obtained after the initial boring program, indicated that the dam would be more ideally positioned if the right abutment was moved about 400 feet upstream. The centerline was therefore realigned keeping the left abutment approximately in its original position. Three additional borings were added to complete the subsurface exploration for the new dike alignment. In all, 10 borings were drilled for subsurface exploration of the 1,980-foot-long dike (see Figure 5-1).

All of the borings were drilled vertically, thus the vertical joints were not explored. The logs presented descriptions of the rock, percent of gypsum, percent core recovery, percent rock quality designation (RQD) and results of water pressure tests. In general, core recovery was reduced in the upper 10 to 20 feet where weathered beds resulted in little or no core recovery through some intervals. Below about 20 feet, core recovery was generally in the 90 to 100 percent range. On the other hand, RQD values were very low. It was learned that the core was not measured and logged until some time after drilling; this may have adversely impacted the RQD values.

4.4.2 Water Pressure Tests: Water pressure tests were conducted, generally at 10-foot intervals, in all of the borings. Results of tests were expressed as permeability coefficient in feet per year (ft/yr) and much discussion in the Final Design Report (1) concerned permeability of the foundation. It must be understood that conversion of water pressure test data, as was done for Quail Creek Dike, does not express true permeability of a jointed, bedded rock media. The conversion is only one of several methods used to normalize the test data into a convenient form for comparison of results. The values computed more correctly represent the hydraulic conductivity of the intervals that were pressure tested. The results are controlled by the basic permeability of the rock and interconnected open bedding planes and joints. In foundation rocks of the type at the dike, tests that indicate little or no water loss (expressed in ft/yr) indicate only that the interval of hole tested did not penetrate solution channels, open bedding planes and/or joints with high hydraulic conductivity. It is possible that a hole drilled a few feet away or at an angle from vertical would give quite different results.

Boring D.H. 1-D penetrated the buff sandstone, and water pressure tests indicated a high hydraulic conductivity in that unit. All other borings in the dike foundation penetrated the gypsiferous rich siltstone beds stratigraphically below the buff sandstone. Water pressure tests generally indicated that the hydraulic conductivity was high in the upper 10 feet of foundation, while it was low deeper in the foundation. The exception to this pattern was hole D.H. 9-D which indicated very high hydraulic conductivity to a depth of 27 feet.

The near-vertical joints known to exist in the dike foundation would not likely be penetrated by vertical borings. Therefore, one of the major structural features of the rock mass, which could be a major factor in hydraulic conductivity of the foundation, was not explored. Hole D.H. 9-D could have penetrated a near vertical joint in the upper part of the hole.

4.5 Characterization of Expected Foundation Conditions

Decisions on the extent of foundation excavation and treatment required at the dike were heavily weighted by the permeability coefficients computed from water pressure tests in the borings.

Theoretical studies (1) were made regarding solutioning of gypsum. It was concluded that the likelihood of solutioning of gypsum lenses causing serious seepage problems was relatively small.

In recognition of the relatively high hydraulic conductivity of the buff sandstone, a triple-line grout curtain was designed and installed between Sta.s 0+00 and 2+00. Based on the computed coefficient of permeability for the remaining rock foundation, it was concluded that grouting with cement grout would not be effective below a depth of about 10 feet. Hence, about 1,700 feet of the dike foundation was not provided with a grout curtain.

4.6 Post-Failure Exposures of the Foundation

Water cascading and sweeping through the dike eroded and cleaned the foundation rock for a short distance upstream, through the dike section, and downstream for some distance. This cleaning action has allowed a thorough study of the foundation rock. The investigations conducted for the Independent Review Team were solely for the purpose of determining the cause and mode of failure.

4.6.1 Rock Foundations: Geologic mapping of the exposed rock consisted of identifying 11 units in the bedding sequence that could be distinctly identified based on variable rock type and physical characteristics (see

Figure 4-1 and Photographs 17-32). Close visual examination of the various rocks resulted in a determination that gypsiferous silty dolomite was a predominant rock type. A stratigraphic column with detailed description of the rock units is presented on Figure 4-2. In order to confirm the field identification of rock units and joint-filling materials, a petrographic study was conducted. X-ray diffraction tests were used to supplement the microscopic examinations where necessary. Results indicate that many of the rocks previously called siltstone and silty dolomite are actually a fine-grained dolomite, more appropriately called dolomicrite. Joint-filling materials have generally been identified as gypsum, dolomite, and quartz mixed with clay minerals consisting of illite, kaolinite, and smectite, all listed in decreasing percentage of presence. A report on the petrographic studies is in Appendix C.

A significant observation in the weathered rock zone was the presence of many expanded bedding planes. The bedding expansion is more pronounced where gypsum is present in the form of small filaments or pillars supporting adjacent beds (see Photographs 6, 32, and 32B). The gaps between beds are as much as 1-inch wide. With water flowing through a gap, solutioning of gypsum would be enhanced resulting in a continually enlarging conduit for water to pass through. Collapse of some expanded beds under embankment and reservoir loads probably occurred. Expanded bedding was particularly noted in Units 4, 6, and 7. Expansion is a function of thickness of gypsum laminae and was observed in all units except 5.

4.6.2 Jointing: One structural feature of the foundation clearly exposed by the water action was the pervasiveness of the joint system in the rock. As previously noted, joint sets were identified during design studies, but could not be directly observed in foundation rocks because of soil cover. It should be noted, however, that the existence of principal joint sets was identified prior to embankment placement in the cutoff excavation by geologists from the Utah State Division of Water Resources, who prepared a geologic map of the cutoff trench on December 16 and 23, 1983 (Appendix E). Field notes accompanying the geologic map comment on the open condition of some of the joints.

Post-flood exposures show the rock units to be differentially jointed with more competent units (2, 4, and 5) showing stronger joint patterns. Many joints in the vicinity of the dike were measured, and 132 measurements were plotted on a contoured stereo net to demonstrate the range of jointing. This plot shows the principal sets to average about N22°E, N24°W, and N81°W, all with near-vertical dips. The stereo net is shown on Figure 4-5.

Important observations of the joint system included the strong upstream-downstream trend of one of the sets, the close spacing of some joints, openings up to about 1 inch along some joints, the nature of filling materials, and evidence of erosion and solutioning of gypsum-rich filling materials (see Photographs 17-23). Solution channels can also be seen in conjunction with joints and bedding planes in the fresh bedrock. These channels, formed by ground-water flow, are likely old geological features, typical of limestone, dolomite, and gypsum-rich rocks. Although not actively enlarging under present natural conditions, these channels provided conduits for passage of reservoir water to areas of erodible and/or soluble gypsum-rich joint fillings and expanded bedding planes.

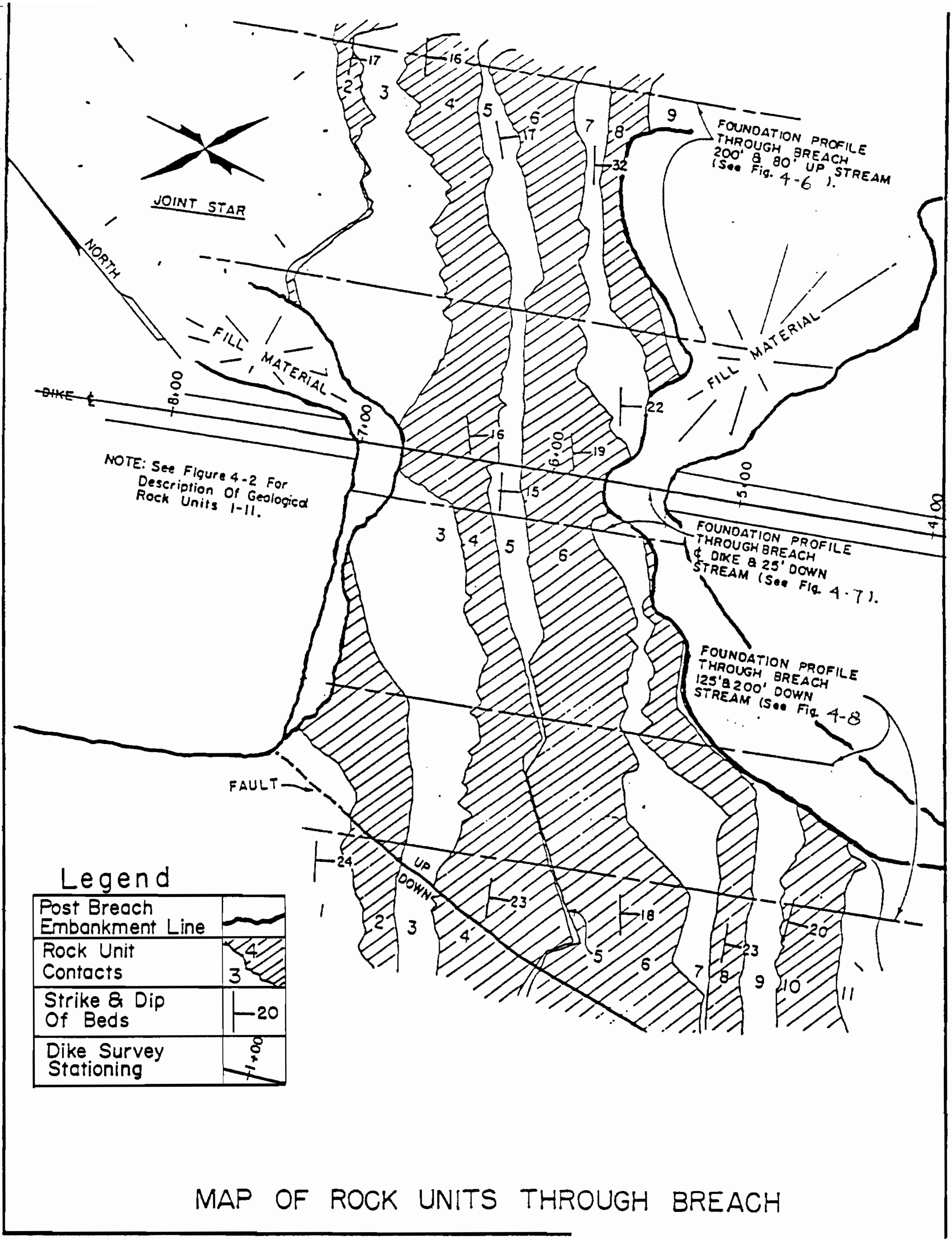
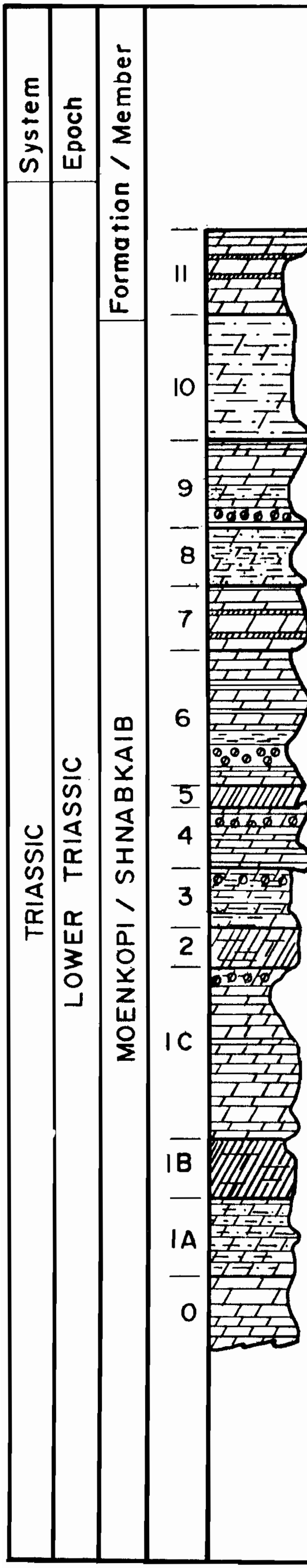


Figure 4-1

Map of Rock Units through Breach

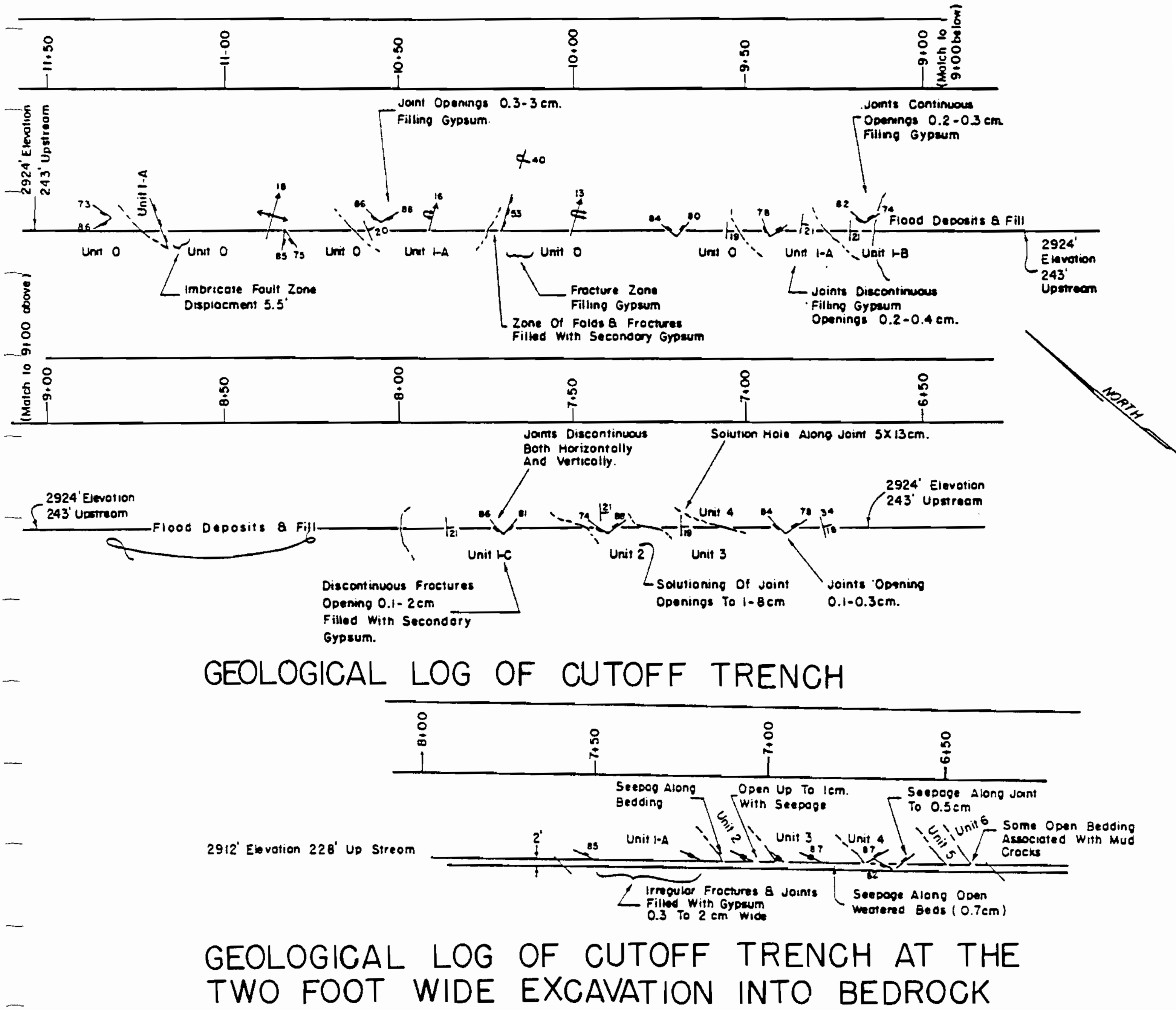
STRATIGRAPHIC SECTION OF EXPOSED ROCK IN QUAIL CREEK DIKE BREACH



		FEET
11	GYPSUM and GYPSIFEROUS DOLOMICRITE and SILTY DOLOMICRITE: Pale olive, yellowish gray, pale red and grayish red; very gypsiferous, secondary gypsum present in fractures subparallel to and across bedding; thinly bedded to laminated; open beds at weathered surface.	18′
10	DOLOMITIC SILTSTONE: Grayish red, pale red; very gypsiferous; frequent laminae of gypsum roughly parallel to bedding; fractures with secondary gypsum; weathering produces open bedding with gypsum pillars.	16′
9	SILTY DOLOMICRITE: Mostly pale olive to grayish red; very gypsiferous; micaceous; lower 1.1 ft. is light olive gray bed overlain by 2-3 inch layer of gypsum with slickenside surfaces, followed by a 1.6 ft. "brick red" bed; frequent white and clear gypsum laminae which expands when weathered creating open beds with gypsum pillars.	11.2′
8	DOLOMITIC SILTSTONE and SILTY DOLOMICRITE: Mostly grayish red and pale red, some yellow gray beds in unit; wavy bedding; nodular gypsum zone similar to unit 6; joint continuity and density relatively low; resistant gray bed at top forms hogback; weathered beds often open with gypsum pillars.	7.4′
7	DOLOMICRITE and GYPSUM: Light olive gray to yellowish gray; silty; white and clear gypsum laminae up to 0.6 cm thick; brittle unit with higher joint density; weathered surface has open beds supported by pillars or filaments of gypsum; joints often open due to weathering.	8.2′
6	DOLOMICRITE: Mostly grayish red with yellowish gray beds; interlaminated with siltstone and clear gypsum; several feet of curled mud cracks at base; 1.5 foot zone of nodular gypsum; thinly laminated strata present, but mostly laminated to very thinly bedded; resistant unit which forms large hogback ridge; density of continuous joints is low, but major joints extend through unit.	16′
5	GYPSUM: Pale olive and light olive gray; thinly laminated with some laminae of clear gypsum; silty; brittle unit with relatively high density of joints which are often differentially open due to weathering and/or solutioning; less resistant, weathers to sugary texture.	2.7′
4	DOLOMICRITE: Pale olive to light olive gray; very gypsiferous, gypsum usually occurs as thick laminae; nodular zones and diapiric contortions of bedding are present; ranges from laminated to very thinly bedded; unit is brittle with laterally continuous joints; weathered surfaces consist of extensively expanded gypsum pillars with fractured beds of dolomite.	7.3′
3	SILTSTONE and DOLOMICRITE: Color is pale red, grayish olive to yellow gray; mixed rock; very gypsiferous, gypsum occurs mostly as laminae and cross laminae; silty; less brittle unit, joints are relatively infrequent.	8′
2	GYPSUM and DOLOMICRITE: Pale olive and yellowish gray; laminae of clear gypsum frequent; thin laminae of siltstone; contorted bedding; micaceous; resistant unit, weathered surfaces are expanded, yellow gray granular gypsum.	4.9′
1C	DOLOMICRITE: Pale olive and grayish red; thinly laminated to very thin-bedded; frequent gypsum laminae along and across bedding; thin laminae of micaceous siltstone present; weathers to expanded beds of gypsum pillars and fissle "shaley" surface.	23.5′
1B	DOLOMITIC GYPSUM: Very light olive gray, grades to light reddish pink; laminated; frequent laminae of white and clear gypsum; jointing density is relatively high and joints are opened due to solutioning and/or weathering.	7.1′
1A	SILTY DOLOMICRITE: Light reddish brown to grayish green; very gypsiferous; laminated to thinly laminated with frequent laminae of clear and white gypsum along and crossing bedding; weathers to open beds with gypsum pillars; density of continuous joints is relatively low.	10′
0	GYPSIFEROUS DOLOMICRITE: Light olive gray to light gray brown; thinly laminated with laminae of clear gypsum frequent; contorted bedding, jointing density is relatively low.	9′
	Totol	149.3′

Figure 4-2

Measured Stratigraphic Section of Dike Foundation



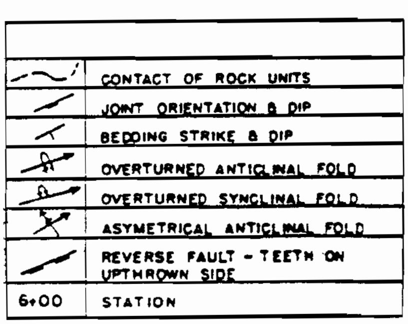


Figure 4-3
Geologic Log of Upstream
Cutoff Trench

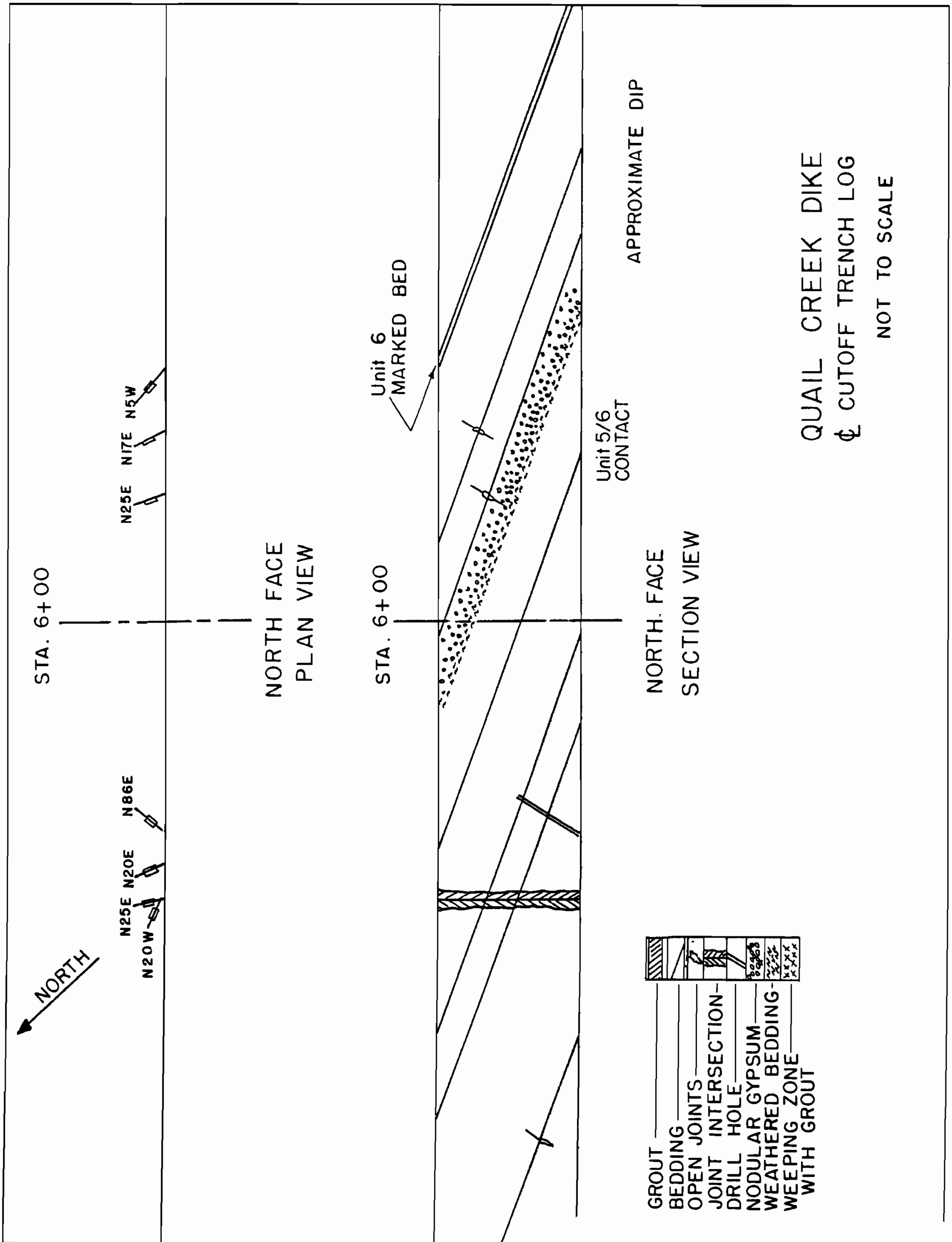
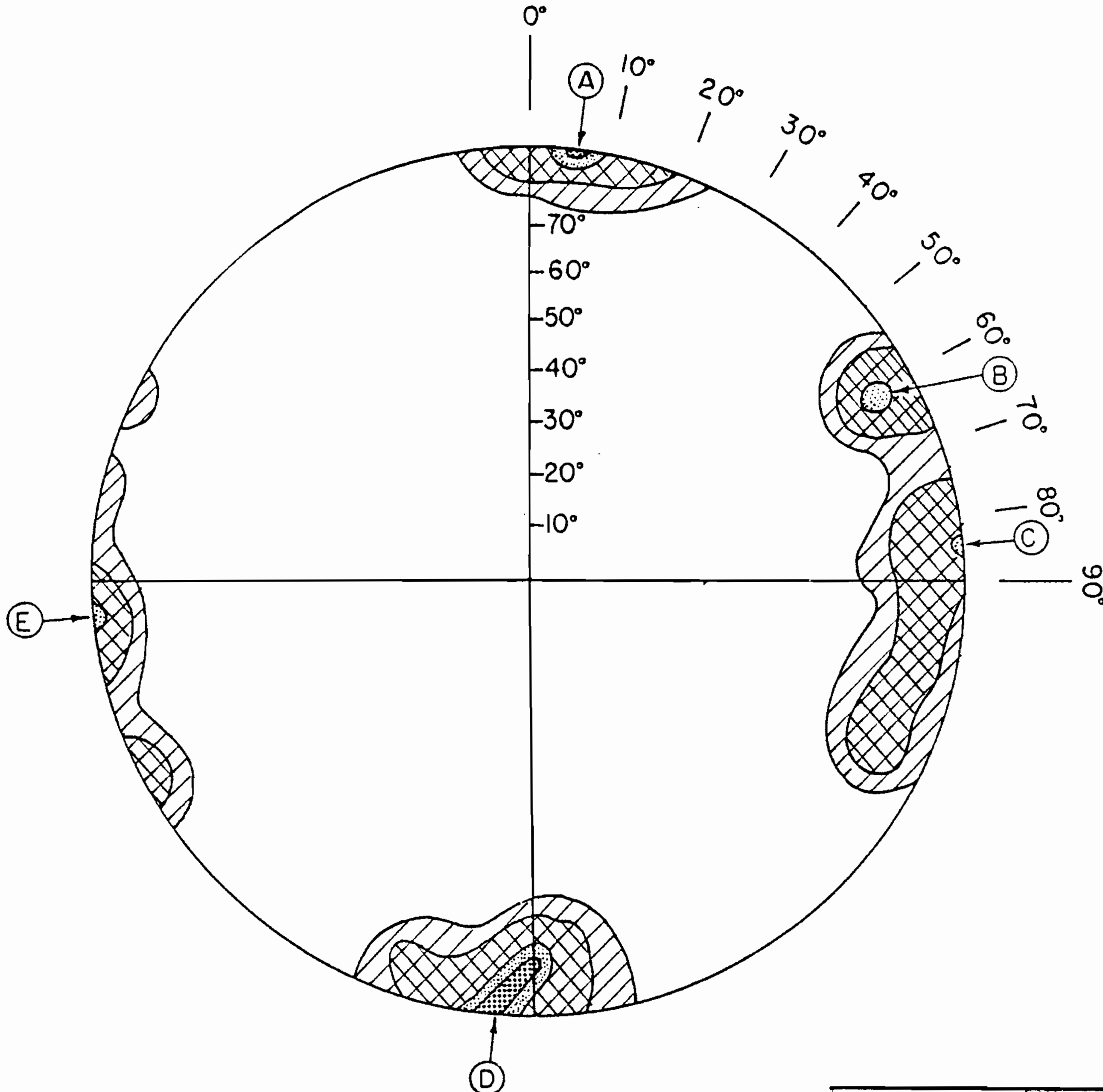


Figure 4-4

Geologic Log of Centerline
Cutoff Trench



CONTOUR OF JOINT POLES PLOTTED ON LOWER HEMISPHERE OF SCHMIDT STEREONET.

13 % or More	
10 % to 13 %	
5 % to 10 %	
1% to 5%	

	STRIKE	DIP
A	N80W	89SW
B	N23W	80SW
C	N1E	89W
D	N82W	86N
E	N1W	89E

Figure 4-5

Contour of Joint Poles

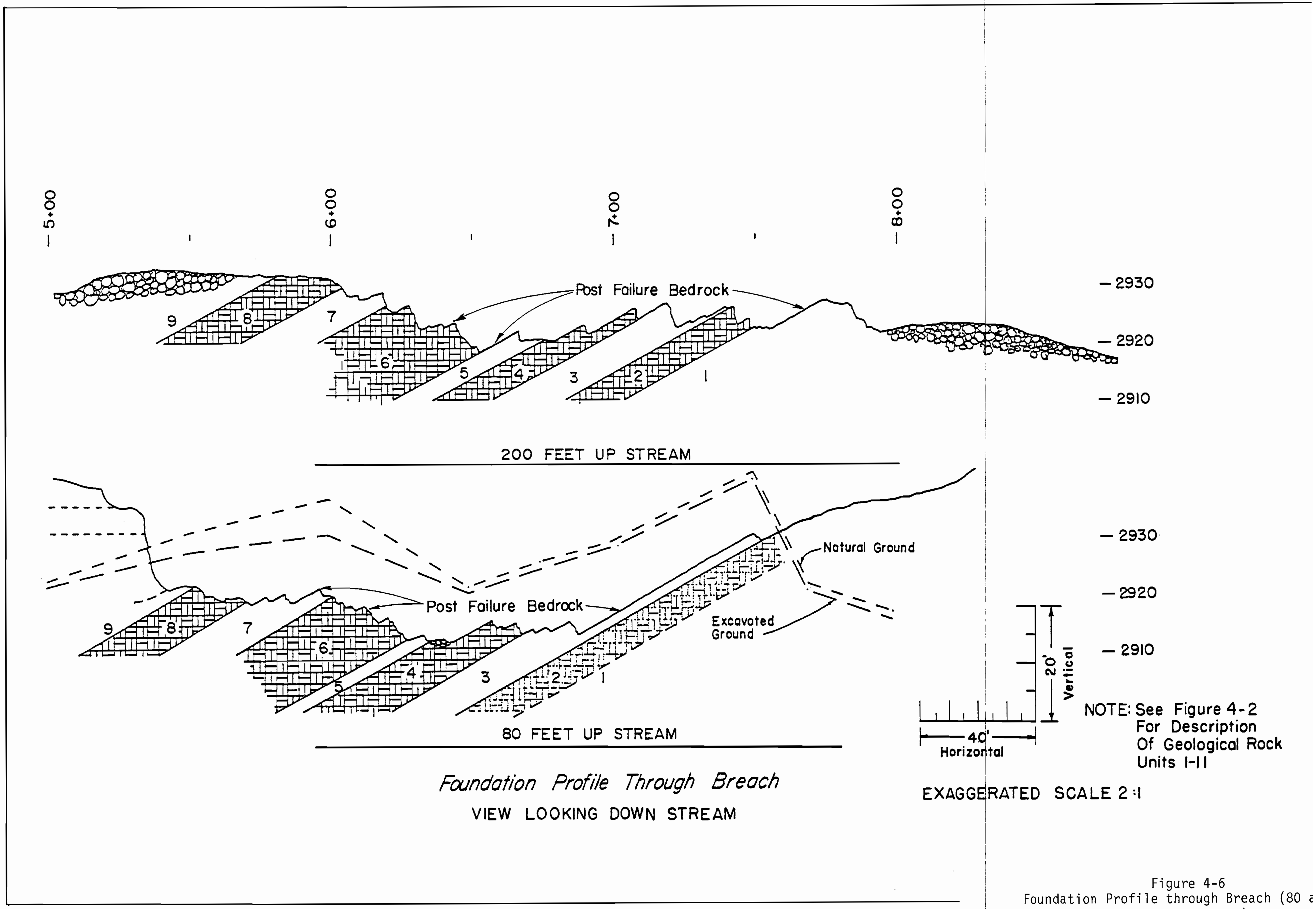


Figure 4-6
Foundation Profile through Breach (80 a
200 ft. upstream)

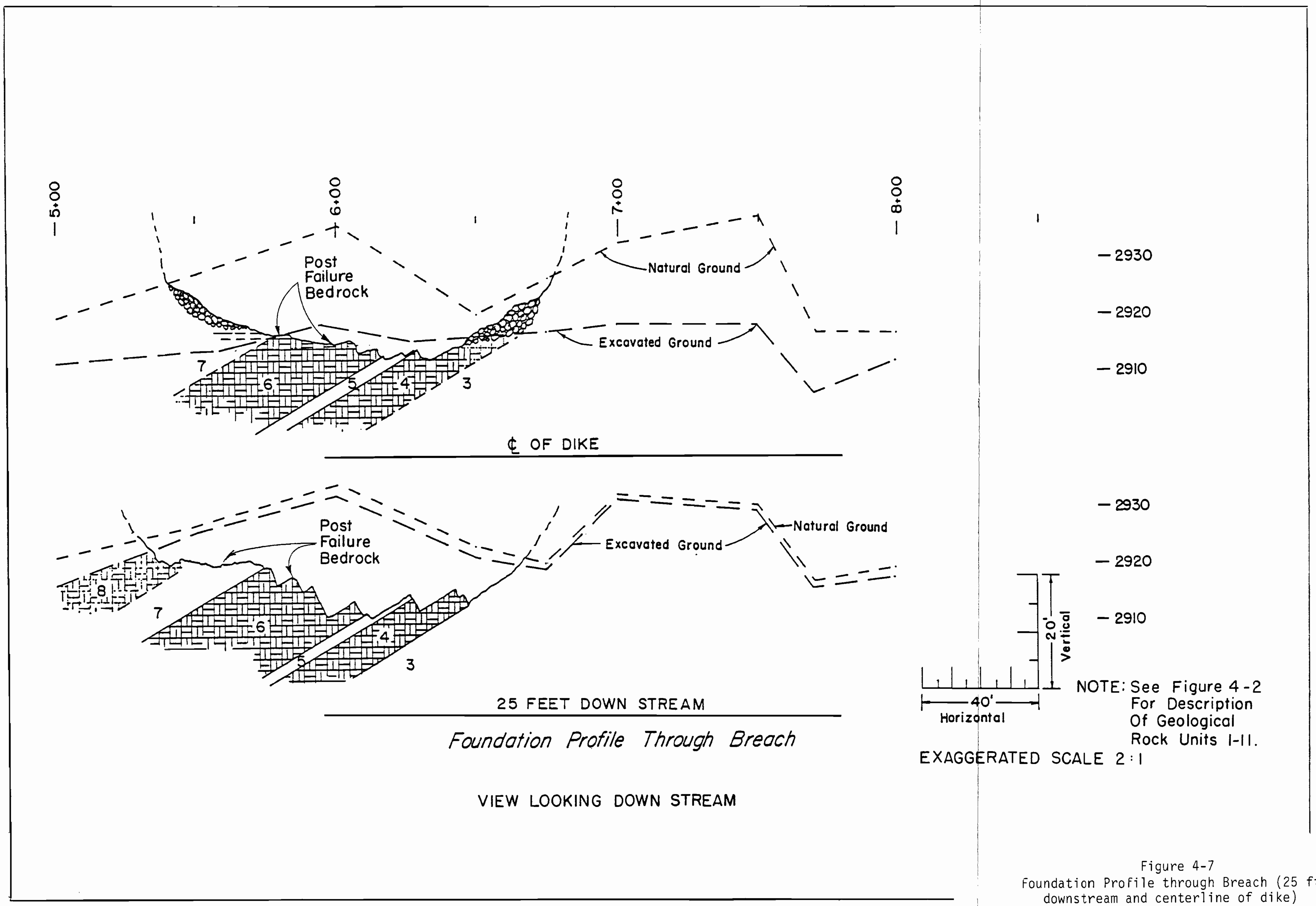


Figure 4-7
Foundation Profile through Breach (25 ft. downstream and centerline of dike)

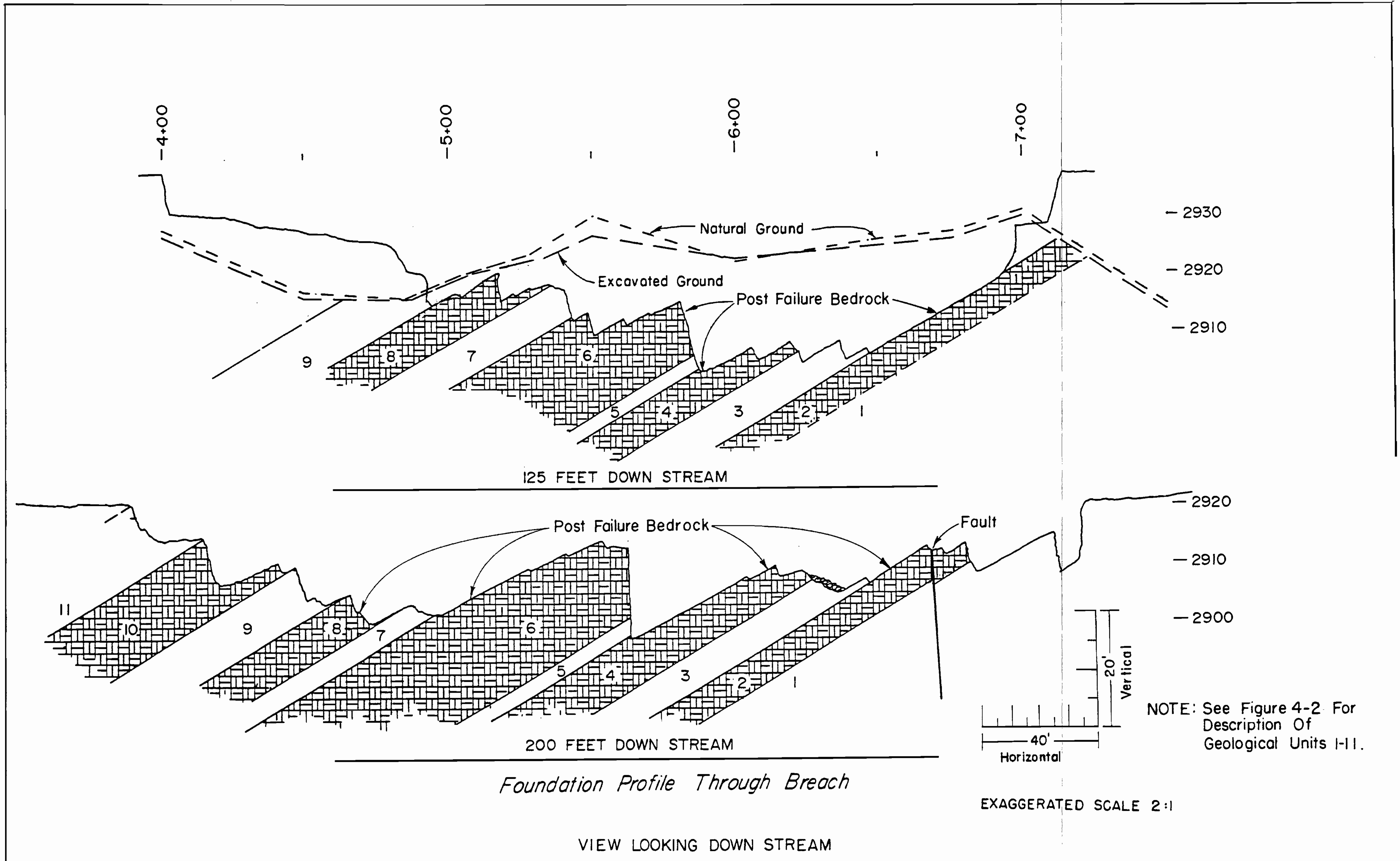


Figure 4-8
Foundation Profile through Breach
(125 and 200 ft. downstream)

5. DESIGN CONSIDERATIONS

Quail Creek Dam and Dike were designed by the firm of Rollins, Brown, and Gunnell, Inc. Design began in 1982 and was submitted to the WCWCD in a Final Design Report dated August 1983. Construction Contract Documents were from the same firm and were dated September 1983.

5.1 Foundation Considerations

Exploration for the dike is shown in plan and profile on Figure 5-1. The initial exploration consisted of seven vertical holes. The alignment was then shifted upstream to reduce fill volume, and three additional vertical holes were drilled. Based on water pressure tests from the vertical exploratory drilling, it was concluded that rock below a depth of about 10 feet would be relatively impervious and permit very little through-seepage. As a result of this conclusion and the lower head on the dike as compared to the dam, the following design decisions were reached:

A. A cutoff trench to an average depth of 10 feet through the upper weathered rock would be adequate to control foundation seepage.

B. Grouting would not be effective or necessary in the dike foundation except for the sandstone in the upper left abutment from about Sta. 0+00 to Sta. 2+00.

C. Special treatment (i.e., dental concrete, slush grouting, etc.) was not required for rock exposed in the cutoff trench except for the sandstone high in the left abutment.

In addition, it was concluded that stripping over the foundation to a depth of 1 foot would be adequate and that the very irregular stripped surface should be prepared by placing Zone I fill in all potholes, undulations, and drainage channels prior to beginning the zoned dike fill. No mention is made of the potential for piping or erosion along the contact between rock and overburden left in place or Zone I placed on the stripped foundation. The undulations and drainages which reflect the erodibility of the bedrock sandstone run perpendicular to the dike axis. Consequently, at several locations, substantial thicknesses of Zone I were placed continuously from toe to toe.

5.2 Materials

Materials available for construction of a zoned dike section (as shown on Figure 2-4) included silty and clayey sands for use in Zone I, a limited volume of medium plasticity clay (weathered shale) for Zone II requiring a haul distance of about 3 miles, large volumes of relatively free-draining sandy gravel for use in Zone III, and processed sand for filter construction. Refer to Appendix B for descriptions of materials, borrow areas, and testing.

The low-plasticity silty and clayey sands also had a high percentage of soluble salts in many samples. Thus, there was concern for both the imperviousness and potential leaching of salts if these materials were used in zone 1. It was concluded that if materials placed in Zone I had at least 15 percent by weight passing the No. 200 sieve and were adequately compacted, the Zone I permeability would be very low and seepage and dissolution of soluble salts would not be a problem. Permeability data are provided in Appendix B.

The medium-plasticity clay or weathered shale was considered to be highly impervious and thus was used in a Zone II upstream of Zone I to increase the safety against seepage and dissolution of soluble salts in Zone I. Permeability data are provided in Appendix B. A shallow thickness of Zone II was also placed on the bottom of the cutoff trench to reduce seepage. Such material would ordinarily be resistant to erosion along the contact. However, this material examined in place tended to be quite friable because of a high percentage of discrete shale particles.

5.3 Filters and Seepage

Internal drainage was provided by a vertical filter zone just downstream of Zone I based on the design conclusion that the low-plasticity Zone I material must be protected by a satisfactory filter. At the main dam, a filter zone was placed between Zone I and the downstream slope of the cutoff; this was not done at the dike. Based on testing of bulk samples of the sandy gravels available for Zone III, it was concluded that if the materials were placed with less than 5 percent passing the No. 200 sieve, the material would be free draining. Thus, the Zone III upstream shell would protect against drawdown effects, and downstream the Zone III connected to the vertical filter would preclude saturation of the downstream embankment. It was also concluded that processed filter would satisfy filter requirements between Zone I and Zone III materials. There was no mention of filter requirements between overburden on the stripped foundation or Zone I placed to prepare the foundation and the underlying rock. Seepage analyses and studies were performed for the main dam. As a result, the foundation was grouted, a line of drain holes was drilled just downstream of the grout curtain to intercept and dispose of foundation seepage, exposed rock contacts were treated with dental concrete and slush grouting, and the upstream abutments and foundation were blanketed with Zone I and Zone II materials. No formal seepage analyses were performed for the dike since it was concluded that by cutting off the upper 10 feet of the dike foundation there would be no seepage of consequence through either the dike or its foundation.

5.4 Stability and Settlement

Stability analyses were performed for the dike for the steady-state seepage condition downstream and the sudden drawdown condition upstream (1). A computer program using Spencer's method of limit equilibrium was used to make the analyses using shear strength parameters from consolidated drained direct shear and triaxial shear tests on Zone I, Zone II, and Zone III materials (Appendix B). Shear strengths for processed filter, Zone IV, and foundation material were estimated. The shear strengths shown appear reasonable, and the factors of safety of 1.5 for steady-state seepage downstream and 1.3 for sudden drawdown upstream are commonly accepted as adequate. Seismic stability was discussed, and the main dam was analyzed for horizontal deformation and pseudostatic stability. The dike was not analyzed. If the dike was constructed of well-compacted materials as intended, and since there was little evidence of liquefiable overburden in the foundation, it was concluded that the dike should safely sustain moderately severe seismic activity.

The dike embankment and foundation were not analyzed for settlement. However, up to 0.5 foot of camber was added to the crest elevation to allow for post-construction settlement. Some consideration was given to the potential for expansion of Zone I material resulting from expanding lattice-type clays or growth of gypsum crystals resulting from soluble salts in the soil reacting with water. It was concluded that neither was likely to occur.

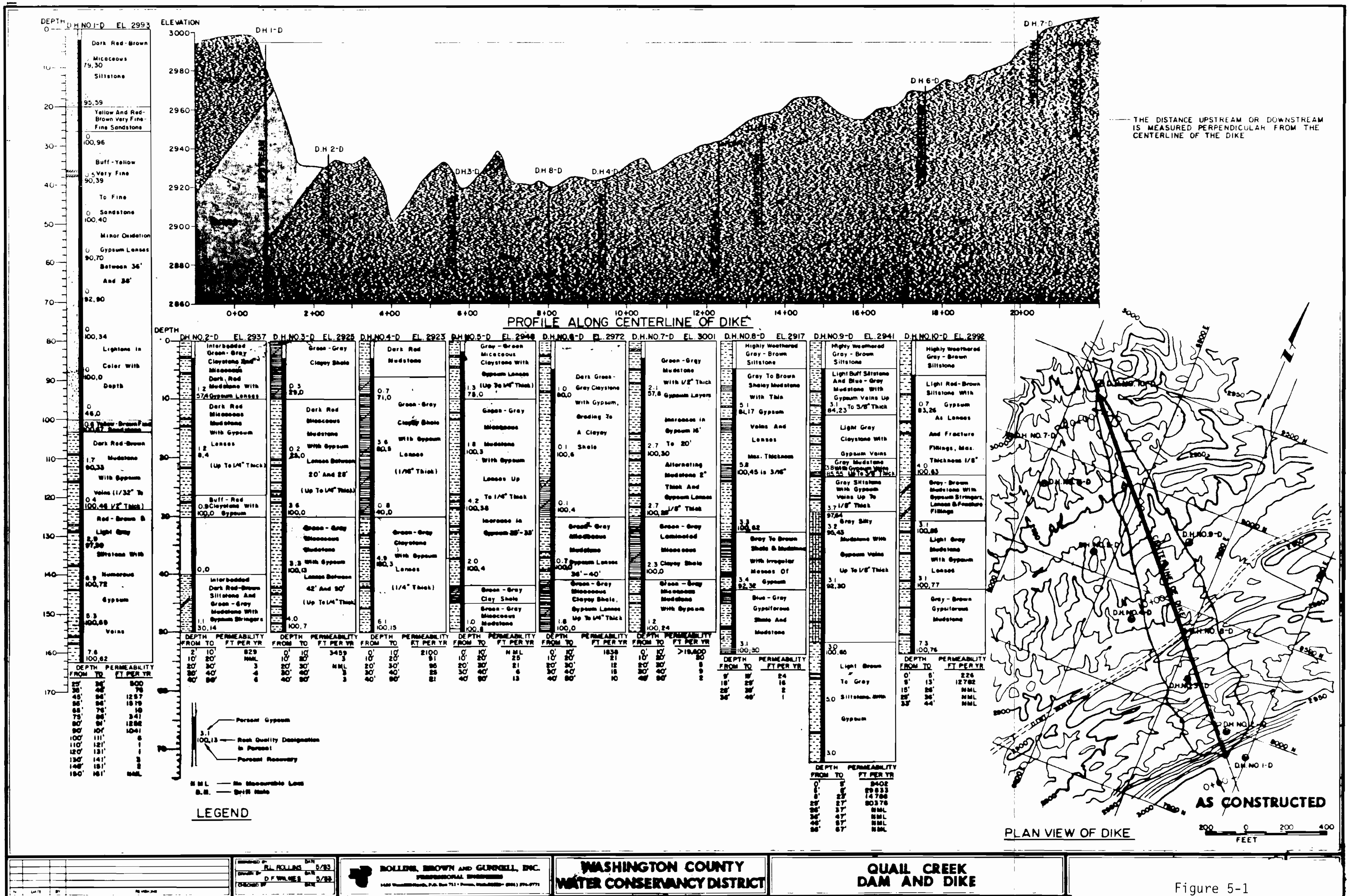


Figure 5-1
Exploration for Quail Creek Dike

6. REVIEW OF FOUNDATION PREPARATION AND DIKE CONSTRUCTION

6.1 Foundation Preparation

The Final Design Report (1), plans and specifications for construction (3), construction logs (26), photographs, and diaries (27) were reviewed. The foundation preparation was designed to consist of a centerline cutoff trench excavated to a variable depth (estimated at 10 feet overall), stripping of the upper 1 foot of existing surface materials beneath the upstream and downstream shells, and grouting on the left abutment cutoff trench. There is no indication of substantial deviation from this procedure. The specified section is presented on Figure 2-4. A review of the final cross sections and construction photographs indicated that the cutoff trench varied from 2 to 23 feet deep. The width varied, with the narrowest portion being 19 feet. The thickness and character of overburden remaining under the shells were not determined. The trench depth was determined in the field and reported to be "based on the ability of the excavation equipment" and field judgment of "sound rock." Final cross sections and photographs (see Photographs 2-6) indicate that the bottom of the trench was cleaned by brooming and air hosing. A grout curtain was installed on the left abutment. From plans and specifications, photographs, and post-failure investigation trenches, it appears that no efforts were directed to treatment of the trench walls. A geologic report of mapping the trench is presented in Appendix E. The layer of Zone II material required to be placed over the excavated trench bottom was found in place, and the thickness appeared to be well over 1 foot and perhaps up to 2 feet in places. No dental concrete was placed, and it was reported that observed rock fractures were "tight or filled." The geologic map indicates open fractures on the trench walls, as does Photograph 6 taken in 1983. It was reported that all rock surfaces were reduced from vertical or overhangs to a positive slope along the trench bottom.

The foundation beyond Sta. 2+00 was treated in accordance with the following specifications (3).

"A. Foundation area to be cleared and grubbed and stripped with equipment working perpendicular to axis to a depth of 1 foot.

B. Gypsum deposits to be excavated and removed from foundation area. These deposits exist in areas approximately as shown in Plan View and are 5 to 10 feet thick.

C. Cutoff trench to be excavated to a depth of approximately 10 feet below the centerline elevation or as directed by the Engineer.

D. Base of cutoff to be backfilled with a minimum of 1 foot of Zone II material within 48 hours after excavation.

E. Foundation area to be scarified to a depth of 10 inches, moisture conditioned, and redensified.

F. All undulations, channels, and holes to be backfilled, with Zone I material following stripping to provide an even surface prior to beginning embankment fill."

Based on photographs, logs, and cross sections, the specifications were apparently met. There is no indication that the thickness of overburden was further determined, that densifications beyond those specified above were attempted, or that the character or composition of the overburden was further evaluated. The thickness of Zone I material placed over the overburden ranged up to 20 feet and was continuous across the foundation as seen in photographs (see Phtograph 8) and recorded on the cross section at Sta. 5+00 and others. The foundation overlain by overburden, Zone I, and embankment materials is shown in photographs and cross sections. A typical section is presented in Figure 2-5.

6.2 Dike Construction

The embankment was constructed without noted differences from the section presented in the plans, except for the substantial thicknesses of Zone I placed above the foundation overburden. This produced a perched zoned dam with Zone III material up to 20 feet above the foundation overburden. Daily logs, photographs, and cross sections were reviewed for specification compliance. Required densities were reportedly achieved for compacted materials. A review of Embankment Materials and Properties is presented in Appendix B. As noted in Appendix B, an early problem of materials freezing was noted and corrected. Zone II materials were generally placed well dry of optimum. No overcompaction or contamination of materials was reported or observed by interviewed personnel. The embankment section remaining exposed by post-failure excavation reveals a well-compacted, zoned section. The filter zone, Zone I, and Zone II form well-defined marker beds.

Post-failure testing included in this report (Appendix C) indicates that required densities and gradations were achieved at the test locations.

7. REVIEW OF PERFORMANCE HISTORY OF THE DIKE

7.1 Structure Response

The compacted embankment portion of the dike appears to have performed well over the life of the project. The dike as constructed showed no signs of distress. Crest settlements, spreading of the shells, cracking, or other forms of movement were not noted. No surface sliding or sloughing of the upstream and downstream slopes was noted, even after the extremely sudden drawdown of the pool during failure. Embankment through seepage was not noted, and piezometers did not reflect high readings within the embankment. Considerable discussions and interviews with operational personnel revealed that no embankment through-seepage was observed. No vegetation on the embankment indicative of seepage was noted. Careful observations of the remaining embankment showed no areas of distress caused by settlement, seepage, or sliding. The embankment did exhibit distress caused by the grouting operation as discussed in Section 11 (see Photographs 42-50 for views of the remaining embankment).

7.2 Piezometric Response

Six piezometers were installed in Quail Creek Dike and foundation. The locations of these piezometers are shown in plan and profile on Figure 7-1 and in cross section on Figures 7-2, 7-3, and 7-4. The water level indicated in a properly working piezometer installation represents the highest water pressure that exists within the "isolated interval" or influence zone that is contributing water to the piezometer. The isolated interval for each of the piezometers is discussed below. Piezometers 15, 16, and 18 were installed well within the foundation rock. Piezometers 17 and 19 were installed near the foundation/embankment contact and appear to have at least some portion of the influence zone below the foundation contact. Piezometer 20 is clearly only in the dike.

Figure 7-5 shows a plot of the reservoir operation since October 1985 along with notation of significant events during the operation and a record of seepage flows.

7.2.1 Response of Piezometer 15: Figure 7-6 shows the response of piezometer 15 which is located at Sta. 1+50 on the left abutment and is about 15 feet below the foundation contact at elevation 2950. The first response of the piezometer was after the reservoir had reached elevation 2963 during first filling. The piezometer continued to rise directly with increases in reservoir levels and fall with the lowering of the reservoir over the next 2 months. The piezometric head fell below elevation 2950 when the reservoir lowered below 2963. Upon the next reservoir filling, the piezometer began responding after the reservoir reached elevation 2971 and again rose and fell directly in response to reservoir levels. The piezometer reached its maximum level of 2963.3 when the reservoir was at elevation 2983.3 on January 20, 1988. Subsequent slight lowering and raising of the reservoir (down to El. 2982.5 and up to El. 2985) resulted in a maximum piezometer reading of 2958.4, or approximately a 5-foot drop in piezometric head. Beyond that time, the only other notable response was an anomalous reading on October 10, 1988. Although this low reading was taken during the grouting near Sta. 5+00 due to heavy concentrated seepage downstream, it does not appear to be related since the prior reading (October 4, 1988 also taken during this period of heavy seepage) and the following reading (October 16, 1988, taken after the seep had been grouted) showed nearly equal values. In summary, the readings from piezometer 15 responded with reservoir fluctuation at approximately 60 to 75 percent total head loss. The rapid character of the piezometer response indicates that it was related to foundation seepage rather than flow through the Zone I materials of the dam.

7.2.2 Response of Piezometer 16: Figure 7-7 shows the response of piezometer 16 which is located at Sta. 7+50 (50 feet to the right of the right side of the breach). The piezometer was 37 feet downstream of centerline and was set 29 feet below the foundation contact. The piezometer water level was apparently near elevation 2925 during reservoir filling and began to increase after the reservoir reached elevation 2965. The piezometer then generally rose and fell in response to reservoir filling and lowering. The sparcity of readings does not allow determination of whether or not any lag in response existed. The maximum piezometric level reached was approximately at the dike/foundation contact. During this first filling episode, there were two interesting piezometric responses: First, there was a rapid drop of piezometric head followed by a partial recovery of head during the reservoir lowering. Second, the piezometric head during reservoir lowering dropped well below the head that existed in the piezometer prior to filling. This response indicated that the rock around the piezometer was tight (that is, able to hold water in the piezometer hole prior to filling) and that the reservoir filling resulted in changes in the foundation seepage paths that allowed the piezometric head to drain off. During reservoir refilling, the piezometer rose with the reservoir level, but not to levels comparable with the first filling. While the reservoir was holding relatively steady near elevation 2970 in the summer of 1987, piezometer 16 showed a sharp increase in pressure (between July 6 and July 13, 1987) which subsequently dropped (between September 8 and 16). These changes coincided with pressure grouting occurring near Sta. 6+00 to Sta. 6+50. The piezometric head then gradually increased to elevation 2931.8 in response to reservoir filling to elevation 2983.3 on January 11, 1988. From that point higher heads developed for no change in reservoir elevation (for example, on July 25, 1988, at reservoir elevation 2983.3, the piezometric head was 2955.0, or 23.2 feet higher.) In August 1988, the reservoir was lowered about 10 feet and was then raised 7 feet by the end of December. Piezometer 16 reflected the full 10 feet of reservoir drop, but dropped during the reservoir rise after November 13, 1988, including a drop of 1.1 feet between December 26 and December 30. Piezometer 16 was isolated in foundation material, and its responses clearly reflected seepage within the foundation.

7.2.3 Response of Piezometer 17: Piezometer 17 was located in the same hole as piezometer 16 but was isolated 20 feet higher such that it could be influenced by either foundation or dike piezometric heads (whichever was higher). The response of piezometer 17 is shown on Figure 7-8. During initial reservoir filling, piezometer 17 apparently first began responding after the reservoir reached elevation 2963. Like piezometer 16 the response corresponded well with reservoir levels and also experienced a loss and recovery of piezometric head during reservoir lowering. The maximum head reached in the piezometer during first filling was just above the dike/foundation contact. During refilling, piezometer 17, like piezometer 16, did not initially reach the levels that it had reached during initial filling. In fact, the readings from January 1987 through the end of April 1988 indicate that the water level was remarkably stable below or near the foundation contact, even when the reservoir rose to near the maximum recorded elevation. Between May 25 and June 30, 1988, the water pressure rose 15 feet with no change in reservoir elevation. The pressure began to decrease gradually after that time, dropping 10 feet, at least partially in response to the reservoir lowering. During the reservoir rise of 7 feet between November 13 and December, the pressure continued to decrease including a drop of 1.2 feet between December 26 and December 30.

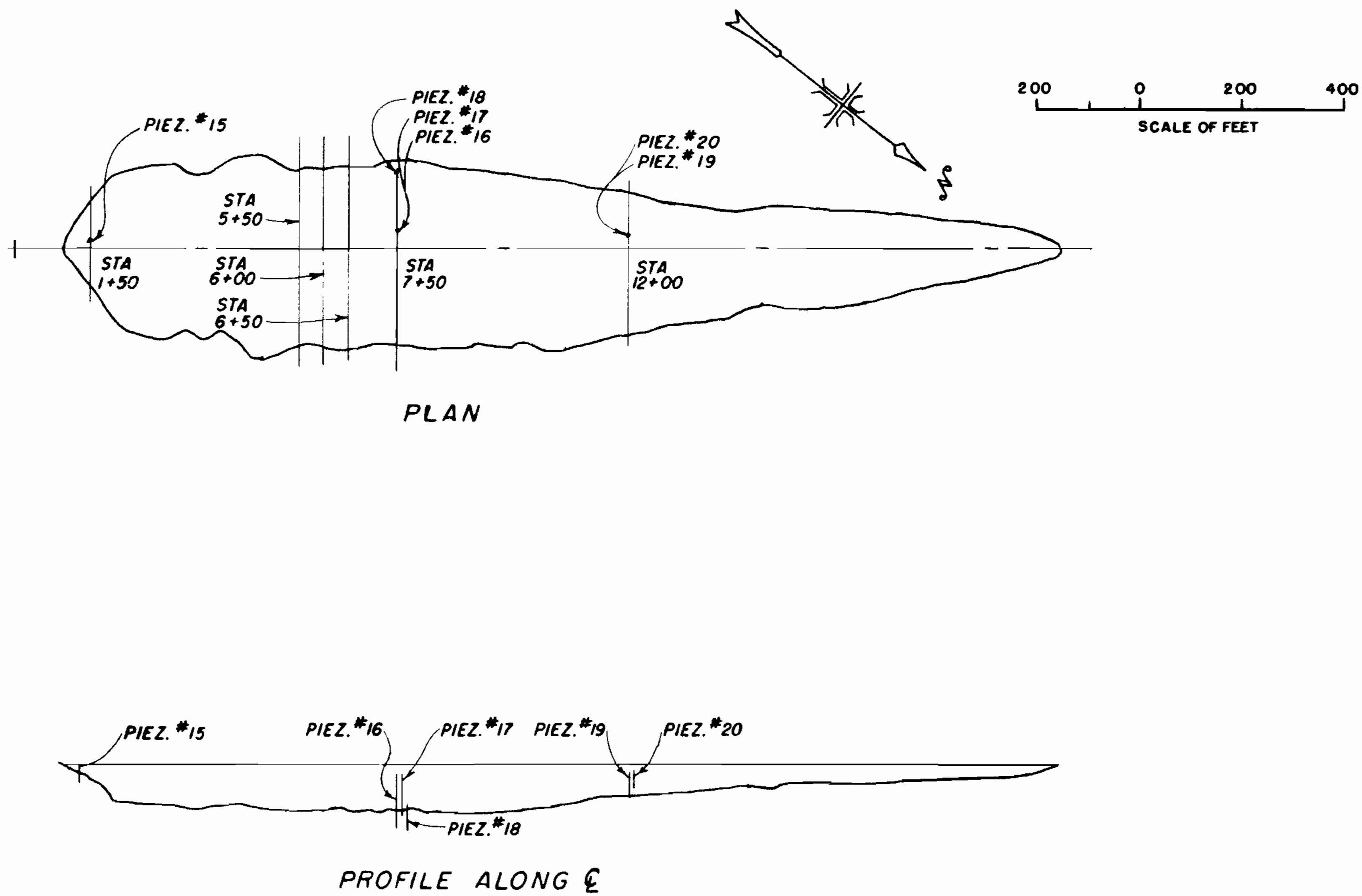
The characteristics of the responses of piezometer 17 clearly indicate that the pore pressures measured were a result of the water pressures in the foundation rather than in zone 1 material.

7.2.4 Response of Piezometer 18: Piezometer 18 is also located along Sta. 7+50 but is 150 feet downstream of centerline, or about 22 feet from the downstream toe. The piezometer was located about 13 feet into rock. Because of the way the rock drops off in the downstream direction (see Figure 7-3) in the area of Sta. 7+50, the piezometer is about 18 feet below the cutoff trench. The response history of piezometer 18 is shown on Figure 7-9. The only available readings for piezometer 18 during first filling are from mid-April to mid-June 1986. These show the piezometer responding to reservoir levels with pressures above the dike/foundation contact by about 8 feet. Such a reading correlates well with the reported seepage at the toe during first filling.

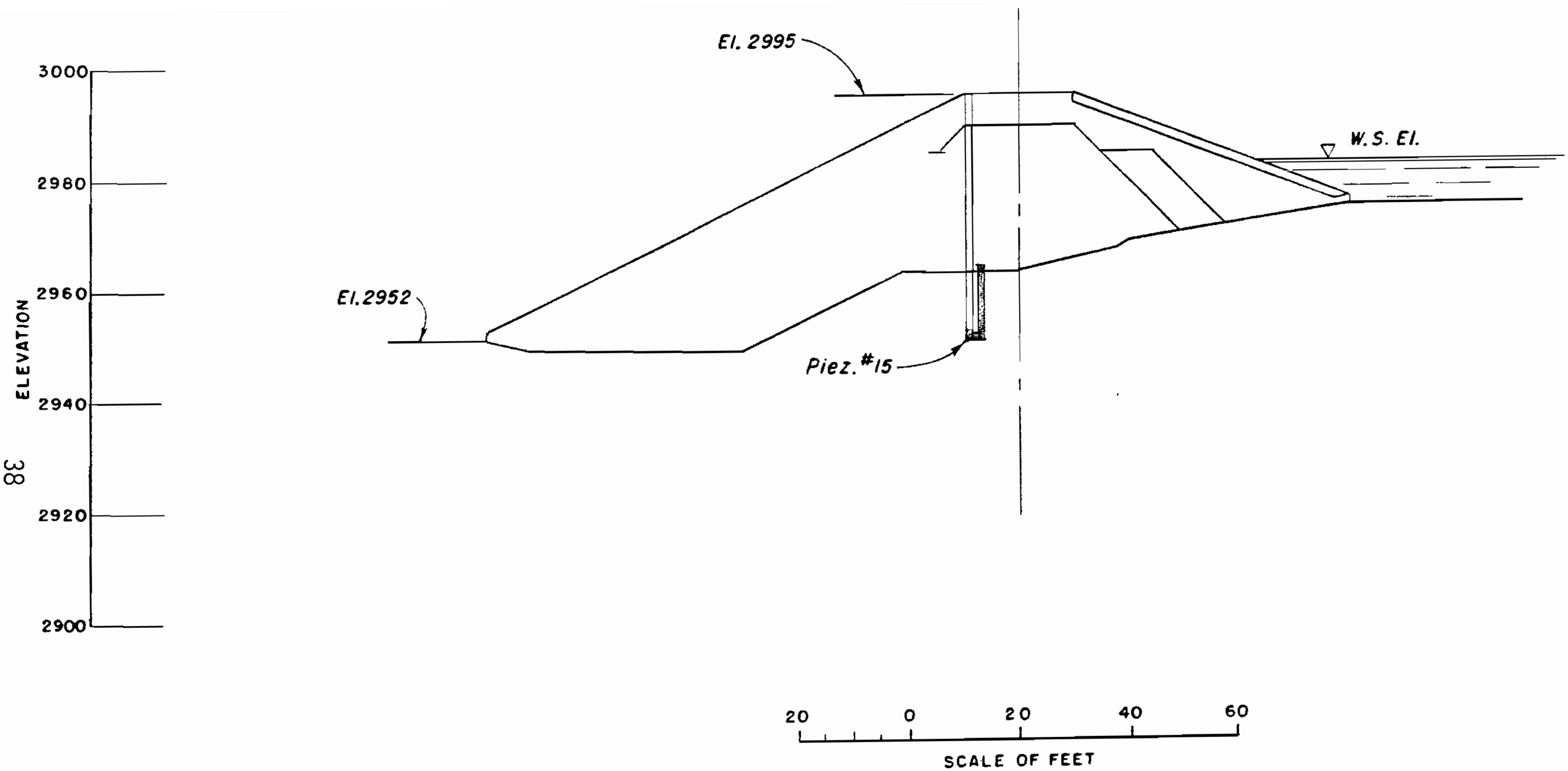
Following reservoir lowering and refilling, the piezometer response is available for reservoir elevations above 2937 and shows a steady increase. However, as with piezometers 16 and 17, there is less piezometric head at a comparable reservoir elevation than in the initial filling by about 10 feet when the reservoir reached elevation 2970 in April. By June 28, 1987, the piezometer under relatively constant head had gained 11 feet of head and was within 1-1/2 feet of the top of the fill at the piezometer location. The next reading taken on July 20 showed a 6-1/2-foot decrease in head. Then an abrupt drop of 11 feet to elevation 2905 took place between September 9 and 14, 1987. These variations were associated in time with the extensive seepage occurring at the toe near Sta. 6+00 and the completion of grouting. The piezometric head rose slowly (from 2905 to 2907) as the reservoir rose from elevation 2966 to 2983 on December 31, 1987. Then the piezometer began rising steadily with no appreciable increase in reservoir head. For example, at elevation 2983 on August 17, 1988, the piezometric head was 2915.2, or an increase of 8 feet of head. Subsequently, the piezometric elevation rose to 2920 just prior to failure. The rapid response at the onset of filling showed that this piezometer was controlled by foundation seepage rather than embankment seepage.

7.2.5 Response of Piezometer 19: Piezometer 19 was located at the dike/foundation (Zone III) contact along Sta. 12+00, 30 feet downstream of centerline. Based on the survey of Sta. 12+00 and the anticipated depth of the piezometer, it was determined that the piezometer was founded about 6 feet into foundation rock. The response of the piezometer is shown on figure 7-10. Upon first reservoir filling the piezometer indicated that it was in a draining mode (losing water into the fill and/or foundation). Upon refilling in 1986, the piezometer rose from its base elevation 2933.4 (January 1987) to elevation 2938.4 (March 1987) which is approximately the dike/foundation contact. No change in piezometric response took place during the remaining 1-3/4-years of operation of the reservoir.

7.2.6 Response of Piezometer 20: Piezometer 20 is located at Sta. 12+00 in the Zone IV material 30 feet downstream of the centerline and about 20 feet above the dike/foundation contact. No water pressure was ever observed in the piezometer as the phreatic surface through the dam had not developed.

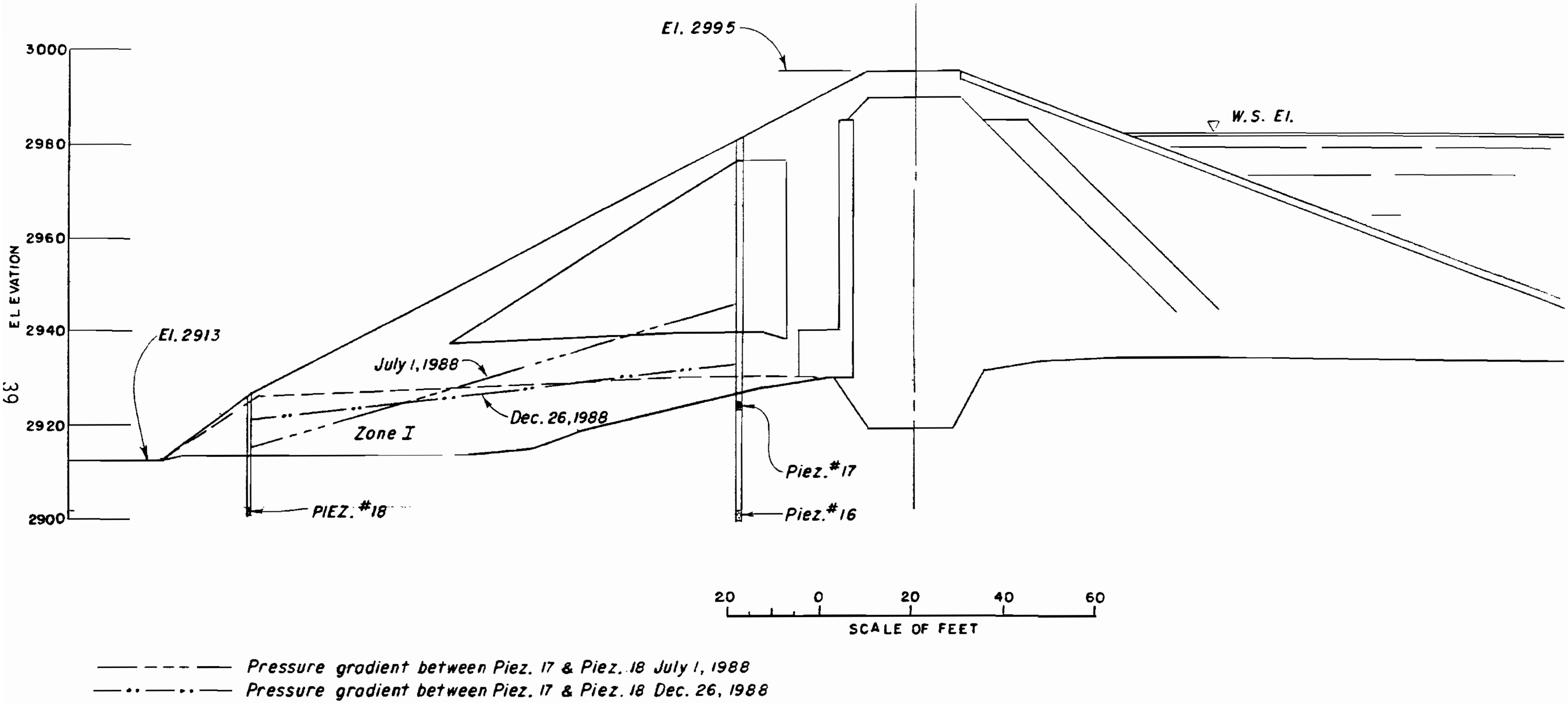


PLAN & PROFILE OF PIEZOMETER LOCATIONS
QUAIL CREEK DIKE
FIGURE 7-1

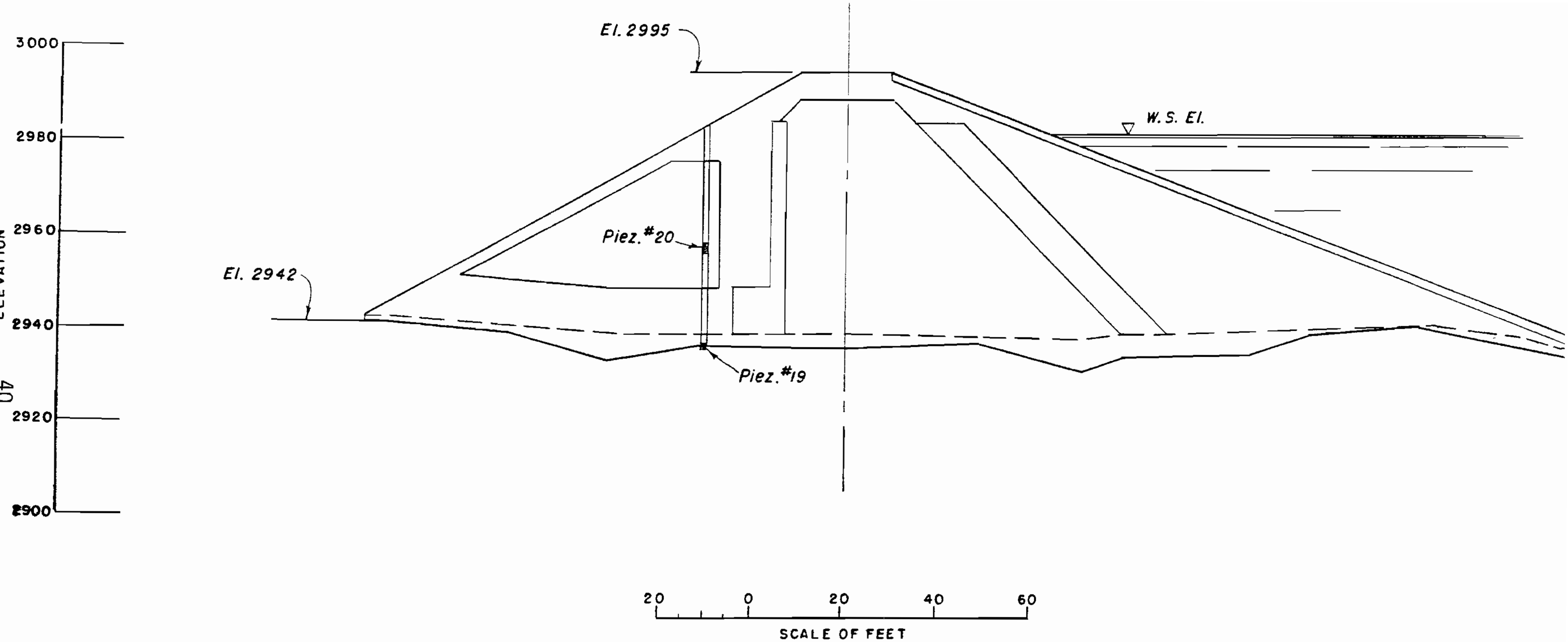


PIEZOMETER AT STA. 1+50
QUAIL CREEK DIKE

FIGURE 7-2

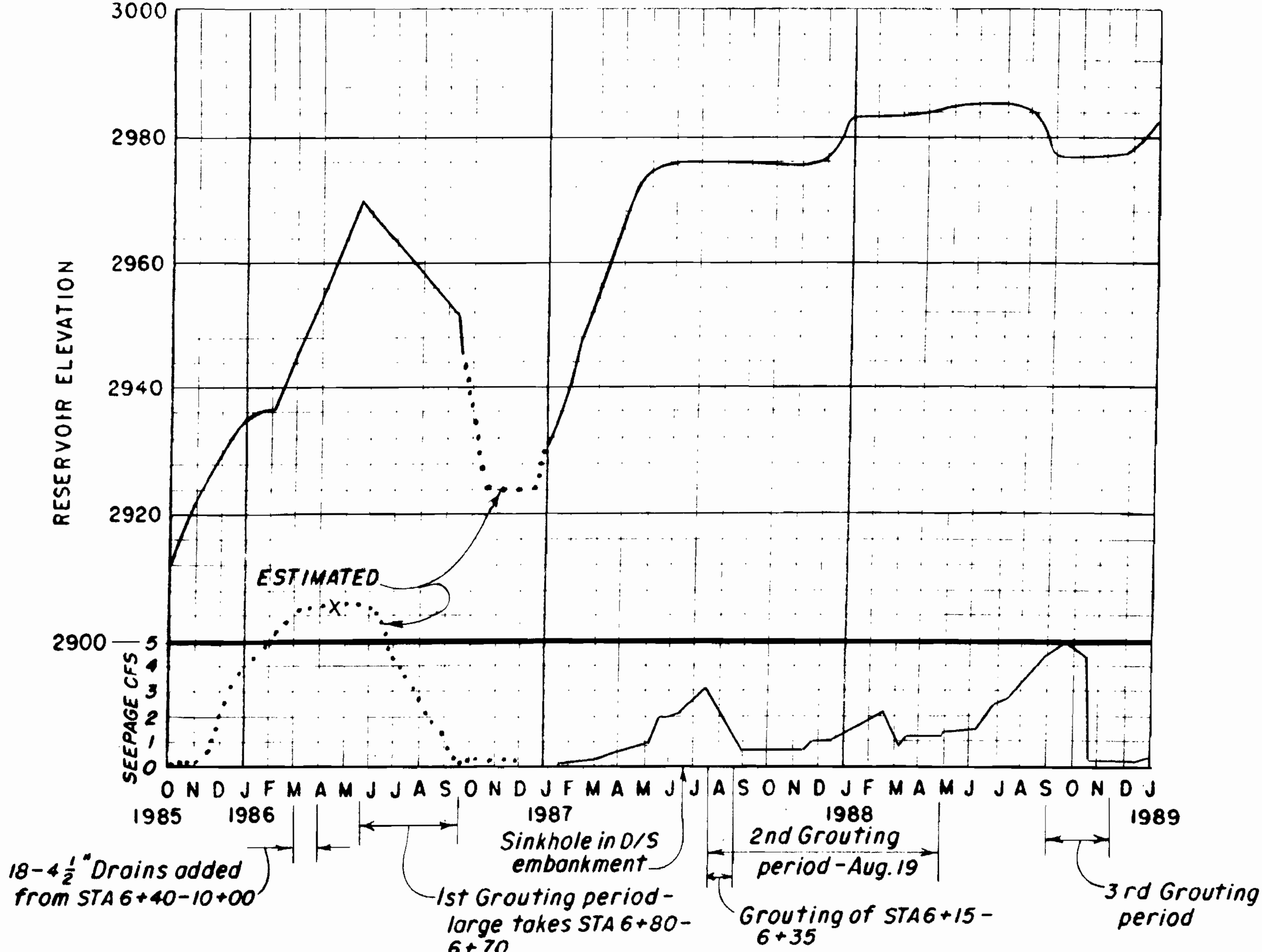


PIEZOMETERS AT STA. 7+50
QUAIL CREEK DIKE
FIGURE 7-3

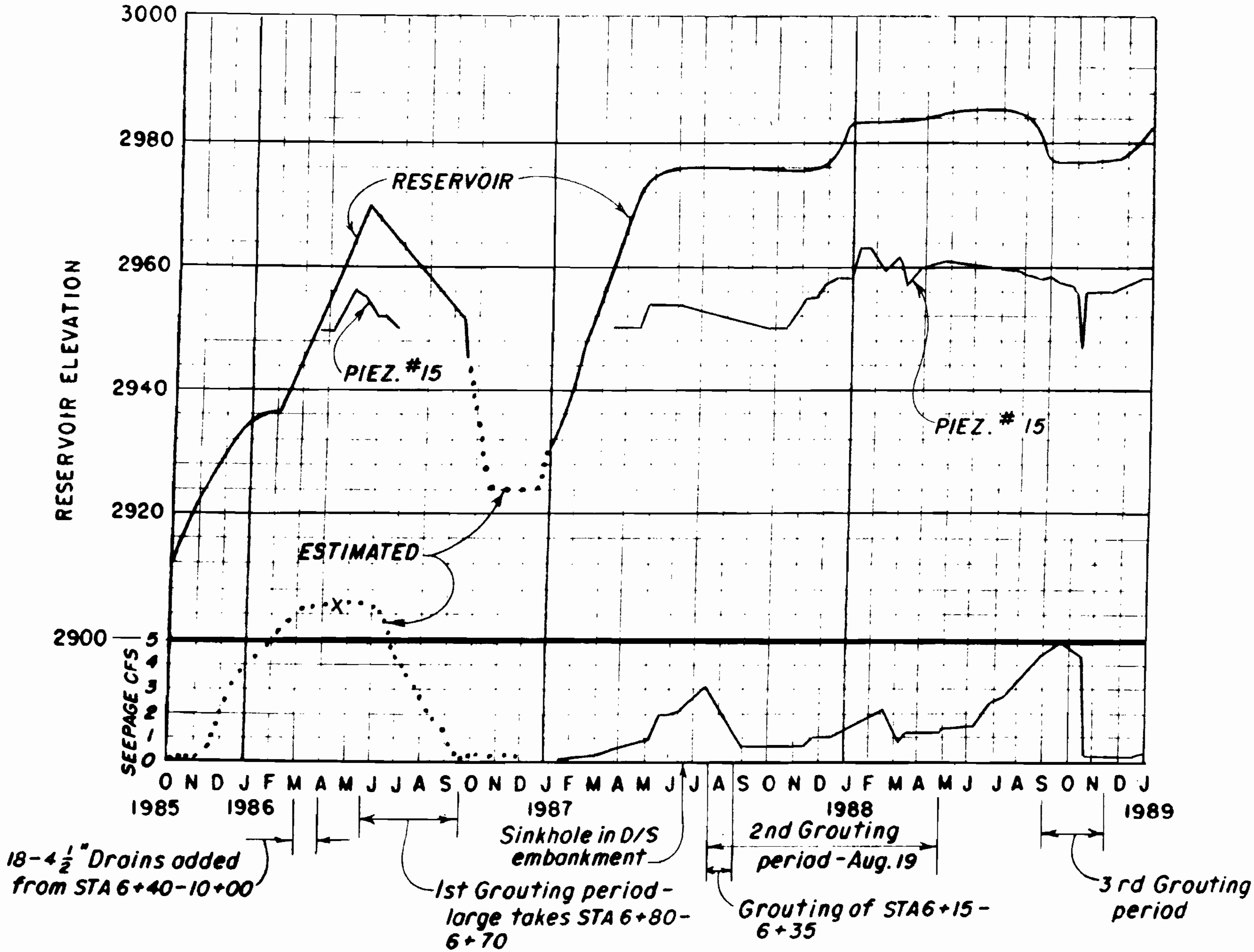


PIEZOMETERS AT STA. 12+00
QUAIL CREEK DIKE

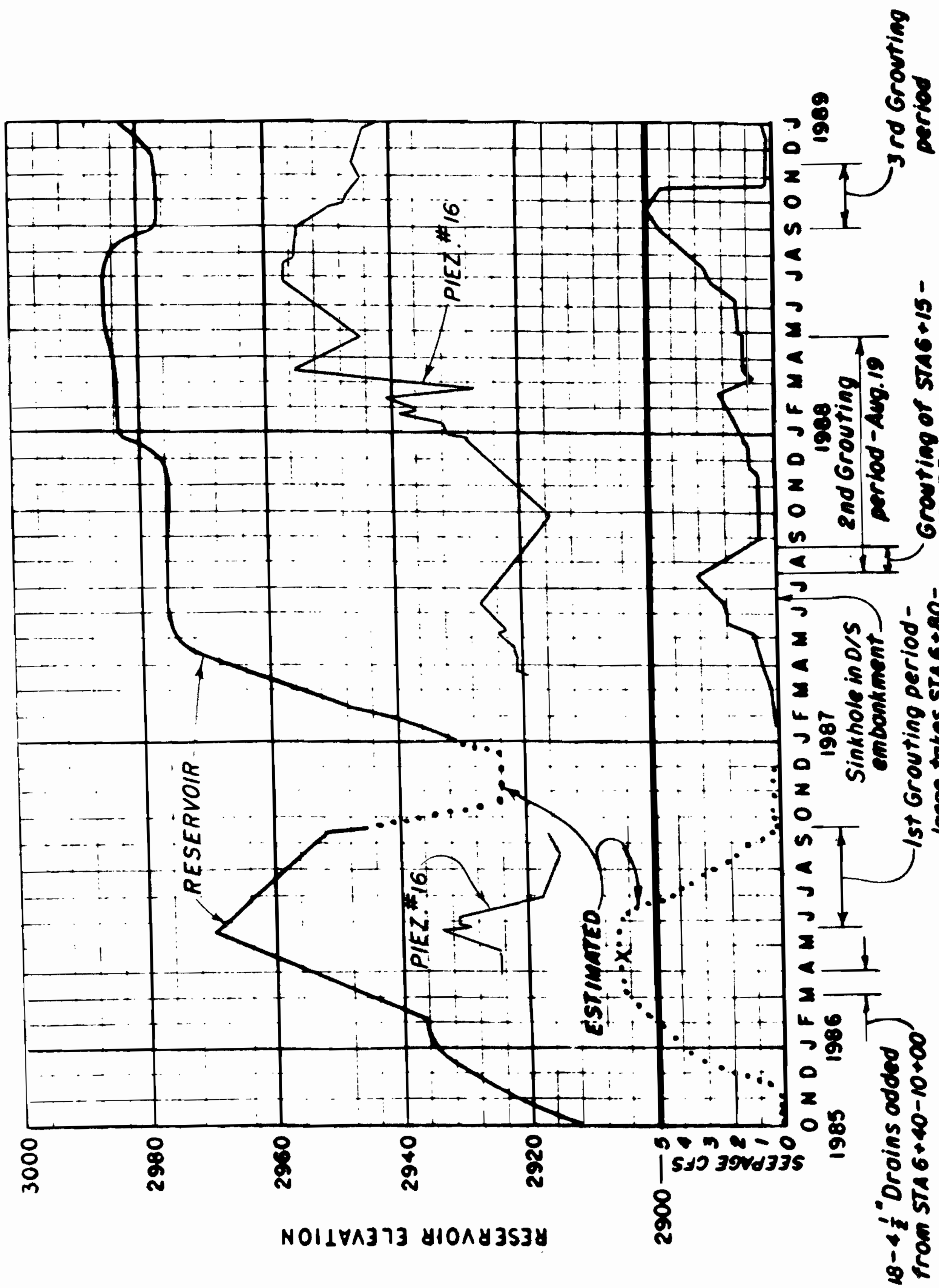
FIGURE 7-4



RESERVOIR OPERATION
FIGURE 7-5

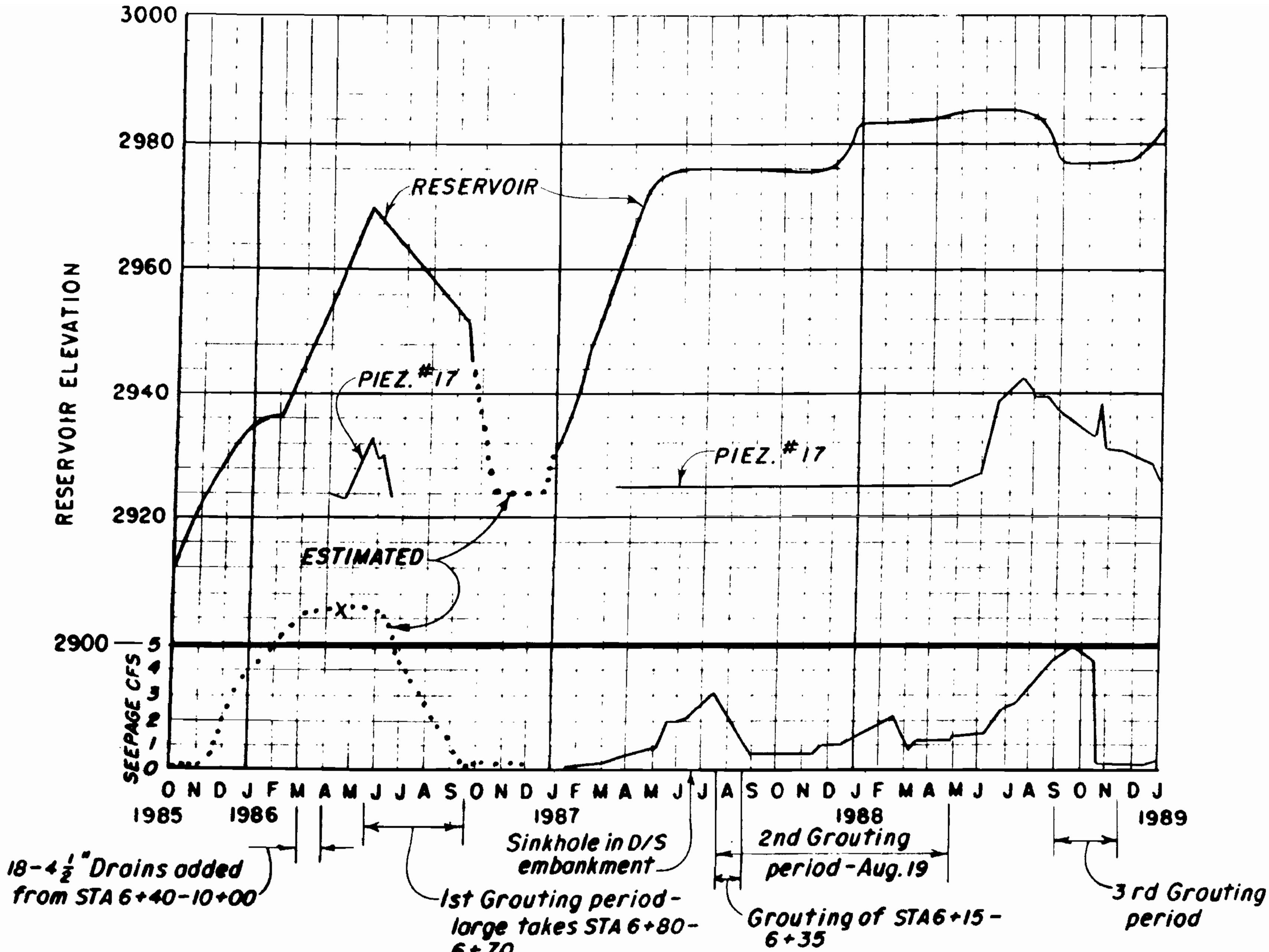


RESPONSE OF PIEZOMETER 15
FIGURE 7-6

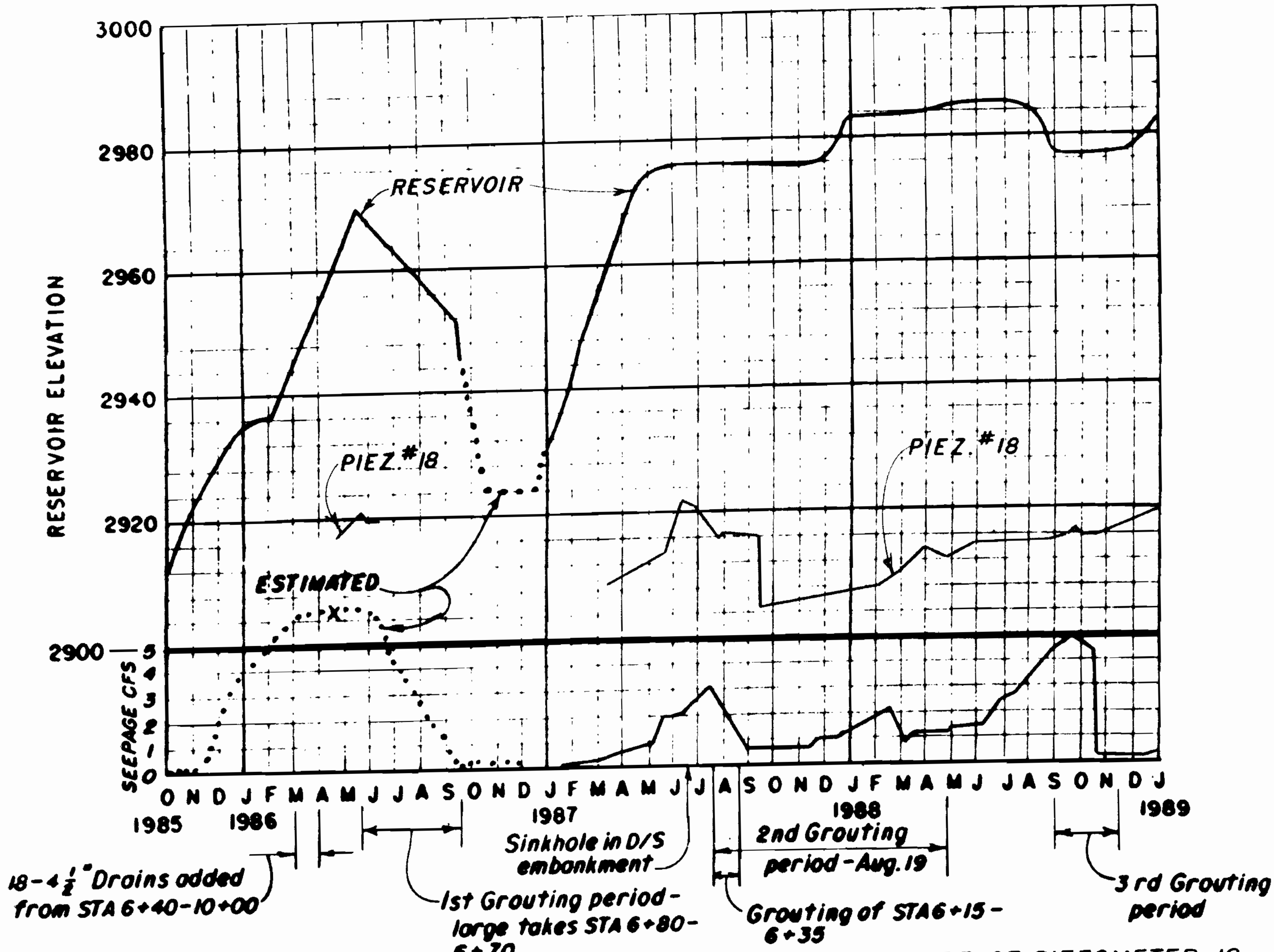


RESPONSE OF PIEZOMETER 16

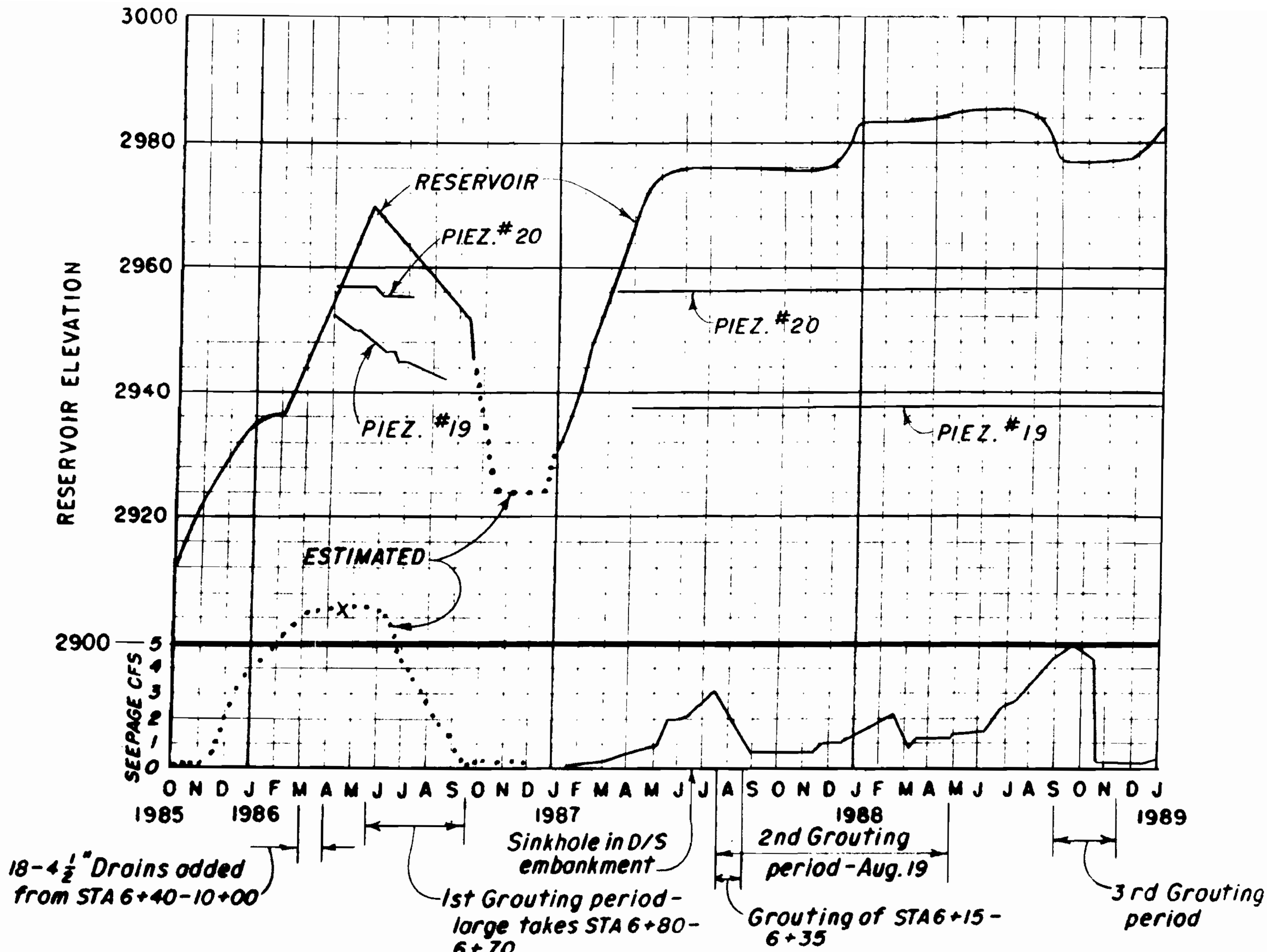
FIGURE 7-7



RESPONSE OF PIEZOMETER 17
FIGURE 7 - 8



RESPONSE OF PIEZOMETER 18
FIGURE 7-9



RESPONSE OF PIEZOMETERS 19 & 20
FIGURE 7 - 10

8. FOUNDATION SEEPAGE AND REMEDIAL GROUTING

On first filling of the reservoir in 1985, as water began to rise against the dike, seepage began to occur downstream and at the toe. By March 1986 foundation seepage had increased to the point that it was considered desirable to collect the seepage and reduce pressure at the toe. Thus, 18 inclined drain holes (see paragraph 2.4) were drilled into the foundation from the downstream toe between Sta.s 6+40 and 10+00. In April 1986, plans and specifications for grouting the dike foundation were prepared. A contract for grouting was awarded on April 22, 1986, and work started on May 12, 1986. At that time the reservoir elevation was about 2969 and total seepage from the dike was estimated to be 6.3 cfs. The major seepage areas were downstream of the toe at about Sta.s 3+00 and 6+50.

8.1 1986 Grouting Sequence

Grouting through the dike embankment was started on May 12, 1986, and was completed on September 11, 1986. At the completion of grouting the seepage downstream of the toe was estimated to be 0.3 cfs with the reservoir at elevation 2949.

During this sequence of grouting, holes were drilled through the embankment at a 70° angle (20° off vertical) into the foundation. The holes were drilled along and in the plane of the dike centerline. A review of grouting records indicates that, generally, standard grouting procedures were used. The split spacing method of grouting was used. Generally, primary holes were drilled on a 40-foot spacing and split at least once. In areas of high grout take, the spacing was split so that holes were on a 5-foot spacing. Also, at sections of high grout take, a second and third line was installed to reinforce the grout curtain. Type II cement was used for grouting. The thinnest mix used was 5:1 (water to cement ratio by volume) and grout was thickened as required. In holes with large takes, mixes as thick as 0.5:1 were used. Fillers such as sand were used to bulk the grout. It was reported that, in general, guage pressure of 70 pounds per square inch (psi) was used for grouting when the casing was set in rock. When casing was pulled up into the embankment, grouting pressures were reduced to 10 to 20 psi. The practice of pulling the casing into the embankment and grouting under pressure is not considered a standard procedure.

In general, this first sequence of grouting located several areas along the dike foundation where open conduits were discovered at the embankment foundation contact or at shallow depths within the foundation (see Figure 8-1a-c). The large conduits were indicated by large water losses and large grout takes. Areas where hole spacings were split to 5-foot centers and where large quantities of grout were injected were as follows:

<u>Sta. 1+10 to Sta. 1+30</u>: This area, originally grouted during construction with a 3-line curtain, was regrouted. Two locations from 30 to 50 feet below the foundation contact took more than 100 sacks of cement. Hole 1+50 took 322 sacks in the interval between 12 and 22 feet below the foundation contact.

Sta. 2+30 to Sta. 3+00: Significant grout takes of 135, 197, 254, and 168 sacks were injected into different holes at or within 30 feet of the foundation contact in this reach. The bottom of casing was pulled back so that as much as 15 feet of the embankment was grouted. The 168-sack take was recorded at the contact with a packer set about 3 feet into the embankment. Other injections of 38 and 57 sacks were made at the embankment/foundation contact area. An extra line of holes was drilled 10 feet upstream of centerline from Sta. 2+22.5 to Sta. 2+67.5. Although grout takes were not as large as those encountered along centerline, injections amounted to as much as 90 sacks in one interval.

Sta. 4+70 to Sta. 4+90: This was an interval where conduits were located deeper in the foundation. Grout take of 326 sacks occurred from 28 to 35 feet below foundation line in hole 4+80, and a take of 560 sacks occurred in hole 4+85 in the interval between 30 and 50 feet. An extra line of holes was placed 10 feet upstream of centerline from Sta. 4+67.5 to Sta. 4+92.5. Grout takes as high as 49 sacks occurred from 40 to 50 feet below the foundation contact, and 37.5 sacks were injected into the interval from 10 to 20 feet above the foundation contact. Two additional holes were drilled for a closure line, located midway between the first two lines; the holes accepted no grout.

Sta. 6+20 to Sta. 6+60: Large grout takes occurred at shallow depths in the foundation and also in the embankment. The greatest take of 805.5 sacks was recorded from 7 to 12 feet into the foundation. Records indicate that 100.5 sacks were injected from 2 to 10 feet into the embankment above the foundation in hole 6+55 and that 323 sacks were injected from 4 to 7 feet into the embankment in hole 6+60. Three additional holes were drilled 10 feet upstream of centerline from Sta. 6+22.5 to Sta. 6+62.5 with no significant grout takes.

Drilling along remaining parts of the embankment encountered sporadic and isolated zones of large grout take such as a 110-sack take in the interval from 10 to 50 feet below foundation contact in hole 8+80 and 234 sacks from 5 to 35 feet in the foundation in hole 12+70. Hole patterns and grout takes are shown on Figure 8-1a-c.

8.2 1987 Grouting Sequence

In June and July 1987, substantial seepage developed in the vicinity of Sta. 6+00 near the downstream toe. On June 25, a sinkhole about 2 feet in diameter was reported near the downstream toe; further observation identified a wet zone about 2 feet wide at the contact of the Zone I and III embankment material. It was concluded that water was moving through the bedrock hogback ridges, which had been left in place, at about Sta. 6+00. A graded filter was proposed to cover the seepage area. During excavation of the sink area, a hole about 8 to 10 inches in diameter was discovered in the Zone I material adjacent to the shale ridge. A "dirty" flow estimated at 0.5 to 1 cfs was exiting the hole. The excavated area was enlarged and a graded filter blanket was placed. This instance was the first observed evidence of water emerging along the embankment/foundation contact with enough force to erode or blow out the Zone I material.

The grouting program was started on about July 14, 1987, with the reservoir at elevation 2970. This grouting sequence was completed in April 1988. The main thrust of this grouting sequence was to locate and grout the channels causing the seepage in the vicinity of Sta. 6+00 at the toe and again to regrout the upper left abutment through the buff sandstone. Both vertical and angle holes were used and grouting procedures were similar to those described for the 1986 sequence.

On July 17, 1987, after water testing and grouting holes between Sta. 6+00 and Sta. 6+70, it was observed that flow from the upper toe drain was reduced by about one-half, but that the flow began surfacing. A sinkhole about 2 feet in diameter had also formed above the drain rock at Sta. 6+35 in the Zone III embankment. On July 21, while grouting hole 5+95, water flowing from the toe drain pipe turned muddy. By this time, a sinkhole about 30 feet up the embankment had increased to about 5 feet in diameter with water actively bubbling up. Equipment was mobilized to fill and blanket this sinkhole which was at least 4 feet deep.

On July 23, 1987, while grouting hole 5+95, grout flow was observed from the top drain and from three locations in the embankment with heavy grout flow in the vicinity of the sinkhole. While grouting hole 6+05 in the top 5 feet of the foundation, grout showed at the embankment sinkhole after injection of 5 sacks of 2:1 mix and stopped after 10 bags of 1:1 mix. A total of 192 sacks of grout was injected into this zone. A large flow of water continued from the embankment sinkhole area. While grouting hole 6+45 in the lower 10 feet of the embankment with 4 sacks of 2:1 mix, waterflow ceased at the embankment sinkhole area and returned to the drain. On July 24, all flow from the drains was clear and no water was issuing from the embankment.

On July 27, 1987, when grouting the interval that included about 5 feet of embankment and 5 feet of foundation in hole 6+00, grout was flowing at the toe nearly as thick as at the injection point. Grout mixes were as thick as 0.8:1 at this time. Hole 5+90 was grouted on July 28. Large grout takes occurred from 7 feet in the embankment to 15 feet into the foundation. Bran and cottonseed hulls were mixed into the grout for filler with a 0.8:1 mix. The hole finally sealed after accepting 372 sacks of cement and fillers. While grouting at Sta. 5+90, water emerging at the toe moved to Sta. 6+35 and boiled out of the drain rock 10 feet above the toe.

By July 30, 1987, no significant change in toe seepage had occurred since the start of grouting. It was concluded that there was a softened and eroded area of the core trench between Sta.s 5+90 and 6+05.

On August 11, dye was injected into the embankment/ foundation contact zone in hole 5+97.5 and appeared at the toe in only 2 minutes 45 seconds. A thixotropic stabilizer chemical was added to the grout to improve its thickening and adhesive qualities. With the added chemical grout, hole 5+97.5 sealed off after a total injection of 125 sacks. At this time it was apparent that holes on 2.5-foot centers had not been successful in reducing the seepage. It was suggested that the reservoir be lowered 20 feet to reduce the water pressure, and other alternatives were discussed for stopping the seepage if continued grouting was unsuccessful.

On August 13, 1987, an open conduit was encountered from 5 to 10 feet below foundation contact in hole 5+92.5. This zone was grouted for 2 days using thick grout mixes, bran, cottonseed hulls, and sand. Grout showed at the toe initially, but on August 14, a thick mix including bran, stabilizer, and sand plugged the conduit. Water seepage at the toe was essentially stopped. Grouting work was completed in April 1988. At that time seepage from the dam was about 1 cfs with a full reservoir (El. 2985).

General observations concerning data reviewed from the second grouting sequences are as follows:

<u>Sta. -2+20 to Sta. 2+15</u>: This reach of foundation was grouted for the third time. Grout holes were placed 220 feet beyond the left end of the dike. About 70 vertical and 30 angle holes were drilled with many as close as 2 feet; a few were spaced 1.5 feet apart to obtain closure. Although average grout takes do not appear high, there were sporadic and isolated open conduits. Grout take was as high as 218.5 sacks in a 7-foot interval. Grouting was performed as high as 18 feet above the soil/rock interface in the embankment.

<u>Sta. 3+55 to Sta. 3+95</u>: Holes as close as 1 foot apart were drilled to obtain closure. Highest grout takes were from 7 to 17 feet in the foundation with the maximum being 190 sacks. One hole had takes of 86 and 56 sacks in intervals that included both the embankment and the foundation. Closure was generally obtained with the split spacing of grout holes.

<u>Sta. 4+78 to Sta. 5+80</u>: This interval was extensively grouted using angle holes in opposite directions and vertical holes. A series of close-spaced vertical holes and crossing angle holes were used to locate an open conduit at about Sta. 5+25 between 58 and 68 feet below the foundation contact. Maximum grout take in this zone was 675 sacks.

<u>Sta. 5+80 to Sta. 6+85</u>: A series of angle holes as close as 1 foot apart defined an area of open conduits at or near the embankment/foundation contact between about Sta.s 6+15 and 6+35. About 1,500 sacks of cement were injected into this area with the highest take being 335 sacks from 5 to 10 feet below top of foundation. Much grout in this area was injected with the upper packer 5 to 8 feet into the embankment.

<u>Sta. 6+45 to Sta. 6+96</u>: A series of vertical and angle holes were used to locate two open conduits from about 29 to 40 feet beneath the top of foundation at Sta. 6+85 and from 35 to 38 feet beneath the top of foundation at Sta. 6+96.

Graphic presentation of the grout hole patterns with grout takes are shown on Figure 8-2.

8.3 1988 Grouting Sequence

Seepage 150 feet downstream of the dike toe at Sta. 6+25, which was not completely stopped during the 1987 grouting sequence, gradually increased until mid-June 1988 when the rate seemed to increase faster. From mid-July to August 1, the flow was increasing at the rate of 0.4 cfs per week. On August 29 it was reported that the reservoir had risen from elevation 2982 to 2983.5 due to a flood. Seepage from the dike was showing a heavy reddish color which had not been observed prior to the flood. Seepage at this time was about 5 cfs. By the next day the color of the seepage had cleared some. Drilling for grouting was again started on August 30 and was completed on November 18, 1988.

On August 31, while inspecting the seepage areas, traces of purple clay were found in the channel. This clay was similar to the Zone II material placed in the bottom of the cutoff trench and in the upstream cutoff trench. A layer had also been placed from the trench to the upstream toe of the dike. It was concluded that the clay must be coming from one of those sources.

On September 1, 1988, a longitudinal crack was observed on the crest between Sta.s 6+50 and 6+70. While grouting hole 6+60 at 70 psi with a 3:1 mix and a packer 2 feet below the top of foundation, the crack was observed to be widening and extending. Pressure was reduced to 10 psi, and extension of the crack stopped. This was the first time that evidence was observed to indicate hydraulic fracturing of the embankment could occur when water testing and grouting near the foundation/embankment contact at the pressures being used. Other incidents of hydraulic fracturing observed and reported were on September 5 and November 11.

The major thrust of the 1988 grouting sequence was to locate and seal the conduit(s) allowing seepage through or under the embankment. All of the areas of high grout take in previous grouting sequences between Sta.s 4+25 and 6+90 were drilled and tested, but it was not until September 21 that a major opening was discovered in hole 5+47.5. That angle hole penetrated an opening where drill rods dropped 10 inches at a point about 40 feet beneath the top of foundation at about Sta. 5+05. Of interest is that several other holes penetrated the general area with no grout take except for vertical hole 5+03 which took 67 sacks in the conduit area. Grouting of the 8-foot interval in hole 5+47.5 was very difficult and continued from September 21 to October 14. Heroic efforts were made to plug the conduit against substantial flow caused by the reservoir head. Bulking materials used included sand, barite, bentonite, bran, pea gravel, 3/4-inch gravel, cottonseed hulls, straw, wheat, and chemical stabilizers. Material as heavy as the 3/4-inch gravel was carried and ejected with the seepage downstream.

After 10 days there had been no success at plugging the conduit in hole 5+47.5. The options considered included (1) lowering the reservoir, (2) asphalt grouting, and (3) placing concrete. Lowering the reservoir was considered a last resort by WCWCD. For technical and logistic reasons, asphalt grouting was ultimately rejected, and on October 11, injection of concrete mixes was started. The initial 5 yards of concrete went down with no trouble, but the sand and gravel flowed out at the toe 45 minutes after placement. Additional concrete was placed and the downstream flow was reduced, but the quantities of concrete at that time were not sufficient to plug the conduit, and the material washed out. After arranging for continuous concrete loads and for a combination of grouting and concreting, the conduit was finally plugged on October 14 when the hole was rechecked and backfilled. About 100 cubic yards of concrete and 2,356 sacks of cement plus large quantities of filler materials were injected into the 8-foot interval in hole 5+47.5. Seepage at the downstream toe had been reduced to 0.1 cfs on November 18 with the reservoir at elevation 2976. Grouting records for both the 1987 and 1988 sequences are shown on Figure 8-2.

8.4 Analysis of Seepage and Grouting Data Reviewed

Analysis of the remedial grouting programs taking into account the site geologic conditions and other data leads to the following conclusions:

A. Initial seepage on filling of the reservoir was through open joints and bedding planes in the shallow foundation rock.

B. Water-carrying conduits were very erratic and were formed by water travel along both joints and expanded bedding planes.

C. Deep conduits (below about 30 feet) in the foundation were likely pre-existing solution channels along joints and bedding planes which were able to conduct large flows to disperse in the upper 30 feet of the foundation.

D. Grouting of existing conduits forced the water entering from the reservoir to find other interconnected joints and bedding planes where gypsum and other filling materials could be solutioned and eroded by the flowing water.

E. Grouting restricted the network of drainage channels in the foundation downstream of the cutoff, thereby increasing hydraulic pressure at the embankment/foundation contact. Grouting forced water at higher pressure against remaining overburden and Zone I embankment material. Water flow was concentrated at the location of the hogback ridges which were elongated, intensely fractured mounds of bedrock protruding above the average level of the downstream dike foundation. Several hogbacks are located within the failure zone from Sta. 3+00 to Sta. 7+00. (See further discussion in paragraph 4.3.1).

F. Evidence of piping of Zone I embankment material along the foundation contact was seen as early as July 1987.

8.5 Foundation Seepage

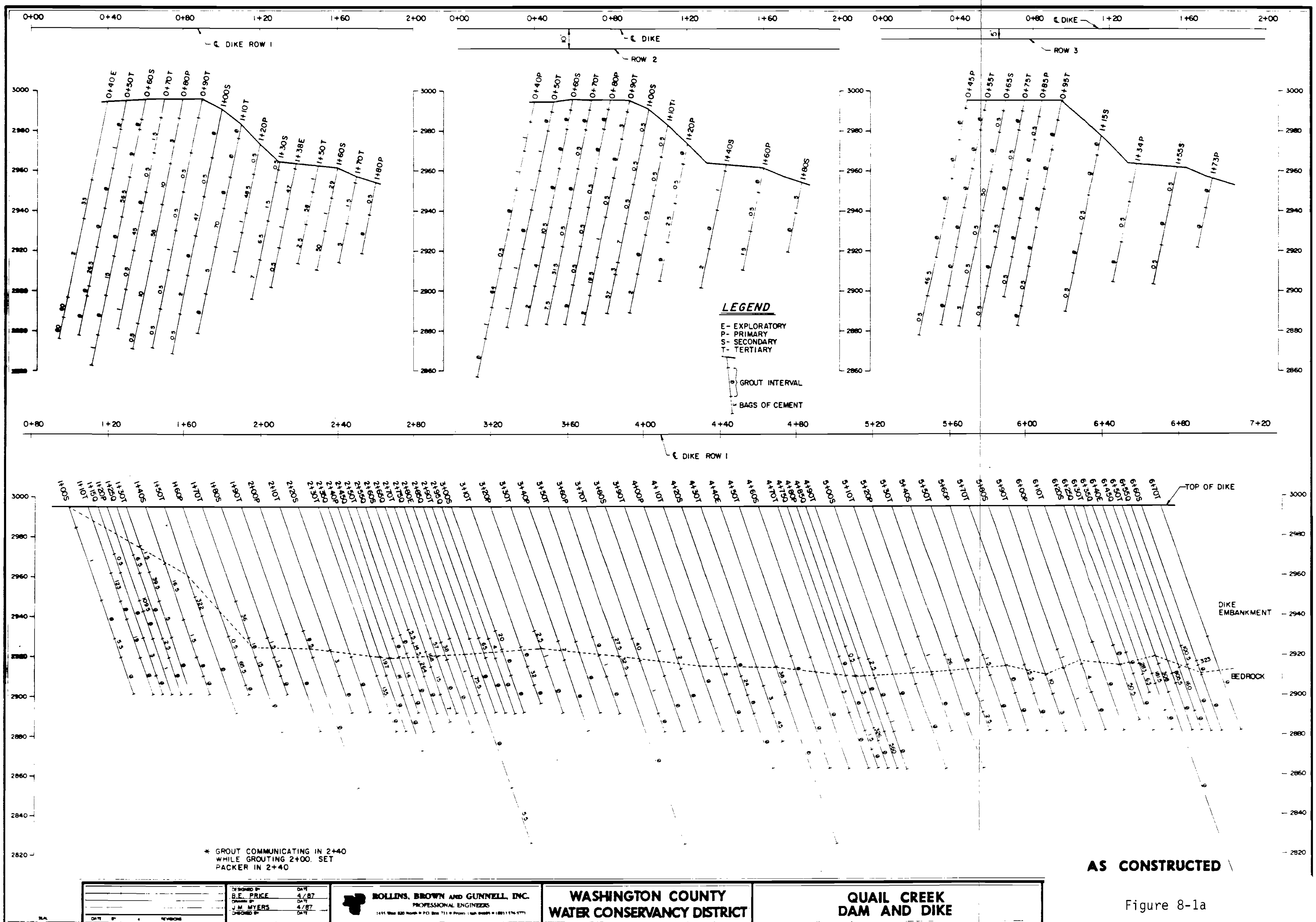
One of the major points of interest in the investigation of the operational performance and the ultimate failure of Quail Creek Dike is the amount and possible effects of the erosion/solutioning of foundation materials. One of the ways to determine the significance of the erosion/solutioning phenomenon is by examining the quantity of seepage. However, since the amount of seepage is also related to the reservoir elevation and the effects of grouting, these factors must also be considered when drawing any relationship between erosion/solutioning and seepage. Figure 7-7 shows the available information related to these parameters and reveals considerable insight to their relationships. The observations that can be drawn are:

A. Large increases in seepage were primarily related to concentrated seepage conduits in the foundation. When these conduits were grouted shut, the seepage returned to the same or below the levels of seepage prior to the development of the concentrated leak. The concentrated leaks could have developed by solutioning or piping, or they could have been nearly open prior to filling. Most likely, all of these conditions were operative to some extent in their development.

B. The foundation piezometers show a responsiveness to the effects of grouting and show indications of the development of "new" seepage paths. These new paths could have been opened by either erosion or solutioning.

C. Solutioning, if it was occurring, did not result in a steady increase in foundation seepage.

D. The grouting efforts had a greater impact on closing seepage paths than the solutioning had in opening them. This conclusion is strongly supported by the amount of seepage occurring in November and December 1988 (0.15 cfs). This low level was achieved after closing a single, deep-seated leak in the foundation (as opposed to a generalized grouting program). If solutioning had been occurring steadily and pervasively in the foundation, the background seepage level would have steadily risen. This was apparently not the case.



AS CONSTRUCTED

Figure 8-1a

1986 Grouting Sequence

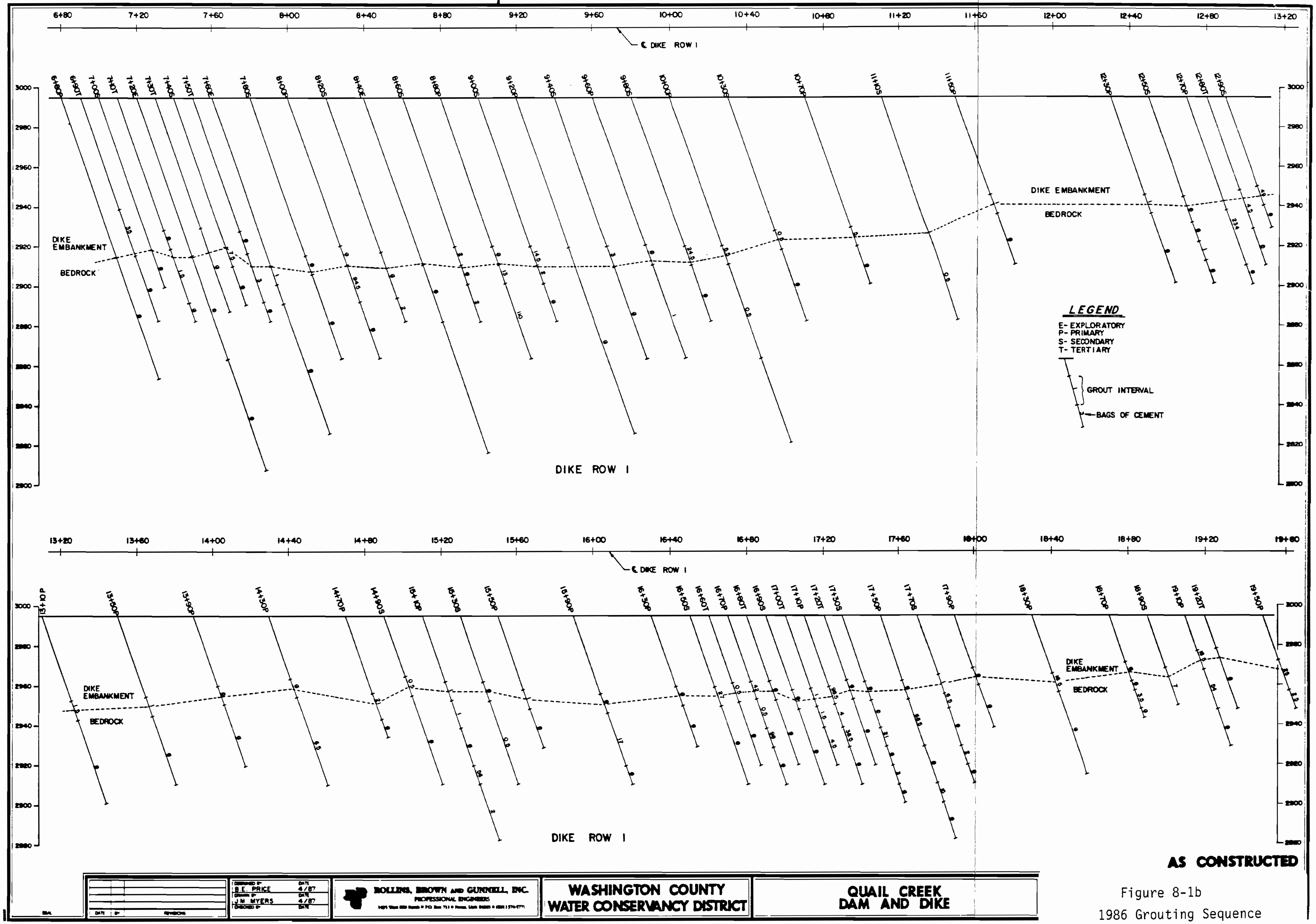


Figure 8-1b
1986 Grouting Sequence

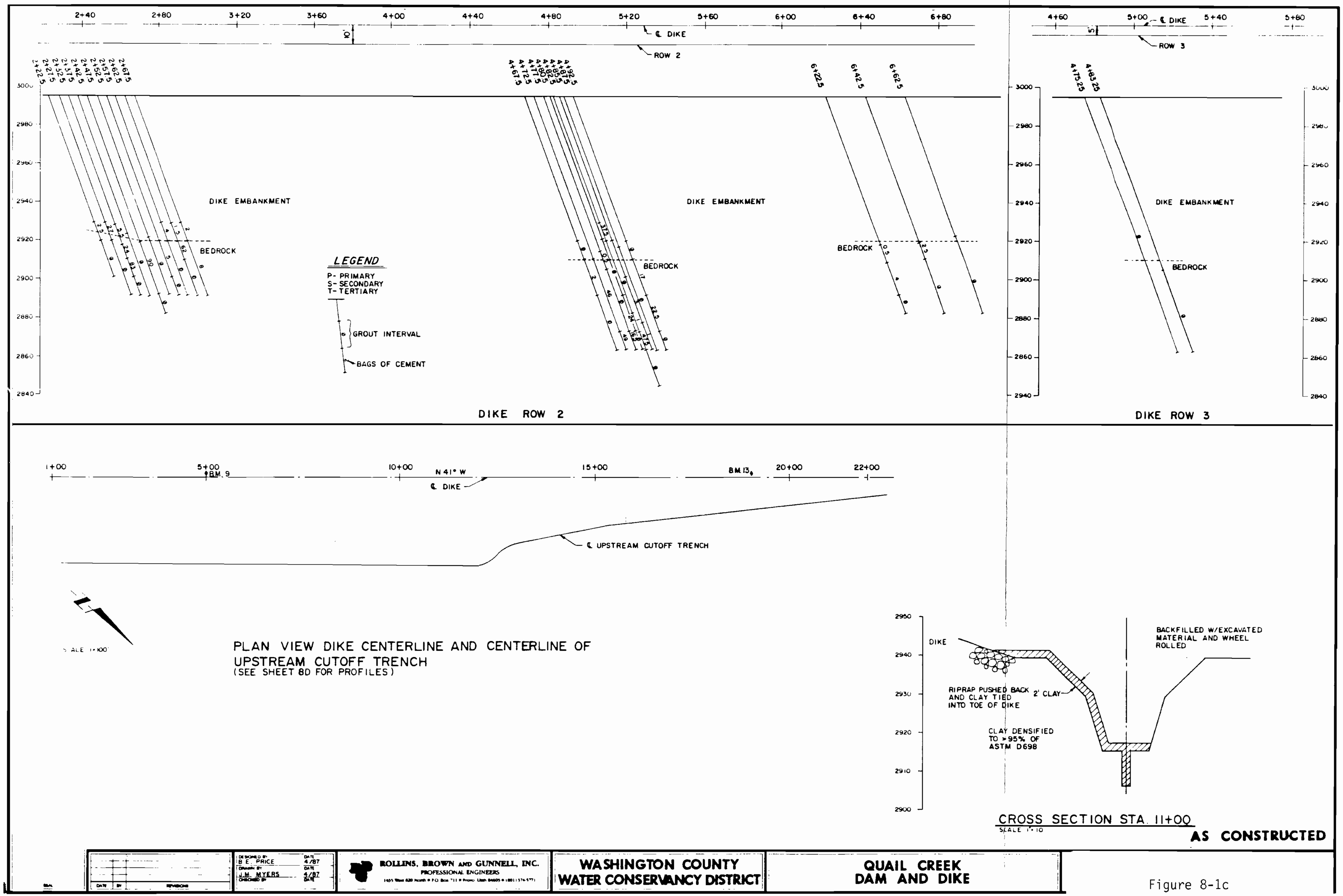


Figure 8-1c
1986 Grouting Sequence

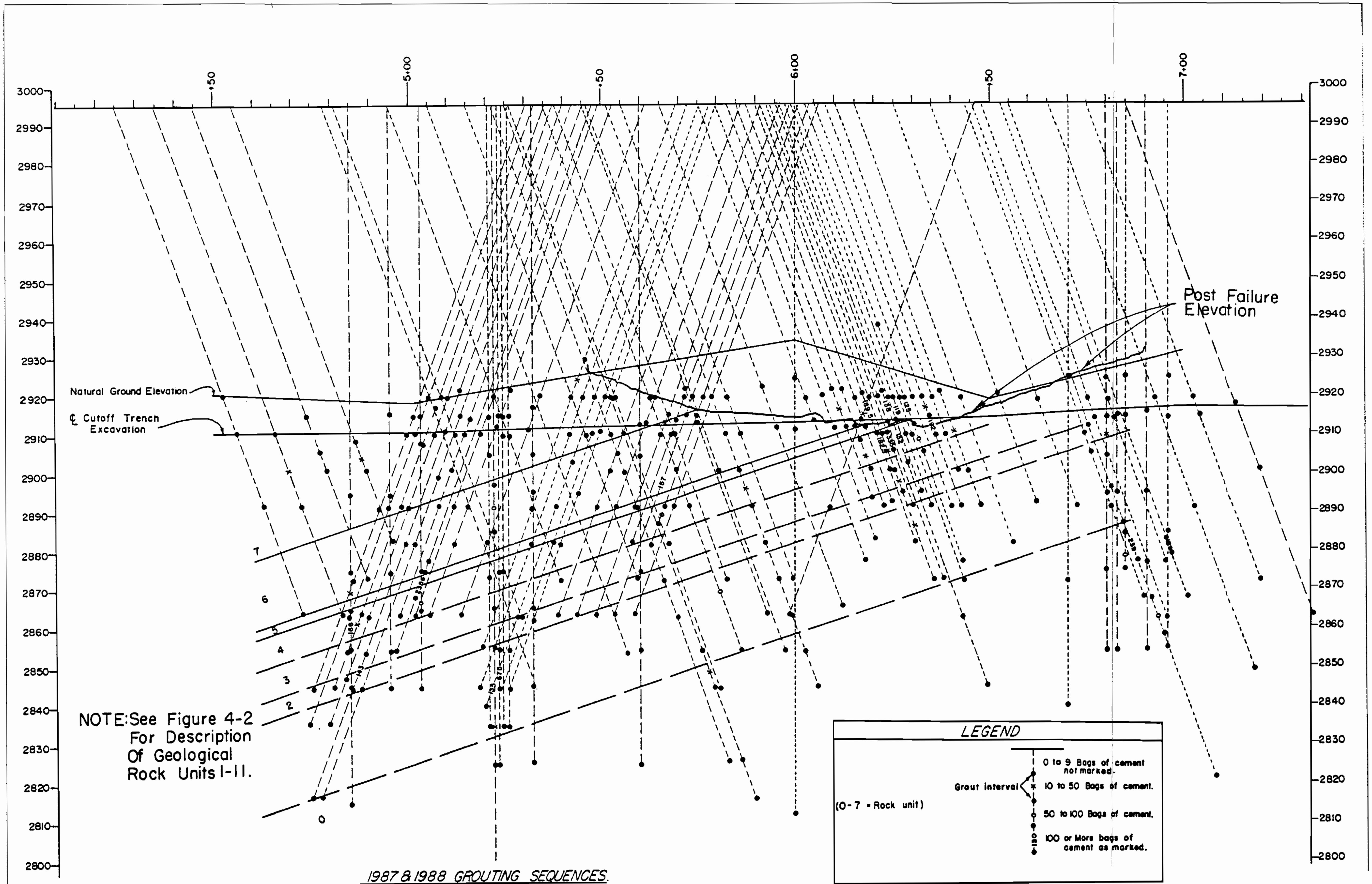


Figure 8-2

1987 and 1988 Grouting Sequence

9. DETAILED DESCRIPTION OF FAILURE

The detailed description of events leading to the failure is based on eyewitness accounts from several people who were involved in the effort to control the increasing seepage emerging at the downstream toe near Sta. 5+90 on December 31, 1988 (see Appendix D). Generally, descriptions of events coincide while recollection of times may differ. Because the final failure occurred at night, visibility was not good in all areas. A light plant set up on the dike crest was primarily focused on the emerging seepage at the downstream toe. It is also noted that the project had endured several previous alarms triggered by increases in seepage volume, discolored seepage, small sinkholes, etc., in 1986, 1987, and 1988. Consequently, witnesses were familiar with the project and necessary responses.

The first indication of a renewed seepage problem on December 31, 1988, was the observation of reddish brown seepage near the road approaching the dike's downstream toe. At about 10:30 a.m. the observer inspected the downstream toe of the dike and found that an estimated 200-300 gpm water was boiling up around the observation well located at about Sta. 5+90 (shown in plan on Figure 2-7). The flow was discolored and was emerging around the outside of the pipe. The pipe was a 12-inch-diameter vertical riser which was attached to a perforated toe drain pipe about 7 feet below the ground surface. The observer noted that the water level inside the pipe was several feet deeper and the water was clear. All of this indicates that the source of upward flow was not the toe drain. The observer called nearby sources for equipment and supplies of gravel. An inspection of the 12-inch Parshall flume located about 600 feet downstream showed that the volume of flow had increased over previous readings, and particles of the purple clay used in the bottom of the cutoff trench and in the upstream cutoff were observed.

For the next several hours, equipment operators and materials were marshaled. By noon the area of upward flow was about 6 feet in diameter, and the water boiling up 4 to 5 inches was carrying fragments of purple clay. Continuing efforts were made during the afternoon to build approaches to the area, and several loads of gravel and crushed rock were dumped. The area and volume of flow continued to increase until late in the afternoon WCWCD officials advised the Washington County emergency management director to prepare for a possible downstream evacuation.

With night approaching, a light plant was set up on the dike crest and focused on the downstream seepage area. It should be noted that during the evening several efforts were made to locate whirlpools or vortexes marking flow entrances in the reservoir adjacent to the dike. None were found.

By about 10:30 p.m., the seepage flow was estimated at about 70 cfs and the nature of the emerging flow changed from vertical boiling to a concentrated horizontal flow from a growing hole at the dike toe. At this point the effort to control the seepage was stopped, personnel and equipment were moved to safe locations, and downstream evacuation was ordered.

Between about 11:00 and 11:30 p.m., observers report that a wedge of the downstream slope about 50 feet wide, extending about one-third of the way up the slope and located over the horizontal exit hole, suddenly dropped down several feet and for a few seconds seemed to block the flow. Soon thereafter flow resumed, increased, and started removing collapsed material. The breach then grew upstream toward the reservoir by a continuing series of sloughs or collapses of the near-vertical rear or upstream face. Final breach through the dike to the reservoir occurred at about 12:30 a.m. on January 1, 1989. As the reservoir was released, the breach grew in width from about 100 feet to about 300 feet when flow through the breach stopped at about 1:00 p.m. on January 1, 1989. About 25,000 acre-feet of water flowed through the breach. The flow scoured and removed some of the original rock formation so that the maximum depth of breach below the dike crest is about 85 feet (see Figures 4-6, -7, and -8). Photographs 12-16 show the breach.

10. DISCUSSION OF POST-FAILURE INVESTIGATIONS

Failure of the dike resulted in a thorough cleaning of foundation rock in the breach area and downstream. The breach also created a cross section cut through the embankment on each side. Although these new exposures provided interesting observations and valuable data, the Independent Review Team considered that detailed investigations would be necessary to reliably identify the most likely causes and mechanism of failure. Immediately following the dike failure, geologists and engineers of the Utah State Division of Water Rights, under direction of the State Engineer, were on the site accumulating data. Members of the State Engineer's staff assumed responsibility for conducting numerous engineering and geologic studies requested by the Team, and WCWCD assumed responsibility for arranging and providing excavation equipment and for providing onsite supervision of equipment. These groups are commended for accomplishing the post-failure studies expeditiously.

10.1 Description of Investigations

In order to expedite the review and have an early examination of exposures, the Team requested that studies and investigations be conducted in two phases: Phase I was a continuation of work already started by the State Engineer's staff and work that needed to be done immediately. Phase II consisted of work of secondary importance or that may take additional time to accomplish. The following actions and investigations were accomplished at the request of the Team:

Phase I:

A. A photographic record of the site immediately after failure was developed (see photographs).

B. A concise photographic record of the site prior to and after embankment layback was made (see photographs).

C. A geologic map of the exposed foundation rock through the breach area was prepared. See Figure 4-1.

D. Six profiles were surveyed across the breach including one on centerline, two upstream, and three downstream. Profiles of original ground, excavated level, and post-breach surfaces are shown on Figures 4-6, -7, and -8.

E. Eleven units of foundation rock were identified across the breach and a measured section prepared. See Figure 4-2.

F. Twenty-three samples from rock units and joint filling material were collected for the Petrographic and X-ray Diffraction Mineralogical Study. See Appendix C.

G. Samples of purple clay were collected from the cutoff trench, Zone II in the dam, and upstream cutoff trench for X-ray diffraction analysis. See Appendix C.

H. A log of the upstream cutoff trench from Sta. 6+50 to Sta. 11+50 was prepared. The log was completed in Phase II. See Figure 4-3.

I. More than 240 joint attitudes were measured throughout the damsite and in the Upper Red Member above the foundation and 132 were plotted on a stereo net. See Figure 4-5.

J. Key features, such as grouted joints, pipe holes at foundation contact, sinkholes, drain holes, and collapse features were surveyed and mapped. See Appendix G.

K. A trench was excavated into foundation rock along centerline and logged. See Figure 4-4.

L. The embankment section on the right side was excavated about 50 feet into undisturbed material. The upper slope was laid back to prevent additional sloughing. Three to 5 feet of soil was left over the bedrock initially. Backhoe trenches were excavated to bedrock in both longitudinal and transverse directions in the presence of the Independent Review Team so that details of the embankment/foundation contact condition could be observed.

M. Portions of the downstream drainage trench were excavated to observe the condition of the drain pipe and rock.

N. Zones I, II, and III in several locations of the undisturbed embankment were tested for in-place density, and samples were obtained and tested in the laboratory for gradation, atterberg limits, and solubility, as appropriate. Zone II material was also subjected to pinhole tests to measure its potential for dispersivity. See Appendix C.

O. Zone I materials were chemically tested to examine the relative contents of sodium and calcium salts. See Appendix C.

P. Undisturbed record samples were obtained from embankment zones I, II, and III and sent to storage for safekeeping.

Phase II:

Q. The upstream cutoff trench was cleaned to its maximum depth and dewatered, and the geologic log was completed. See Figure 4-3.

R. The left abutment breach section was excavated, resloped, and trenched similar to the right side as described in paragraph L above.

10.2 Post-Failure Foundation

Foundation rock for the dike was closely examined throughout the breach area. The rock mass, as an entity, is capable of supporting all loads imposed by the dike embankment when properly prepared and treated. As a foundation, however, the rock mass has a number of undesirable qualities.

An important undesirable quality is the expanded bedding planes in the weathered zone with erodible and soluble gypsum filaments (see Photographs 6, 32, 32B). Linked with geologically old solution channels along the vertical joint system and deeper bedding planes (Photograph 31), the near-surface expanded bedding planes provided a wide assortment of open and/or easily erodible and soluble paths for water passage (see Photographs 25 through 30). In addition to forming water passage locations, the differential erodibility and solutionability of the beds within each rock unit provided preformed "pipe" locations at the sloping contact of the hogback ridges and the Zone I and Zone III fill. These features are well illustrated in Photograph 26.

The breach foundation has been geologically mapped and cross sectioned as shown on Figures 4-1, and 4-6 through 4-8. A detailed petrographic study of the foundation units is presented in Appendix C. Of greatest significance are the characteristics of the hogbacks forming Units 4, 5, and 6 and the pervasive vertical joint system accentuated by scour on rock surfaces. These features are discussed in paragraphs 4.3.1, 4.6.1 and 4.6.2 and are shown in section on Figures 4-6 through 4-8.

During the field investigations a number of significant features were observed, photographed, logged, and located as shown in Appendix G. Features observed and comments regarding their importance are as follows:

A. The weathered bedrock, particularly along gypsum beds, contains expanded bedding planes which are supported by small filaments or pillars of solutioned gypsum. This type of gypsum filling was also observed along some vertical joints. The expanded bedding planes will carry water allowing erosion and solutioning of gypsum, thereby enlarging the seepage conduit. This process will likely result in collapse of bedding in areas where waterflow has removed all or most of the filling material. This mechanism is likely responsible for sinkholes near the upstream and downstream toes and for the tension cracks observed in the Zone I material near the upstream toe of the dike.

B. Rock Units 2, 4, and 5 are differentially jointed. Exposures on the more competent units, such as 2 and 4, show the joints to be closely to widely spaced (1 to 10 feet). Many joints are open to waterflow; some are filled with erodible materials and some have geologically old solution channels.

C. Review of records and field observations confirms that there were large grout injections into Units 4 and 5. The nature of filling material along bedding planes and joints would create very difficult grouting conditions until the filling materials were eroded by reservoir waters.

D. Large quantities of grout were injected into vertical joints in Units 2, 4, and 5. Grout was observed in Unit 5 joints at least 80 feet upstream and 500 feet downstream of centerline.

E. Significant grout filling of solutioned joints occurred beneath the cutoff trench. Joint intersections also had solid grout plugs. The grout fillings were of such magnitude that fillers used during grouting could be identified.

F. Numerous locations of grout injections into Zone I and II materials were observed in trenches excavated through the embankment. There is no question that the embankment was hydraulically fractured by the drilling and grouting operation.

G. Piping holes were observed along the Zone I foundation contact from the upstream toe to near the cutoff. Similar holes were observed on the downstream side.

10.3 Results of Soil Testing

Soil testing after the failure indicated that materials were placed within the specified gradations and required compaction except for the filter drain material. This is not considered a serious defect as the samples may not have been representative and the minus No. 200 material was within specifications. The filter drain material was outside gradation requirements in the sand sizes. Zone III material, while meeting the specification requirements, had about one-third of its material in excess of 1-1/2 inches. This would mean that the matrix material may be less pervious than had been assumed.

10.4 Post Failure Embankment

Several areas of the embankment showed distress after failure. The distress was related to grouting, soil/rock interface seepage, and associated piping and dissolved gypsum contamination. The grouting distress was evidenced in the following forms:

10.4.1 Hydrofracturing of the Embankment Evidenced by Reported Cracking of the Crest: Hydrofracturing or other related movement of the embankment was noted during grouting operations on three occasions, Sept. 1, Sept. 5, and Nov. 11, 1988, when a crack running parallel to the dam crest in the vicinity of Sta. 6+50 was observed during grouting. The grouting operation was stopped and the cracking stopped. When the grouting operation resumed the cracking started again. The cracking was reported to be lateral spreading or hydrofracturing. Based on the grout stringers found in the post-failure embankment, it is clear that the embankment was hydrofractured from the core trench to the crest when grout was being injected.

10.4.2 Hydrofracturing or Other Methods of Contamination of the Embankment with Grout: Many hydrofracture zones were discovered in the post-failure embankment. These included grout emergence on the downstream slope of the dam. Hydrofracturing of the embankment in localized zones was found during excavation in the core trench in the vicinity of the soil/rock contact. Pockets and stringers of grout were found at various locations and elevations within the core trench. The downstream toe drain was completely grouted at several locations. Grout was also found along the soil/rock contact downstream of the centerline beyond the core trench. During excavation of the left abutment embankment, a grout stringer could be observed running from near the upstream crest of the dam, through the Zone I materials, and ending in the filter zone. The depth of the plane normal to the excavated slope was not determined. Grout pipes and grout pockets were seen on the left abutment failure surface. During grouting operations several large "takes" are noted when the grout pipes were raised above the rock contact. Grout was observed emerging on the downstream slope during grouting operations.

<u>10.4.3</u> **<u>Grouting of Downstream Filter Zones and the Downstream Toe Trench</u>**: Grout contamination of the filter zone, Zone III materials, and the downstream toe trench was found at several locations.

Grouting at pressures up to 30 psi gage in the embankment and at higher pressures (70 psi plus) in the rock mass near the soil/rock contact is the main contributor to the embankment contamination.

<u>10.4.4</u> **<u>Seepage and Associated Piping</u>**: Evidence of seepage and associated piping was observed in earth materials upstream and downstream of the dam at the soil/rock contact. These are documented by Photographs 34-39. It was not apparent if the piped materials were in-place overburden or compacted Zone I. During test trench excavation of the downstream area, several embankment pipes were found at the soil/rock contact. Some were in areas where rock overhangs appeared to have been backfilled with Zone III materials. Several areas including the downstream soil/rock contact and the downstream toe trench showed contamination with precipitates of calcite and dissolved gypsum. These are reported in Appendix C. Also piping in the rock surface immediately below the soil/rock contact appeared to have been caused by gypsum solutioning and removal by seepage along the fracture.

11. MECHANISMS OF FAILURE

The failure development was witnessed from nearly the outset of its surface expression till the point of rupture. As a result of the completeness and reliability of the observations, there is no doubt concerning the general nature of the mode of failure: Seepage waters passing under or through the dike or at the dike/foundation contact began carrying dike embankment materials. Through a process of backward erosion, the seepage and erosion process accelerated carrying more materials at a faster rate until caving occurred into the developed opening and the breaching process began.

11.1 Process of Subsurface Erosion by Piping

For the process of subsurface erosion by piping to occur, three conditions must be met:

A. There must be a flow of water through or adjacent to erodible foundation or embankment materials, and the gradient of the flow must be great enough to initiate erosion (that is, picking up and carrying away particles of embankment or foundation). More erosion-resistant materials require a greater erosive force (higher pressure and velocity).

B. The material in which the initial erosion takes place (embankment or foundation) must be capable of supporting a "roof" such that a "pipe" is formed to maintain the flow of water carrying suspended materials.

C. There must be an unfiltered exit or escape point for the material to be carried away.

The material being eroded may be carried away through a "pipe" of fixed size (nonerodible) or the "pipe" itself may continue to erode and allow an accelerated discharge. The rapid development of failure and the description of failure of Quail Creek Dike show that the pipe or tunnel was indeed enlarging during the failure process.

Although the general failure process is evident, there are several possible pipe paths that could produce the type of failure observed at Quail Creek Dike. For reference, general possible piping mechanisms are presented in Appendix F.

The Team investigated the materials in the dike and foundation and reviewed the records of seepage and grouting, the records of piezometers in the dike and foundation, and the as-constructed conditions. They made investigations and observations in the field to determine the most likely piping failure mechanism at Quail Creek Dike. The Team's conceptual ideas of the failure process reached on the basis of review of the design, construction, and performance information were confirmed in many respects by field evidence in and adjacent to the breach section. Based on these studies and observations, the Team was able to confidently identify the causes of failure and the specific mechanism of failure.

11.2 Adverse Factors Leading to Failure

As has been the case in many dam failure investigations, the investigation of the Quail Creek Dike failure revealed that several adverse conditions combined to result in the breaching of the dike. Many of the factors or conditions by themselves would not have resulted in failure of an embankment, but as juxtaposed here, they made the dike failure inevitable. The adverse factors included:

A. Open jointing was initially present in the foundation. There was seepage at the toe of the dike immediately upon the initiation of reservoir filling. This condition, which was not foreseen in the design concept, indicated that open joints with direct connection to the reservoir were carrying water under the dam with little head loss. The presence of the water flowing in joints is not necessarily a problem, but at this site it resulted in several adverse effects:

    1. The open joints provided a path for flowing water to accelerate the process of solutioning and differential erosion along joints and bedding planes in the foundation.

    2. The open joints provided a concentrated source of water to accelerate weakening of the weathered rock on the walls of the cutoff trench, the unconsolidated overburden left on the slopes of the rock ridges existing upstream and downstream of the cutoff trench and unconsolidated overburden on the foundation of the dike upstream and downstream of the cutoff trench (see Photograph 8). The concentrated source of water also locally weakened Zone I, Zone II, and Zone III materials.

    3. The open joints produced an unwanted effect (excess seepage) that resulted in two remedial responses that contributed to the overall failure mechanism development: (a) the construction of an unfiltered toe drain, and (b) the implementation of extensive grouting programs.

B. Unprotected erodible materials existed or were placed at unprotected contacts along the interface of the dike and foundation.

    Materials in the cutoff trench at the dike/foundation contact which could normally be considered erodible included weathered rock, unconsolidated overburden, and Zone I. In addition, as illustrated in Photograph 8, the as- built dike incorporated rock ridges into the construction. The valleys between these rock ridges were filled with unprotected, erodible Zone I material in order to provide a level surface on which to begin placement of the zoned dike. At many locations, this resulted in Zone I material being placed continuously from upstream to downstream. Although the Zone I itself was quite impermeable, the placement of this material adjacent to erodible and/or solutionable rock units placed the material in jeopardy of being locally saturated and piped through the rock or forming a pipe itself. This condition was exacerbated by orientation and stepped and sloping existing rock faces (see Photographs 25-27).

C. Grouting of the fractures and joints closed many of the subsurface drainage paths through the foundation.

For subsurface erosion of the embankment materials to occur, there must be a flow of water strong enough to pluck the material from its matrix and carry it away. If the foundation of a dam behaves as a drain, then water is carried away from the contact of the dam and foundation. In the case of Quail Creek Dike, the cumulative effect of the grouting program was to reduce generalized seepage to a low value (about 0.1 cfs) and as a consequence to raise the pressure and increase the gradient at the dike/foundation contact. This effect is apparent in piezometers 16, 17, and 18 during and after the second grouting period (see Figures 7-7, 7-8, and 7-9).

11.3 Most Likely Failure Mechanism

The review Team considers that the following scenario is the most likely mechanism of failure. Seepage began to flow through the foundation immediately upon filling, and this flow began the erosion of joint fillings and produced some solutioning. The water began softening and eroding the materials at the dike/ foundation contact along joints and along the more permeable bedding planes. Water entered the foundation through the reservoir basin and also entered the foundation from the upstream portion of the dike along the dike/foundation contact, passed along or beneath the cutoff trench and exited in part along the dike/ foundation contact (see Photographs 34-37).

During reservoir operation some concentrated seepage conduits developed through the foundation. In one reported case the conduit exited at the contact of a bedding plane on a rock ridge and the Zone I and overburden materials near the downstream toe. The water exiting under pressure caused local saturation of the contact materials and surface erosion (boils) in the Zone I and Zone III. These concentrated seepage occurrences resulted in grouting efforts to shut them off.

The grouting program successfully closed off concentrated seepage conduits and reduced downstream relief of the upstream source of water into the foundation. Pore pressures increased at the dike/foundation contact. Subsurface seepage continued under higher pressures with undetected localized zones of piping occurring at the dike/foundation contact. A network of erosion tunnels developed at the interface of the dam and foundation both upstream and downstream. Some of these erosion tunnels were observed in the remnant of the breach section (see Photographs 34-39). The flow path for most of these tunnels passed into foundation rock and under the cutoff trench and did not carry away significant amounts of embankment materials.

However, one or more erosion tunnels at the rock/embankment interface near the observed point of seepage on December 31, 1988, began the process of undermining the embankment and ultimately encountered and began carrying significant quantities of embankment materials (see Figure 11-1). The embankment materials as well as the rock/embankment contact materials were likely carried away gradually at first, dropping the eroded material in the downstream toe trench. The initial development of this critical erosion tunnel could have taken a considerable amount of time. The erosion tunnel development continued along the contact of the weathered rock foundation materials, the overburden, and the Zone I. The tunnel or pipe likely developed initially in the overburden material or opening in the bedding plane and then expanded in size in the Zone I. Zone I was observed in the Team's investigation to be able to support a piping roof (Photograph 45-46). In the critical erosion tunnel, a path through or near the cutoff trench had to develop which allowed the pipe to ultimately enlarge rapidly and carry the volume of water required to cause the observed failure response.

One uncertainty in the scenario is the precise means by which the subsurface erosion process crossed under, along the base of, or through the cutoff trench. This is of interest because the materials in the trench were observed to be unsaturated except at locations close to the contact. Three means considered by the review Team are:

A. Certain areas of the cutoff trench were relatively shallow (less than 10 feet in depth). The backward erosion process could have passed just below the cutoff trench through a zone of weak or erodible rock at one of the low spots. As the erosion in the tunnel increased in size, the embankment materials would have been contacted and thus been in the path of the erosion. The Zone II purple clay was observed in the discharge waters during the failure.

B. It is possible that the water pressure testing or grouting of the embankment could have caused hydraulic fracturing, creating a fractured plane across the cutoff trench that was not subsequently grouted.

C. Although the zones I and II were observed to be impermeable and "dry" in the remnant, it is conceivable that locally the embankment materials had become saturated due to a high-volume, high-pressure flow of water along a joint. This saturation was observed in the Zone I near the downstream toe seepage in 1987. The post-failure excavation also revealed such occurrences. Thus locally a flow could have been occurring through the Zone I materials, and the erosion tunnel could have crossed the cutoff entirely through the Zone I materials.

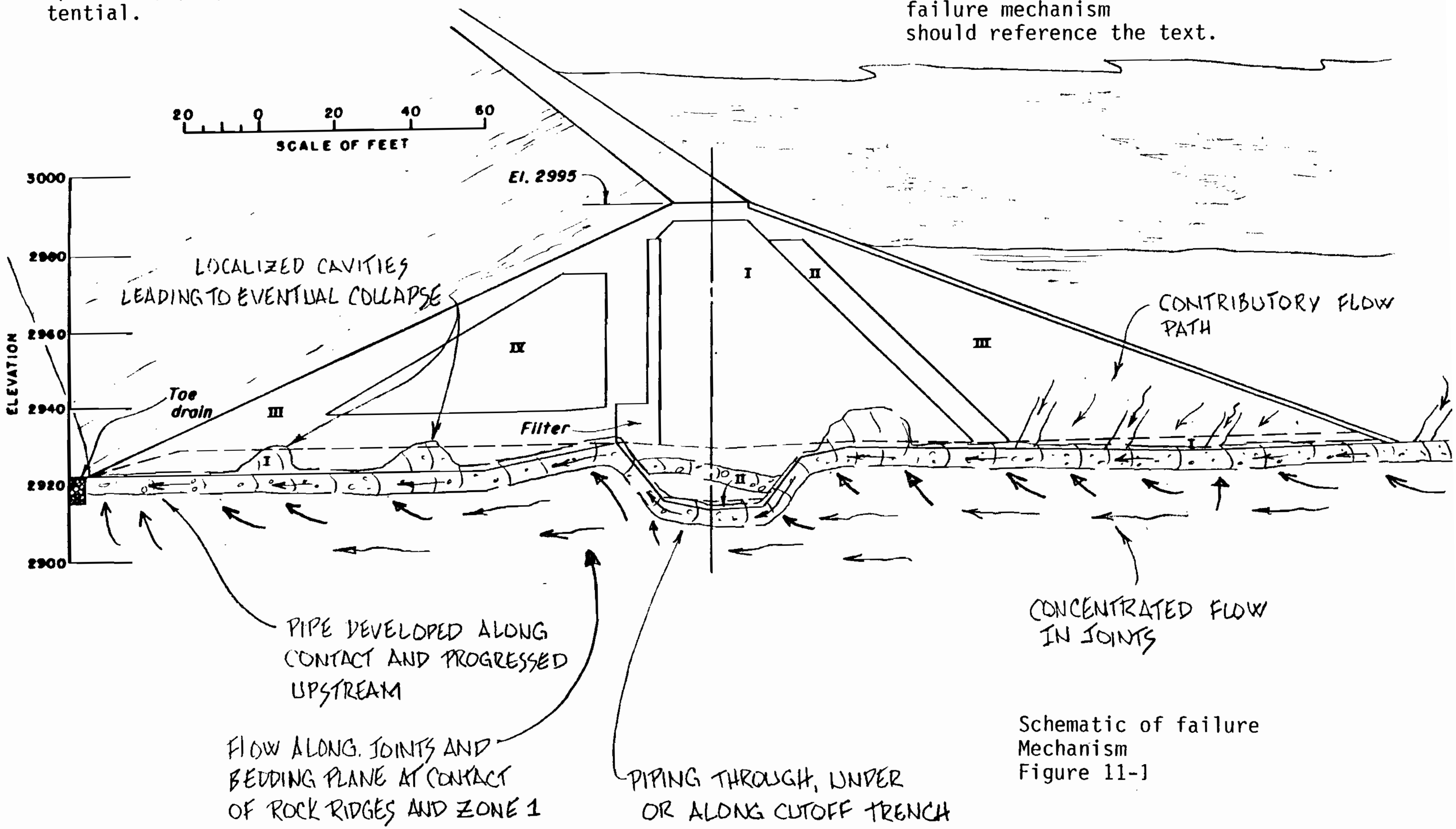
Based on the Team's field observations, the flow along a joint causing weakening and erosion susceptibility of the contact along the base of the cutoff trench seems the most likely possibility. However, any of the above processes were possible, and it is clear that a pipe through or along the base of the cutoff did develop.

Once the pipe reached a direct source of water, the erosion within the tunnel accelerated rapidly allowing more materials to be carried out which allowed more water to enter, etc. This process created an increasingly larger unsupported cavity through the dike at the dike-foundation contact (see Figure 11-2). Collapse of the dike into the erosion cavity occurred incrementally from downstream to upstream, but the force of the flow from the upstream was great enough to erode the caved material.

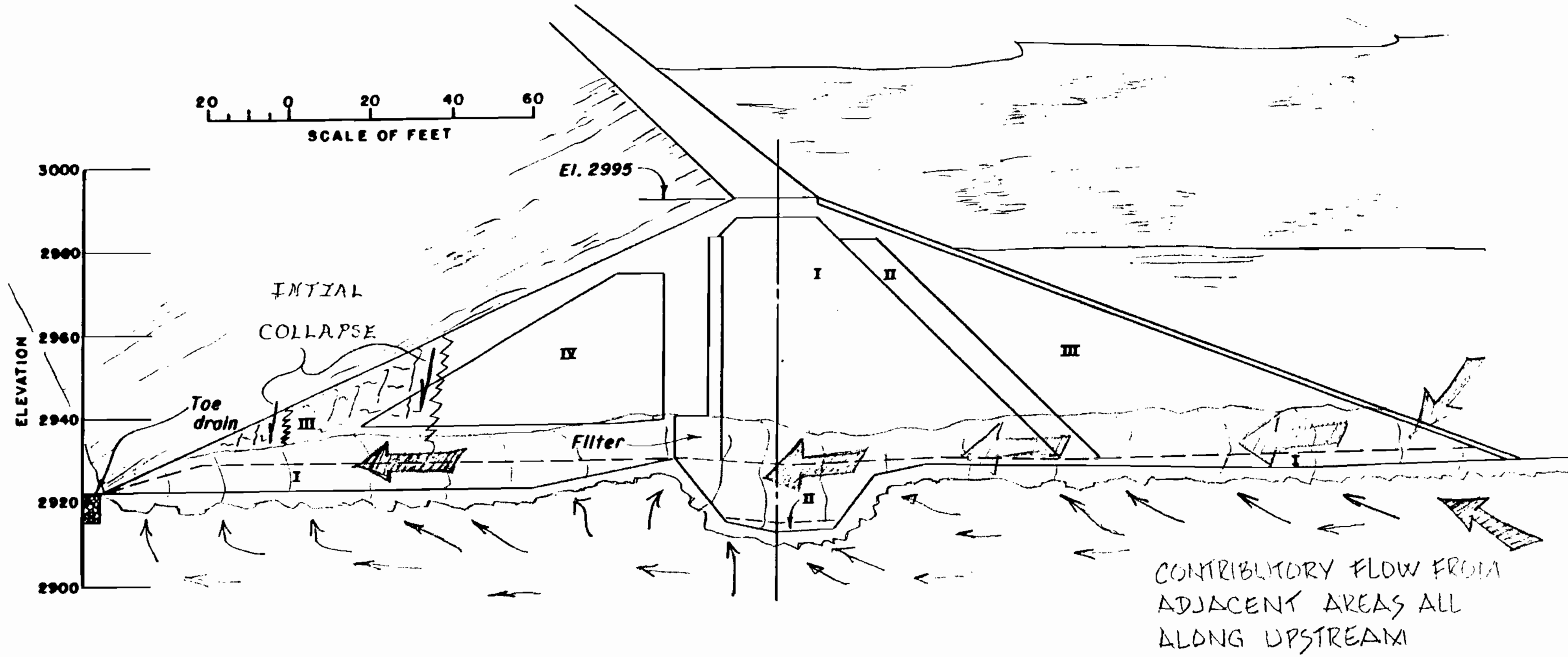
The failure mechanism can never be known with absolute certainty when the breach carries the evidence away. However, the direct observations of evidence at the site leave the Team with little doubt as to the validity of the failure mechanism described.

As piping progresses upstream the hydraulic gradient increases, as the hydraulic gradient increases flow volumes and velocities increase upstream piping potential.

NOTE: This schematic is presented to aid in visualization of a failure mechanism. Embankment details and failure zone representations are provided for illustration only. All technical discussions concerning the Quail Creek Dike failure mechanism should reference the text.



Schematic of failure Mechanism
Figure 11-1



NOTE:

This schematic is presented to aid in visualization of a failure mechanism. Embankment details and failure zone representations are provided for illustration only. All technical discussions concerning the Quail Creek Dike failure mechanism should reference the text.

Schematic developed stage of failure
Figure 11-2

12. CONTRIBUTIONS OF OTHER FACTORS TO THE FAILURE

Many factors may have led or contributed to the development of subsurface erosion at the site. Some of these factors set the stage that permitted the subsurface erosion to occur. Other factors affected the timing of when the failure would occur. The areas of the embankment where distress may have occurred are limited to the soil/rock or compacted fill/rock interface by the results of the post-failure visual, and laboratory examinations of the remaining embankment and the review of design and construction documents. The embankment above the soil/rock interface appears to have performed well despite having been hydrofractured by several cycles of grouting at the compacted fill/rock interface (core trench). Several factors that may have allowed seepage can be postulated in the vicinity of the failure zone stationing. The major adverse factors leading to failure were discussed in the previous section. Other design or remedial treatment factors which may have had some contribution to failure are discussed below.

12.1 Design Factors

12.1.1 Differential Settlement Caused by a Narrow Cutoff Trench: As was considered in the post-failure examination of Teton Dam, Wyoming, the possibility of cracking due to differential settlement across the cutoff trench can be envisioned. The cracking would be exacerbated in areas such as Sta. 7+50 where the trench was excavated to a depth of 23 feet, 13 feet beyond the designed depth. The width of the trench was reduced to 19 feet at Sta. 7+50. The steepened side slopes required to reach the greater depth would also have increased the influence of the discontinuity. The stiff core soils, Zone I sandy silts, could have cracked and allowed communication of water across the trench zone from trench wall to trench wall. No evidence to support this mechanism was found.

12.1.2 Differential Settlement Caused by Steep Sloping Surface on the Floor of the Cutoff Trench: Construction photographs show that steep rock surfaces were excavated on the floor of the cutoff trench and trench walls where rock layers were encountered. While a positive slope was prepared on the steep surfaces, photographs show that a bench of perhaps as much as 4 to 5 feet may have been excavated. Differential settlements or relief can be envisioned which may result in a void between the rock and soil in these areas allowing seepage. While the clay may have protected the fill, seepage volumes would increase and possibly carry off downstream overburden or shell material. No evidence to support this mechanism was found.

12.2 Remedial Measures

12.2.1 Hydrofracturing: The soil/rock contact and integrity of the compacted fill in the cutoff trench above the interface were certainly impacted by hydrofracturing. The introduction of water or grout under high pressure separating the earth materials provides several scenarios for seepage increase. The first is simply that a discontinuity is formed which tends to gather and concentrate flows at the grout/soil interfaces. The second is that since a hydrofracture with corresponding large cement takes is usually followed by a plant shutdown to evaluate the take, the hydrofracture is allowed to close without being fully grouted. Since grouts are usually high in water content, after the water is absorbed a weakened zone along the hydrofracture may be formed. Such a weakened zone could have provided a flow path across the cutoff trench.

12.2.2 Toe Drain: The toe drain installed after completion of the dike likely contributed to the mechanism of failure. The drain was installed without a filter to prevent migration of fines. The trench providing near-dike relief of seepage waters increased the flows by shortening the seepage path. The increased flows would carry additional materials and increase the solutioning of the foundation gypsum. Gypsum was found along the excavated toe trench. The use of an unfiltered uniform size rock allowed the fines to wash into the drain and promoted upstream piping. The toe drain was not the cause of failure but may have hastened the process.

12.2.3 Upstream Cutoff: In order to reduce the amount of water being carried under the dike foundation, an additional cutoff trench upstream of the dike was cut and filled with Zone II material, and a surficial blanket of Zone II was laid from the cutoff trench to the upstream toe of the dike. This work was accomplished during the late fall of 1986. Although some of the underseepage flow paths from the reservoir may well have been blocked as a result of this fill placement, there is no evidence that overall seepage was reduced, and the cutoff did not prevent the inflow that produced the concentrated leaks later in the operation or the flows that led to the failure. On the other hand, there is no evidence to indicate that the upstream cutoff trench had any adverse effect on the dike operation.

13. CONCLUSIONS

13.1 General

A detailed discussion of possible modes of failure, the effects of various design, construction, and remedial measures on the failure, and the Team's conclusion as to the most likely mode of failure have been provided. From this, the Team has drawn a number of conclusions, which follow under appropriate headings.

13.2 Primary Conclusion

Failure resulted because embankment materials placed on the foundation, including the overburden left in place, were not protected from erosion by seepage moving along the foundation contact. If the materials had been protected by proper filters, drains, and foundation surface treatment, the failure would not have occurred. This is not a new lesson, but rather a lesson relearned and reinforced. While proper defensive design measures would have prevented failure, the solutioning of gypsum by continuing foundation seepage might eventually have led to unacceptable volumes of seepage from an economic standpoint. Understanding the exact means by which seepage reached and moved along the contact to erode susceptible, unprotected materials is of considerable interest but is not essential to design and construct a safe dam at this site.

13.3 Secondary Conclusions

A. The very early assumption that there would be little or no seepage through the dike foundation below the shallow cutoff was not valid and had a profound effect on design of seepage erosion protection. If true, there would have been little seepage along the contact and no need for defensive measures. The potential for seepage is often difficult to anticipate and evaluate, and in this case the initial reservoir filling showed rapid movement of seepage through the foundation. Several factors influenced seepage through the dike foundation, including the following:

(1) The dike was constructed on thinly bedded, highly gypsiferous sediments which had bedding striking upstream and downstream and a moderate dip of 5° to 25° toward the left abutment. These conditions required special consideration in design of the foundations.

(2) Fractures in the form of three major near-vertical sets of joints were present in the foundation. Pre-design reports identified the joint system, but foundation exploration was not complete enough to fully detect potential seepage problems associated with the vertical joint system.

(3) Expanded bedding is common in weathered horizons of the foundation. Where gypsum-rich sediments occur, the expanded planes are often supported by filaments or pillars of solutioned gypsum. These bedding planes can conduct water

which would solution more gypsum, causing a larger passage for seepage.

The foregoing factors lead to several general conclusions.

(1) Vertical, small-diameter exploration in a complexly folded, vertically jointed anticline with beds striking perpendicular to a dam axis does not provide an adequate evaluation of foundation permeability.

(2) The decision to use a shallow cutoff did not consider the effects of an open joint system in conjunction with expanded bedding planes trending upstream and downstream and geologically old solution channels in the foundation.

(3) The decision to strip only 1 foot of overburden from portions of the foundation upstream and downstream of the cutoff left a highly fractured, pervious rock mass with numerous expanded bedding planes capable of rapidly transmitting considerable seepage along the foundation contact.

(4) The decision to leave upstream downstream trending hogback ridges after stripping the foundation downstream of the cutoff allowed pervious fractured, weathered rock adjacent to residual overburden and Zone I embankment. This geometry provided open conduits for water along the contact on each side of the ridges.

B. The presence of considerable gypsum in the foundation was not the immediate cause of failure since solutioning requires that water move through open fractures, joints, and bedding planes. However, as time passed, the solutioning of gypsum near the unprotected foundation contacts could have increased the volume and velocity of seepage near the contact and thus hastened the seepage erosion process.

C. The remedial grouting did not prove to be a long-term solution for this particular foundation as demonstrated by the shifting locations of seepage emergence during grouting and sporadic outbreaks of new seepage after each episode of remedial grouting was completed. Review of the grouting activity leads to these conclusions:

(1) Grout curtains in a formation where potential seepage paths (joints, fractures, etc.) are filled with either erodible or soluble materials are not permanent and require periodic maintenance grouting to remain effective. Grouting was successful in plugging some conduits, but forced the water to find other channels in which filling materials were eroded and solutioned, ultimately allowing increasing seepage.

(2) Grouting against reservoir head as at Quail Creek Dike assures that materials used in grouting will move downstream in the direction of flow. While much of the material may have passed

through the foundation, there is piezometric and field evidence that remedial grouting restricted downstream exits and caused increasing hydraulic pressure against the embankment/foundation contact.

(3) Grouting through embankments is not desirable but often necessary in remedial work. In this case the records and field evidence indicate that drilling and pressure grouting in the embankment, foundation contact, and foundation immediately below the contact caused hydraulic fracturing of the embankment. Whether or not this contributed to the failure is speculative, but does show that caution must be exercised in grouting near the foundation contact and particularly where the grout pipe is pulled above the contact and the embankment is directly exposed to pressure grouting.

D. Zone I materials proved to be adequately impervious, and there is no evidence of significant seepage through the embankment or that the soluble salt content in some Zone I material was detrimental. It would be preferable to use plastic clays in a Zone I if readily and economically available. If not, then the materials used in Quail Creek Dike are satisfactory if adequately protected against internal erosion. These materials are generally brittle when well compacted and thus subject to cracking and fracture. In regard to specific uses in Quail Creek Dike, there are two conclusions:

(1) It is common to use limited volumes of Zone I or other readily available material to level local irregularities in a stripped foundation prior to placing the zoned embankment. However, the top of rock configuration at Quail Creek Dike is such that rather deep and wide depressions extended from upstream to downstream. The as-constructed sections typified by Figure 2-5 show that these depressions were filled with Zone I materials to a depth of 10 feet or more on an untreated, unfiltered contact and over an area 50 feet or wider before placing the zoned embankment. In essence, a zoned embankment was perched on an erodible foundation from upstream to downstream.

(2) The purple Zone II clay examined in place has a high percentage of unbonded, discrete shale particles and thus is more friable and less resistant to erosion than a typical plastic clay. Therefore, the purple clay may not have provided the extra protection intended in its placement at the base of the cutoff trench.

E. Filters are widely accepted defensive measures against internal erosion. To be effective, filters must be carefully designed and constructed to ensure that materials do not move through contacts or zone boundaries. Field examination of materials and remedial measures at Quail Creek Dike warrant the following conclusions:

(1) In several locations the downstream toe drains consisted of perforated corrugated plastic drain pipe (no filter fabric sleeve) surrounded by coarse concrete-aggregate-size crushed rock. Filter criteria were not met, and the aggregate and pipe had been penetrated by eroded fine- grained material. Thus, in essence, the toe drain may have accelerated (but did not cause) failure by providing a closer uncontrolled exit for eroded materials than the original uncontrolled seepage exits.

(2) The pit-run Zone III sandy gravels observed in place were far from homogeneous as might be expected. Consequently, a wide range of permeability and flow capacity would be expected. The variation of material observed in place while meeting the specification may not meet filter criteria. Consequently, materials may erode and migrate until a natural filter forms. In other cases the material may not be permeable enough to provide adequate flow capacity.

F. Quail Creek Dike was sparsely instrumented, but based on the assumption of little or no anticipated foundation seepage and the height of the structure, the original installation was reasonable. After serious seepage problems developed, the limited piezometer data available show that remedial grouting tended to restrict seepage exits and increase the water pressures under the dam. Additional carefully located and installed piezometers could have been useful in developing remedial seepage control. It is doubtful that additional piezometers would have provided much advance warning of the failure. The seepage flows of primary interest were gathered and measured reasonably well, even though there are some gaps in the data. Data are available for the chemistry of dissolved solids in samples of seepage from various locations, but comparable data are not available for reservoir water near the dike. Comparison of reservoir and seepage samples would provide a qualitative insight to foundation solutioning.

G. There is no indication that seepage through the dike embankment or the quality of its construction contributed to the failure.

PHOTOGRAPHS

1. Aerial view south showing key trench and footprint of dike (12-16-83)

2. View west along key trench from left abutment (1-6-84)

3. Sweeping floor of key trench at 17+00 (1-6-84)

4. View east in key trench at hogback about 17+60 (1-6-84)

5. Cleaning key trench at about 17+50 (1-6-84)

6. Weathered gypsiferous siltstone with open bedding planes, south wall of key trench, Sta. 3+16, 10 feet below surface (12-16-83)

7. View west showing Zone I being placed in key trench and in hollows beside trench (1-12-84)

8. View west past grouting operation on left abutment showing placing of fill (2-8-84)

9. Blanketing upstream from center of dike (10-30-86)

10. View eastward from left abutment showing zones in fill (4-84)

11. Sewer rock dumped on spring at toe of dike, somewhere between Sta. 7+00 and 8+00 (8-3-87)

12. Breach from downstream at distance

13. Breach from downstream

14. Breach from upstream

15. Erosion downstream of dam showing dip of beds into left abutment

16. Right abutment of breach after excavation of the cutoff trench

17. Major joint system in units 4 & 5 at site. North 20 degrees West set trends up and down and North 29 degrees East set trends across picture

18. Jointing in breach section

19. Close up of erosion and solutioning along joints

20. Joint pattern in breach area

21. Joint exposed in breach section

22. Joint with filling downstream of breach

22b. Open joint with infilling of readily erodible and gypsum rich soluble material

23. Bedded sequence of Shnabkaib member at dike. Bedded units indicated on photo

24. View of bedding looking downstream from breach section

25. Erosion along bedding in breach area

26. View of bedding showing variability of erodibility within a rock unit

27. View of openings in bedding in breach area

28. View of openings in bedding in breach area

29. View of openings in bedding in breach area

30. View of openings in bedding in breach area

31. View of geologically old solution channels in upstream portion of the breach

32. Example of expanded bedding planes with gypsum filaments in unit 4 located approximately 800 feet downstream of dike

33. Upstream cutoff trench

34. View of erosion channel near contact of dam and foundation downstream near Sta. 4+50

35. Close up of downstream channels

36. Close up of downstream channels

37. View of erosion channels upstream near Sta. 5+50

38. View of erosion channels upstream near St. 5+50

39. View of erosion channels upstream near St. 5+50

40. View of depression in remaining fill, upstream shell area

41. View of cutoff trench exposed in breach section. Zone II embankment was present at base of cutoff trench

42. View of left embankment showing zoning remaining following sloping of breach side

43. View of right embankment showing zoning remaining following sloping of breach side

44. View from top of remaining right embankment showing post failure exploratory trenches

45. Piping channel found in exploratory trench upstream of dike centerline on left side of breach

46. Piping channel found in exploratory trench upstream of dike centerline on left side of breach

47. Drain pipe encountered in exploratory trench at original downstream toe of dam

48. Purple clay (Zone II) in bottom of exploratory trench at centerline on left side of breach

49. Grout filling crack noted in exploratory trench on centerline of cutoff on left side of breach

50. Grout contamination in Zone I materials on right side of breach

51. Grout extending through Zone I and Zone II on right side of breach at centerline of cutoff trench

52. Grout intrusion through entire depth of cutoff trench on right side of breach

53. Grout intrusion between unit I and Zone I fill

54. Grout plug in Zone I on right side of breach

55. Grout intrusion through Zone II in cutoff trench on left side of breach

56. Grout intrusion into Zone I in cutoff trench on left side of breach

57. Grout filled seam through Zone I on left side of breach

58. Grout filled toe drains

59. Grout filled toe drains

60. Grout filled toe drains

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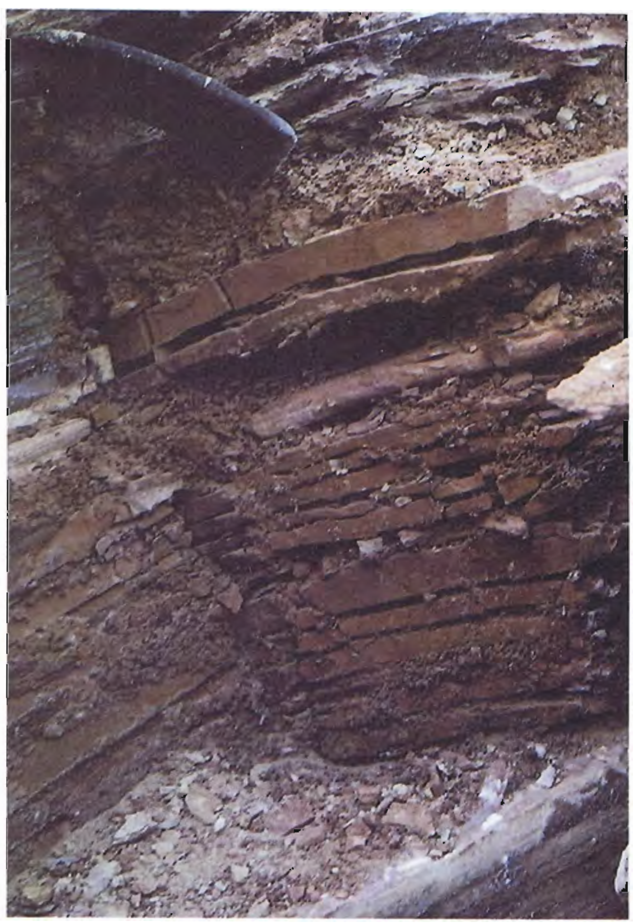
5. Cleaning key trench at about 17 + 50. (1-6-84)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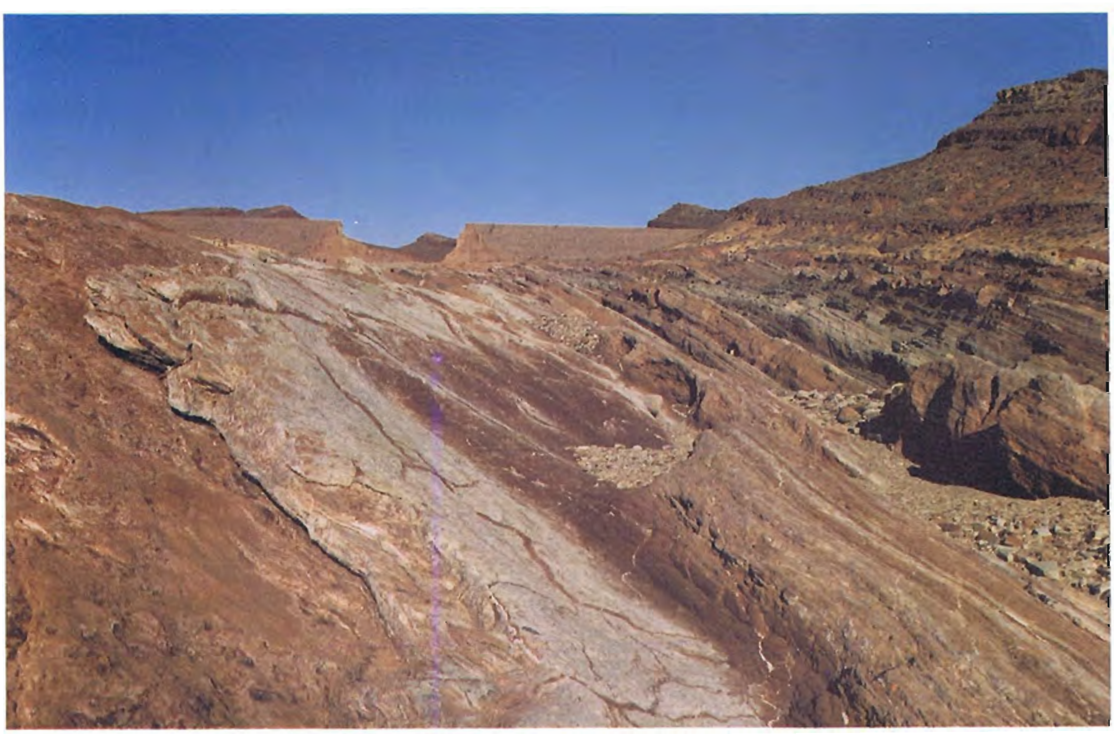

12. Breach from downstream at distance.

13. Breach from downstream.

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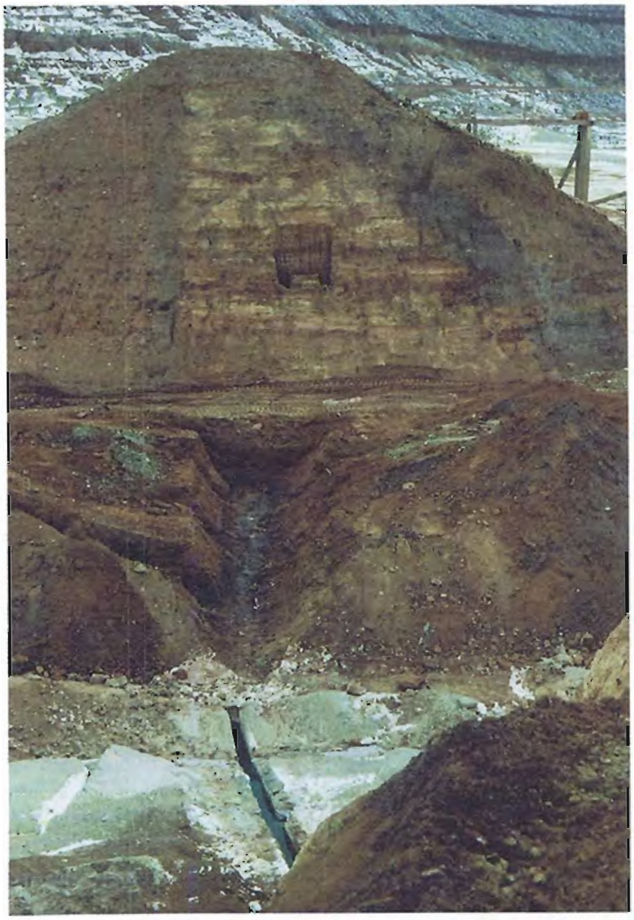
15. Erosion downstream of dam
showing dip of beds into left abutment.



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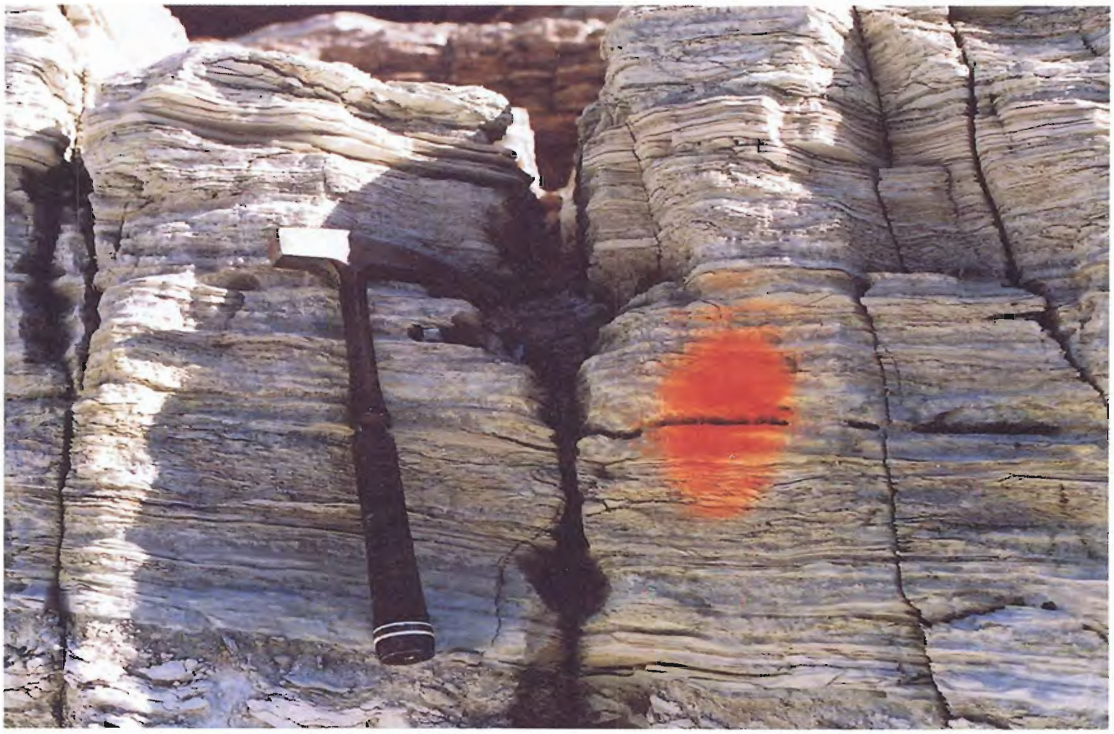
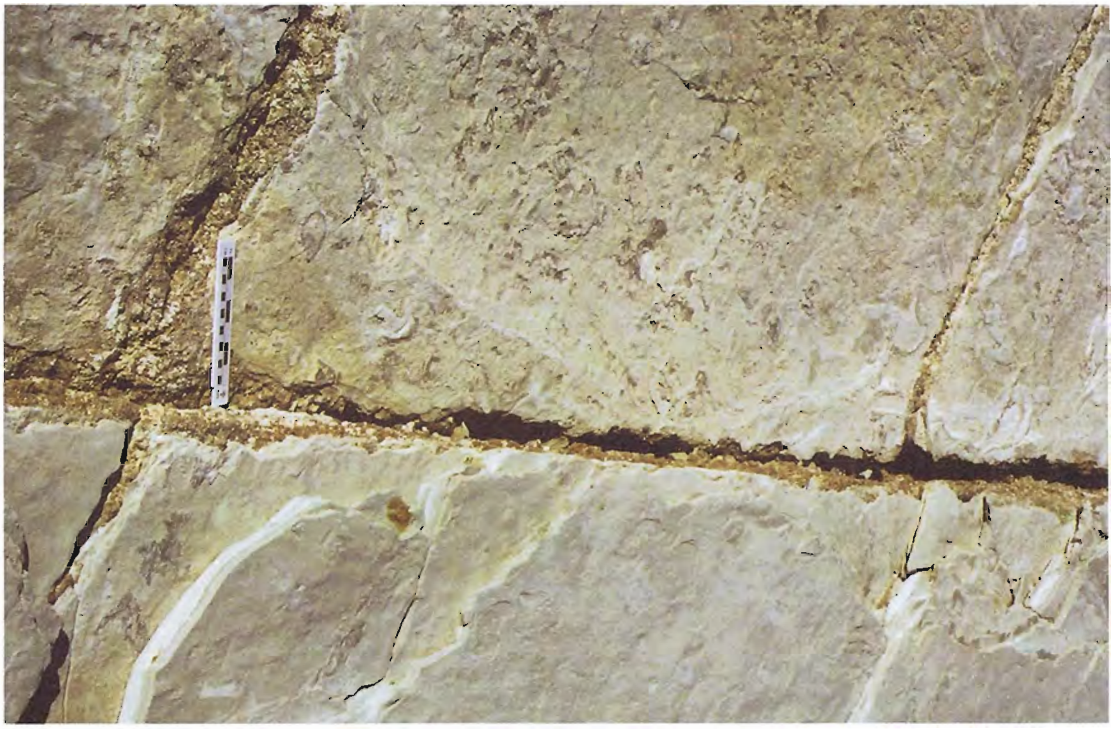


19. Close-up of erosion and solutioning along joints.

20. Joint pattern in breach area.

21. Joint exposed in breach section.

22. Joint with filling downstream of breach.



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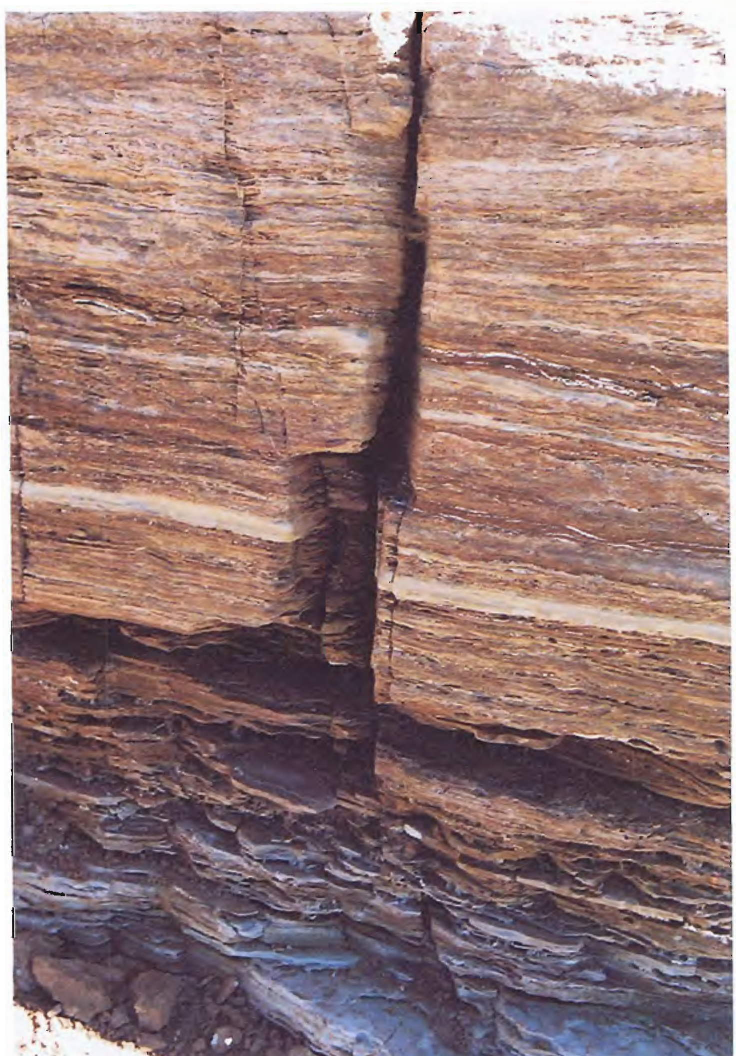
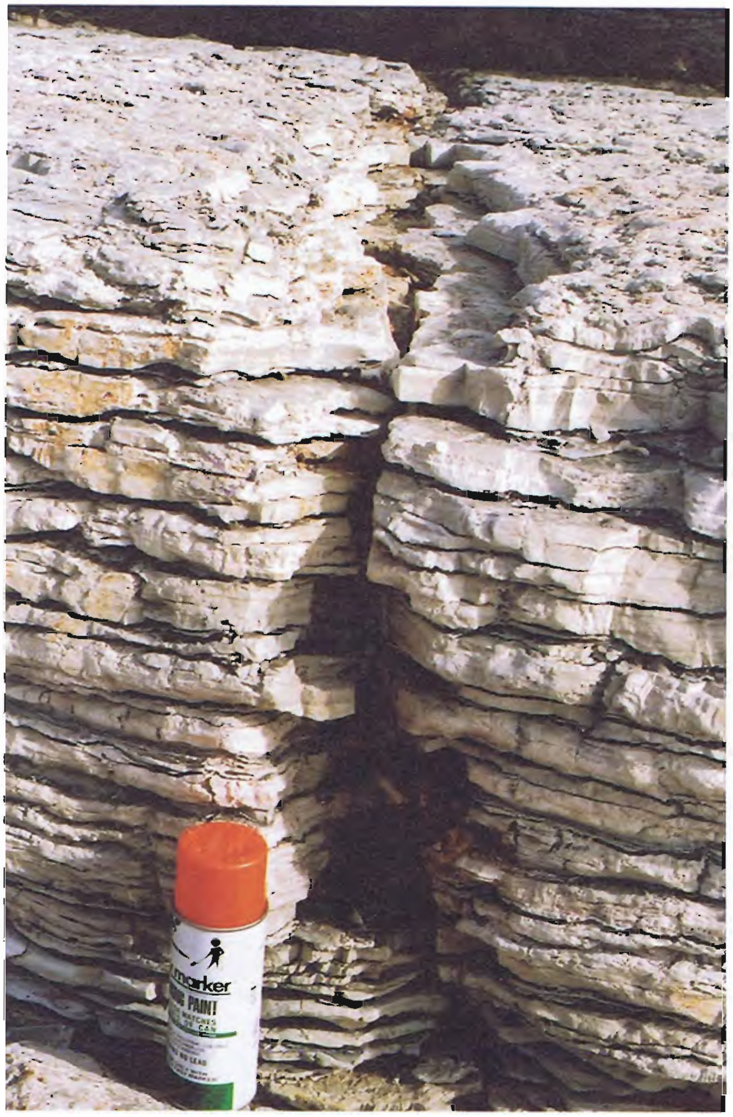


29. View of openings in bedding in breach area.



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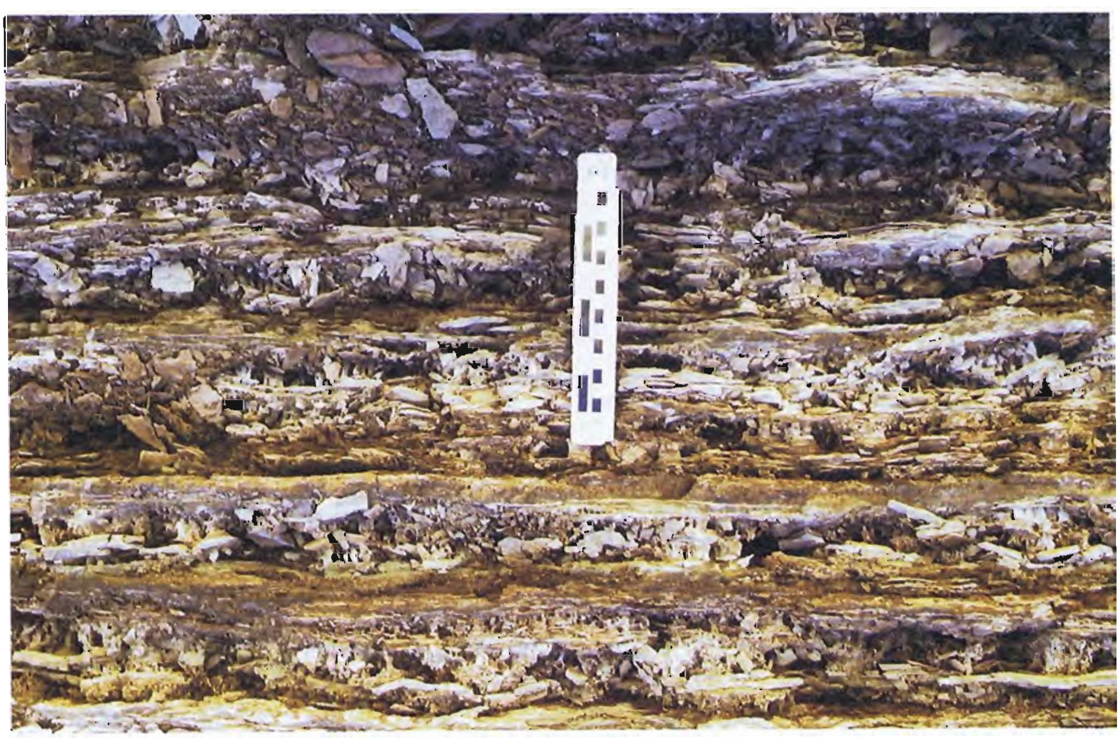

32b. Expanded and differentially eroded beds with high gypsum content.

33. Upstream cutoff trench.

34. View of erosion channel near contact of dam and foundation downstream near station 4 + 50.

35. Close-up of downstream channels.

36. Close-up of downstream channels.

37. View of erosion channels upstream near station 5 + 50.

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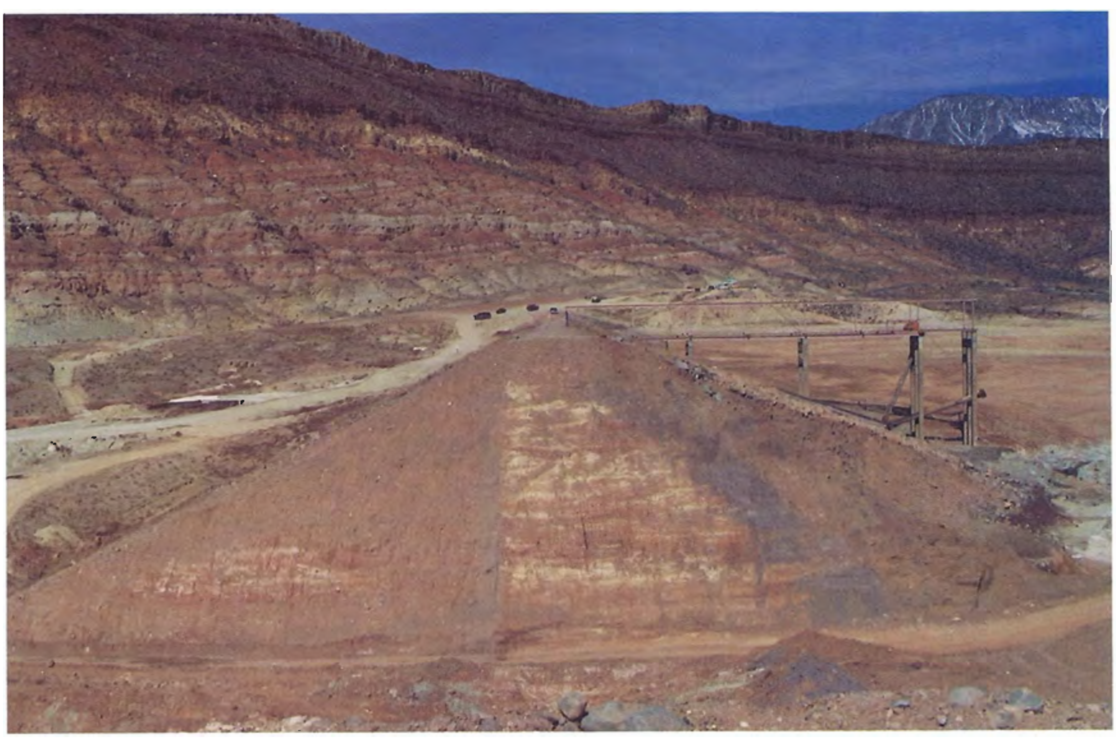

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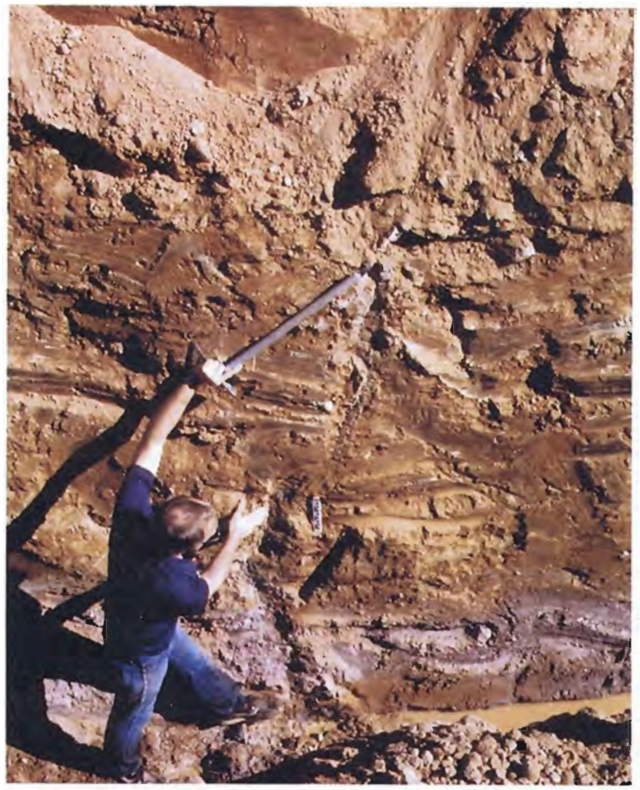

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trench on centerline of cutoff on left side of breach.

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materials on right side of breach.

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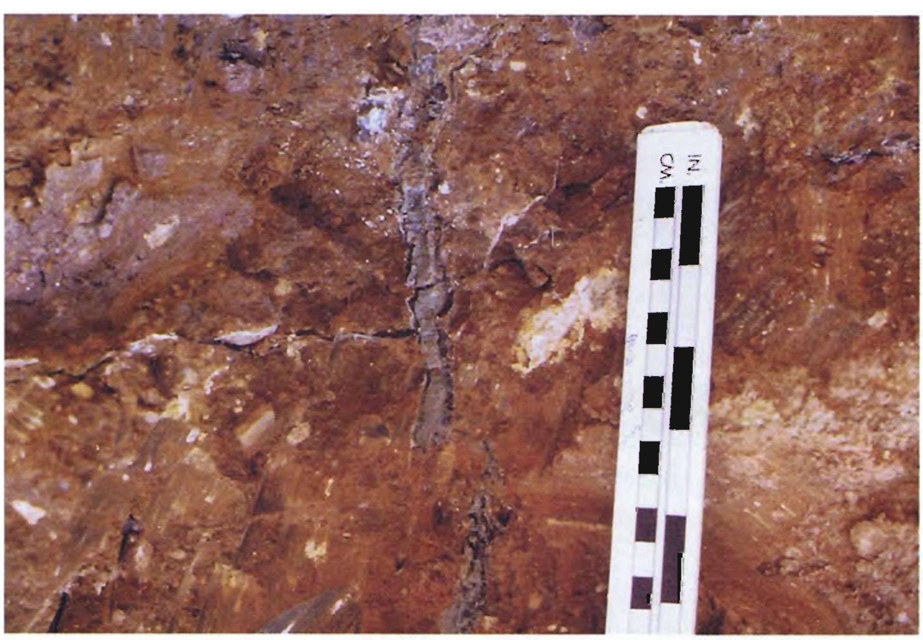
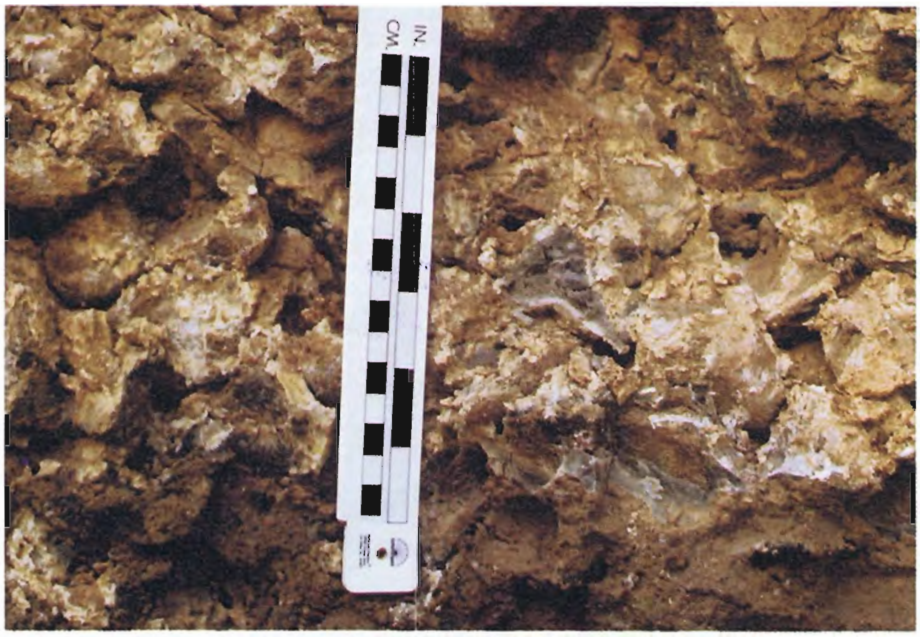
in cutoff trench on left side of breach.



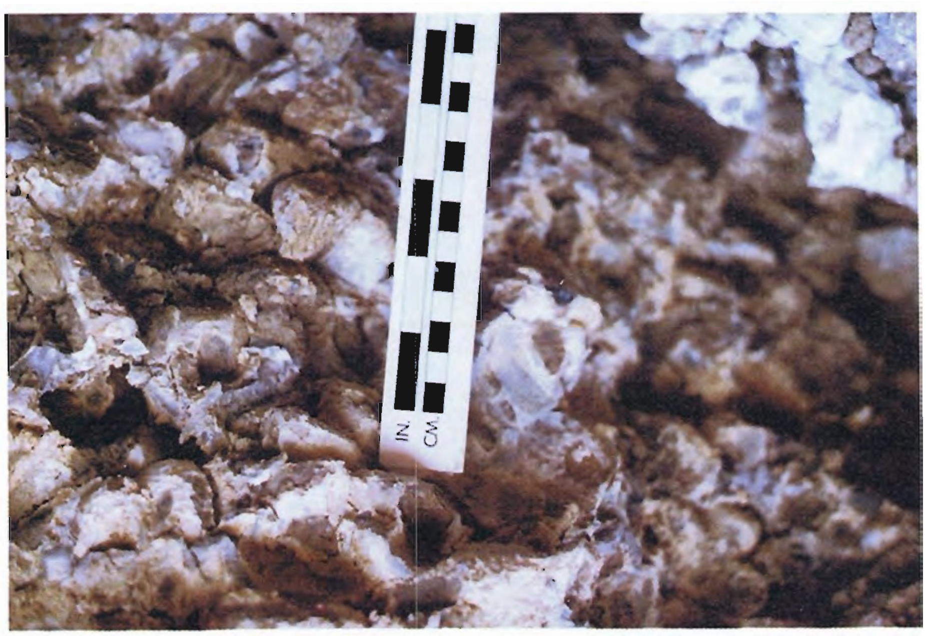
56. Grout intrusion into Zone I
in cutoff trench on left side of breach.

57. Grout-filled seam through Zone I on left side of breach.

58. Grout-filled toe drains.



59. Grout-filled toe drains.



60. Grout-filled toe drains.

APPENDIX A

List of Provided Materials and Individuals Interviewed

LIST OF PROVIDED MATERIALS

1. Final Design Report - Quail Creek Dam & Dike, by Rollins, Brown and Gunnell, Inc., August, 1983
2. Geotechnical Investigation and Design Analysis for Quail Creek Dam, by Rollins, Brown and Gunnell, Inc., March, 1983
3. Specifications for Quail Creek Dam & Dike, by Rollins, Brown and Gunnell, Inc., September, 1983
4. Quail Creek Dike Grout Holes July, 1987 through November, 1988, by Rollins, Brown and Gunnell, Inc.
5. Quail Creek Dike Water Pressure Tests - 1986, by Rollins, Brown and Gunnell, Inc.
6. Quail Creek Dike Excavation Sections, by Rollins, Brown and Gunnell, Inc.
7. Quail Creek Dike Final Embankment Cross Sections and Quantities, by Rollins, Brown and Gunnell, Inc.
8. Quail Creek Dam and Dike Piezometer Readings for 1985-1986 Water Year, by Rollins, Brown and Gunnell, Inc.
9. Quail Creek Dam and Dike Piezometer Readings for Water Year 1987-88, by Rollins, Brown and Gunnell, Inc.
10. Quail Creek Dam and Dike Piezometer Data, 1985 to Present, prepared by the Utah Division of Water Rights
11. As Constructed Plans for Quail Creek Dam and Dike, by Rollins, Brown and Gunnell, Inc.
12. Plans and Specifications for Quail Creek Dike Foundation Pressure Grouting, by Rollins, Brown and Gunnell, Inc., April, 1986
13. Specifications for Quail Creek Dam Blanket Drain and Berm, by Rollins, Brown and Gunnell, Inc., April, 1986
14. Coordinates of Seeps, Drains, and Measuring Flumes, January 24, 1988, by Rollins, Brown and Gunnell, Inc.
15. Log of Borings for Quail Creek Dike, October, 1986, by Rollins, Brown and Gunnell, Inc.
16. Photographic Log of Exploratory Drill Holes for Quail Creek Dike, furnished by Rollins, Brown and Gunnell, Inc.

33. Inspectors' Diaries and Quality Control Testing for Dike Construction and Remedial Measures, November, 1983 through November, 1988

34. Bradford Price's Diary, of Rollins, Brown, and Gunnell, Inc., November, 1983, through November, 1988

LIST OF INDIVIDUALS INTERVIEWED

Informal discussions were held with the following individuals. No formal record was made of these interviews.

Ron Thompson	Washington County Water Conservancy District Manager and Legal Counsel
Don Stratton	Stratton Brothers
Omar Matthews	L and M Construction
Lloyd Jessop	Washington County Water Conservancy District - Project Operator/Dam Tender
Dr. Ralph Rollins	Principle - Rollins, Brown and Gunnell, Inc.
Brad Price	Project Engineer - Rollins, Brown and Gunnel, Inc.
Gerald Stoker	Utah State Division of Water Rights - Area Engineer
Dr. James Baer	Professor of Geology, BYU - Geologic Consultant for Rollins, Brown and Gunnell, Inc.
S. Bryce Montgomery	Former Project Geologist for Division of Water Resources
Ben Everitt	Geologist for Division of Water Resources
Dr. M. Dane Picard	Professor of Geology, University of Utah
Dwight Miller	Chief Inspector - Creamer and Noble Engineers

APPENDIX B

Review of Geotechnical Investigations, Design Criteria, and Embankment Design

1. REVIEW OF GEOTECHNICAL INVESTIGATIONS, DESIGN CRITERIA AND EMBANKMENT DESIGN

A. Geotechnical Investigations and Design Analysis:

Geotechnical Investigations and Design Analysis are reported in the March 1983 report entitled, "Geotechnical Investigations and Design Analysis for Quail Creek Dam", prepared for the Washington County Conservancy District by Rollins, Brown and Gunnell, Inc., and "Final Design Report for Quail Creek Dam and Dike", dated August 1983, prepared by Rollins, Brown and Gunnell, Inc.

B. Geologic and Subsurface Investigation:

Geological and Subsurface Investigations conducted for design are reported in Sections IV and V of the March 1983 and further discussed in Paragraph IV and V of the Final Design Report. A comprehensive review of the Geology and Subsurface Investigation at the dike site and discussion of design assumptions is presented in Paragraphs 4 and 5 of the main body of this report.

C. Available Embankment Materials:

Investigations conducted for available materials are reported in the Preliminary Report dated March 1983 and amplified in the Final Design Report. The preliminary report was limited to the reservoir basin and an area immediately downstream from the dam along the Virgin River floodplain. The final design report included a number of locations outside the reservoir basin and additional investigations within the reservoir basin. Material types were divided into five categories.

1. Sandy Gravelly Materials:

   Sandy Gravelly Materials were obtainable from borrow areas 3, 3A and 3B. The sandy gravelly material is suitable for use in zone 3 of the dike cross section. Concern for the quantity of oversize materials and for the percentages of silt and clay size range materials were expressed.

2. Silty Sands, Sandy Silts and Clayey Silt Materials:

   Silty materials existed throughout the reservoir basin. Ninety six (96) test pits were excavated to define this material. Borrow Area 1A was identified as an alternative source of these materials.

3. Clay Materials:

   The source for clay materials were identified as Borrow Areas 2A and 2B. Twenty (20) test pits were excavated in area 2A and fourteen (14) in area 2B. Materials were generally a shale which weathered and broke down when exposed to the atmosphere. Most of the material classified as a CL-2 type material. Some classified as a MH type soil.

4. Slope Protection:

   Basalt materials located on the southerly part of Borrow Area 3 were investigated for slope protection.

5. Filter Material:

Filter material was determined to require processing in order to meet required criteria.

D. Investigation of Borrow Area:

Material types, quantities and locations were adequately defined by the investigations.

E. Results of Laboratory Tests:

Laboratory tests consisting of classification tests, particle size distribution analysis, soil moisture density relationships, laboratory permeability, direct shear, triaxal shear, soluble salt content, chemical analysis and dispersive soil tests were conducted in the various borrow areas and results are summarized and reported in the final design report. A brief narrative on each area follows:

F. Description of Borrow Area:

1. Borrow Area 1:

Borrow Area 1 material classified generally as a SM type soil with some MC and Cl-1 type materials. Two samples with a plasticity index of 20 and 24 classified as Cl-2 and CH. Particle sizes after removal of two inch material indicated in excess of 35 percent silt and clay sizes for most samples. Maximum density varied form 98.3 to 126.5 pounds per cubic foot and optimum moisture content varied from 4.8 to 26.5. The 26.5 is a high moisture related to a single sample. The general maximum value is less than 16 percent. Permeability values for silty to lean clay materials ranged from 0.7 to 16.7 feet per year with clay varying from 0.007 to 0.3 feet per year. Direct shear friction angles varied from 33.2 to 35.7 degrees with cohesion varying from 1 to 3 psi. Triaxal shear tests ranged from 32.5 to 39 degrees. Soluble salts in Area No. 1 ranged from 0.1 to 13.3 percent with about 1/3 of the samples having more than 4% soluble salt contents. "The constituents determined during the chemical analysis of Borrow Area No. 1 included calcium, potassium, magnesium, sodium, iron, chlorine, carbonate, sulfate and bicarbonate. It will be observed that the principal anions in the material in the reservoir basin are carbonate and sulfate while the principal cation is calcium. In general the amount of sodium is less than 70 milligrams per liter." The P^H of reservoir materials varied from about 7.8 to 8.1.

"A pinhole test was performed on several samples obtained from the reservoir basin. None of the samples exhibited dispersive-type characteristics." (Text in quote's is from Final Design Report).

2. Borrow Area No. 2:

Borrow Area 2A and 2B samples tested as medium to high plasticity soils. Borrow Area No. 2A material plasticity index ranged from 7 to 44 percent with Borrow Areas 2B materials ranging from 6 to 28 percent. Testing indicated silt and clay sizes ranged from 65 to 90 percent. Maximum density varied from 90.5 to 116 pounds per cubic feet. Low densities reflect high plasticity materials while high densities correspond to low plasticity soils. Optimum moisture content varied form 11.9 to 30.2 percent. The coefficient of permeability ranged from 0.005

to 0.27 feet per year. Direct shear tests friction angle varied for 10.7 to 28.3 degrees. Cohesion varies from 2 to 4.5 psi. A triaxial shear test was performed on one sample. An observed angle of friction of 23.5 degrees and cohesion of 5 psi was reported. Soluble salts ranged for 0.25 to 5.5 percent with one sample at 10.75 percent.

"The results of the chemical analysis for Borrow Area No. 2 indicated that the principal cations are calcium and sodium while the principal anions are carbonate and sulfate. In several of these samples, the amount of sodium is approximately equal to the amount of calcium; however, it should be noted that the amount of chlorine in these samples is relatively small." (Final Design Report).

3. <u>Borrow Area No. 3</u>:

For Borrow Area No. 3 "Bulk samples" were tested. Eleven to 43 percent of the soils in the bulk samples were materials in excess of 2 inches. If the 2-inch material were scalped the gradation for the remaining materials would show fines in excess of 5%. Borrow Area No. 3A. As for Borrow Area No. 3 the bulk samples had 32 to 55 percent of material larger than 2 inches. The remaining samples tested had maximum sizes less than 2 inches and silt and clay sizes less than 5%.

The general composition of borrow Area No. 3 consists of a surface layer of silty sand underlain by sandy gravel. Borrow Area No. 3A classified as poorly graded sands and gravels with less than 5 percent silts and clay. The maximum density of the sandy gravel varied from 128.4 to 137.5 percent. The maximum density of the silty sand varied from 115.9 to 121.8 p.c.f.

"Permeability tests for the sandy gravel were performed on synthetic curves for the granular material having a maximum size of 3/4 inch. The amount of material in the silt and clay size range for these two tests varied from 3.7 percent to 11.3 percent. The results of the permeability test for the sample having 3.7 percent minimum 200 material was 203 feet per year while the permeability coefficient for the granular sample having 11 percent minus 200 material was 110 feet per year. The permeability coefficients for the silty sand in this borrow area range from 6.1 feet per year to 32.5 feet per year. The higher permeability value had 12.4 percent of the material less than the 200 sieve while the sample having a permeability coefficient of 6 feet per year had 39 percent of the material in the silt and clay size range." (Final Design Report).

Triaxial shear tests on two samples of the sandy gravel indicated friction angles of 37.3 and 39.2 degrees. A friction angle of 33.7 degrees was reported for the silty sand materials. The amount of soluble salt in the silty sand in Borrow Area 3 is very small with the percentage ranging from 0.05 to 0.15.

<u>Use of Available Materials in the Dike</u>:

A discussion of material use including the effects of soluble salts on embankment performance is presented in the Final Design Report. The recommended use in the embankment cross section indicates that permeability along with stability and material availability dictated the final cross section. The dike is designed as a zoned section with a silty sand central core underlain by clay in the inspection trench, a clay zone upstream of the central zone and shells consisting of sandy gravel upstream and random fill surrounded by sandy gravel downstream. Internal drainage for through the dike flows were controlled by a sand filter zone downstream of the central core. A filter zone was not provided for the upstream shell. Side slopes of 2.75 horizontal to 1 vertical for the upstream slope and 2 horizontal to 1 vertical for the downstream slope were provided. Conditions of steady state seepage, drawdown and seismic stability were analyzed. Safety factors of 1.5 for steady state seepage and 1.3 for drawdown were determined. A seismic review and discussion is presented for the dam embankment in the Final Design Report. In addition a discussion on the potential for swelling soils was presented. This was initiated to respond to concerns regarding use of gypsum type soils.

G. **Material Testing During Construction**:

The test results provided with the weekly progress reports were reviewed. Materials appears to have been generally placed and compacted in accordance with the specifications. Construction photographs were reviewed and no significant construction deficiencies were noted. Construction photographs do show large amounts of Zone I material placed as a leveling course. Material placements appear to have been orderly with care taken to avoid contamination of zones. Worthy of some note is that problems with freezing soils were noted early in the construction. Also of interest is that the clay material for Zone II were generally placed well dry of optimum (5 to 10 percent less). This was allowed by the specifications and the density requirements were met. The gradation tests of the filter materials were reviewed. The filter materials were generally near the 5 percent maximum allowed for the No. 200 Sieve.

H. **Material Testing After Failure**:

Material testing after the failure consisted of the following:

Zone No. I

Four locations near the cutoff trench for sulfates, solubility and gradation.

Zone No. II

Samples from 2 locations in the upstream cutoff and 2 samples in the centerline cutoff for gradation (including hydrometer), pinhole tests, Atterberg limits and mineralogical analysis.

Zone No. III

Four gradations

Filter zone

Two gradations

The objective of the post-failure testing with the exception of the mineralogical analysis was to compare inplace materials with design investigations. The mineralogical analysis was to determine the clay mineral present in Borrow Area No. 2A and No. 2B materials. This was not done during the design investigation. The clay minerals were determined and found to be primarily illite, kaolinite and smectite. Results of the testing are presented in reports from Chen-Northern, Inc. (Appendix C). Zone 1, II and III materials appear to meet specification requirements. Zone I and II materials testing indicate gravel and cobble sizes. These may be contamination from surrounding zones. There was no maximum size specification. Zone III materials have 18 to 38 percent of materials in excess of 1 1/2 inches. The matrix material would therefore have fines in excess of 5%. Since a maximum size was not specified this is not a specification deficiency however the high fine content would make the materials less pervious. Filter materials meet the specifications upper and lower limits but are out of specification in the sand range. This is not viewed as a serious deficiency as the number of samples is quite small.

APPENDIX C

Post-Failure Soil and Rock Testing

C-1. Chen-Northern Testing

C-2. Mineralogical and Petrographic Report

Chen-Northern, Inc.
350 West 2700 South
Salt Lake City, Utah 84115
801/487-3661

Billings
Boise
Casper
Colorado Springs
Denver
Elko
Evanston
Gillette
Glenwood Springs
Great Falls
Helena
Phoenix
Pocatello
Rock Springs
Salt Lake City
San Antonio
Tri Cities
Yakima

February 14, 1989

Subject: Geotechnical Test Results
Quail Creek Reservoir
Embankment Material
near St. George, Utah

Utah State Dam Safety
Division of Water Rights
1636 West North Temple
Salt Lake City, Utah 84116

Attn: Mr. Hyrum Alba

Gentlemen:

As requested, Chen-Northern, Inc. performed geotechnical laboratory tests on samples obtained from the embankment at the breach of the Quail Creek Reservoir dike. Tests which were performed in our geotechnical laboratory included gradation, hydrometer, Atterberg Limits, and pinhole dispersion tests. The total soluble salts determination was performed by Ford Chemical. The results of the laboratory tests are presented on Table I and Figures 1 through 9. Field density tests (sand cones) were performed at the site with the data presented on Table II.

If you have questions, or if we may be of further service, please call.

Sincerely,

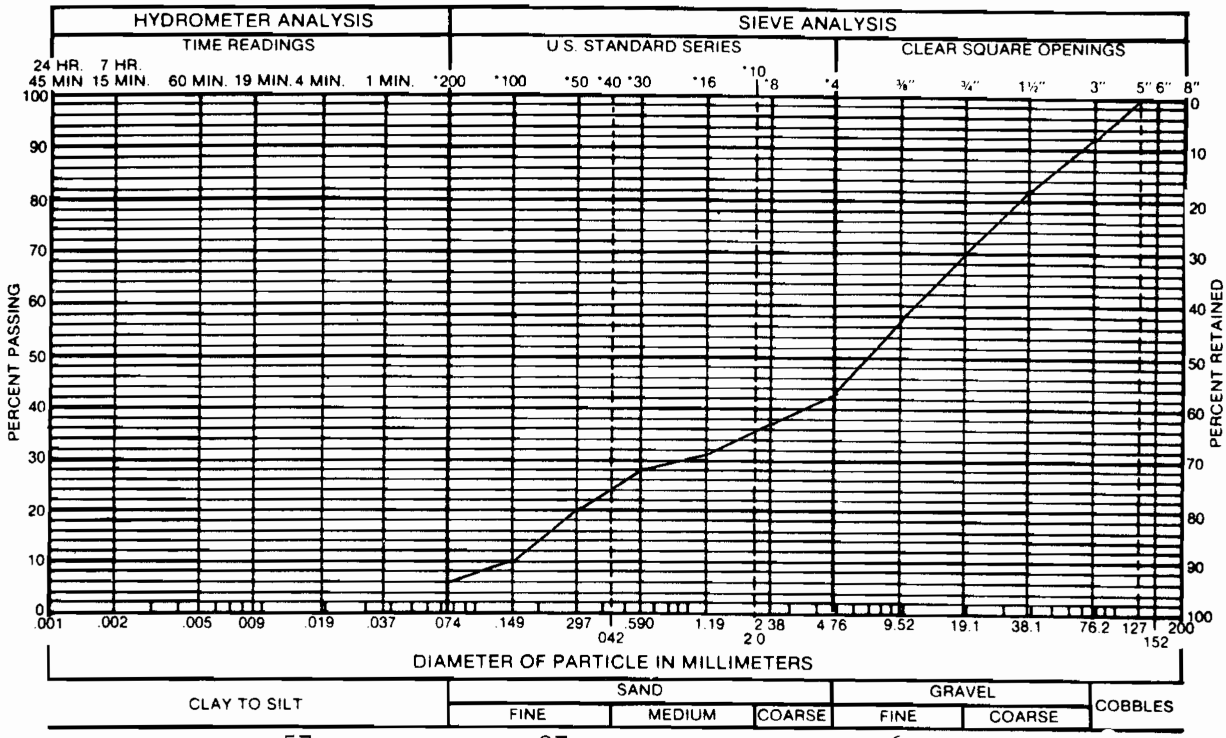
CHEN-NORTHERN, INC.

Walter V. Jones

Walter V. Jones, P.E.
Division Manager

WJ/DBG/cs
Enclosures

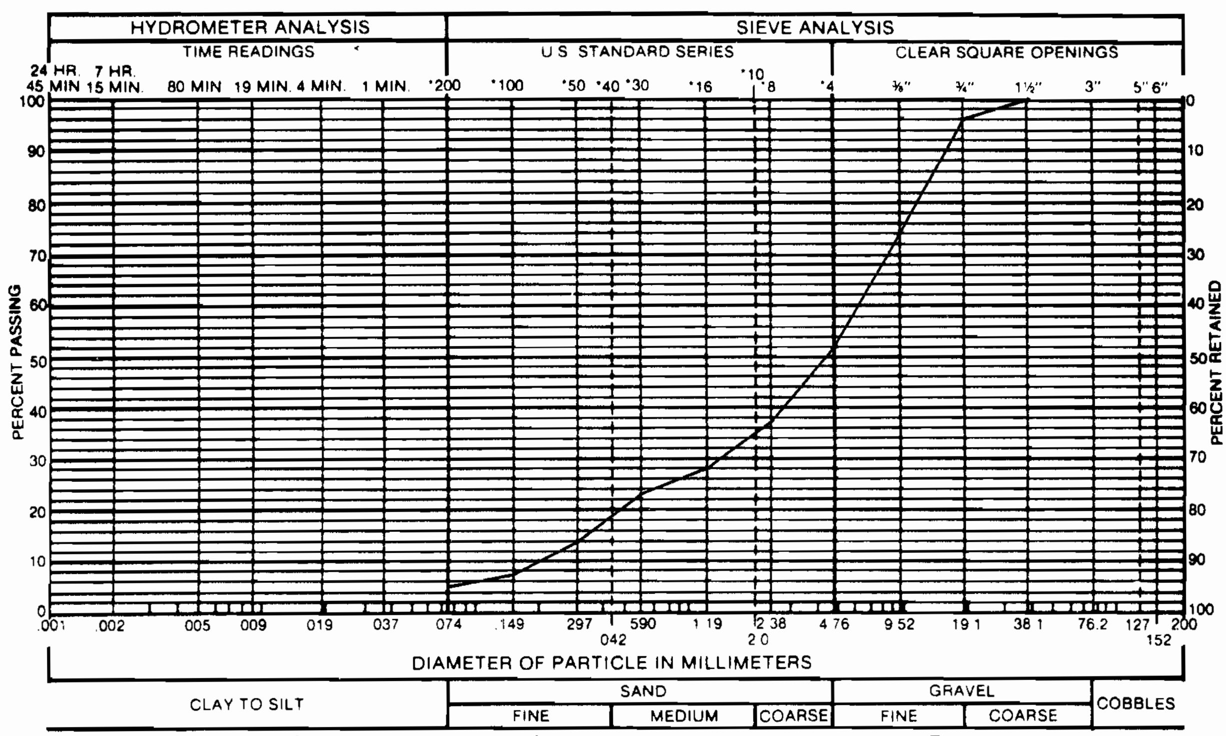
chen and associates, inc.



GRAVEL 57 % SAND 37 % SILT AND CLAY 6 %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Sandy Gravel FROM Zone III, Sample 1



GRAVEL 48 % SAND 47 % SILT AND CLAY 5 %

LIQUID LIMIT % PLASTICITY INDEX %

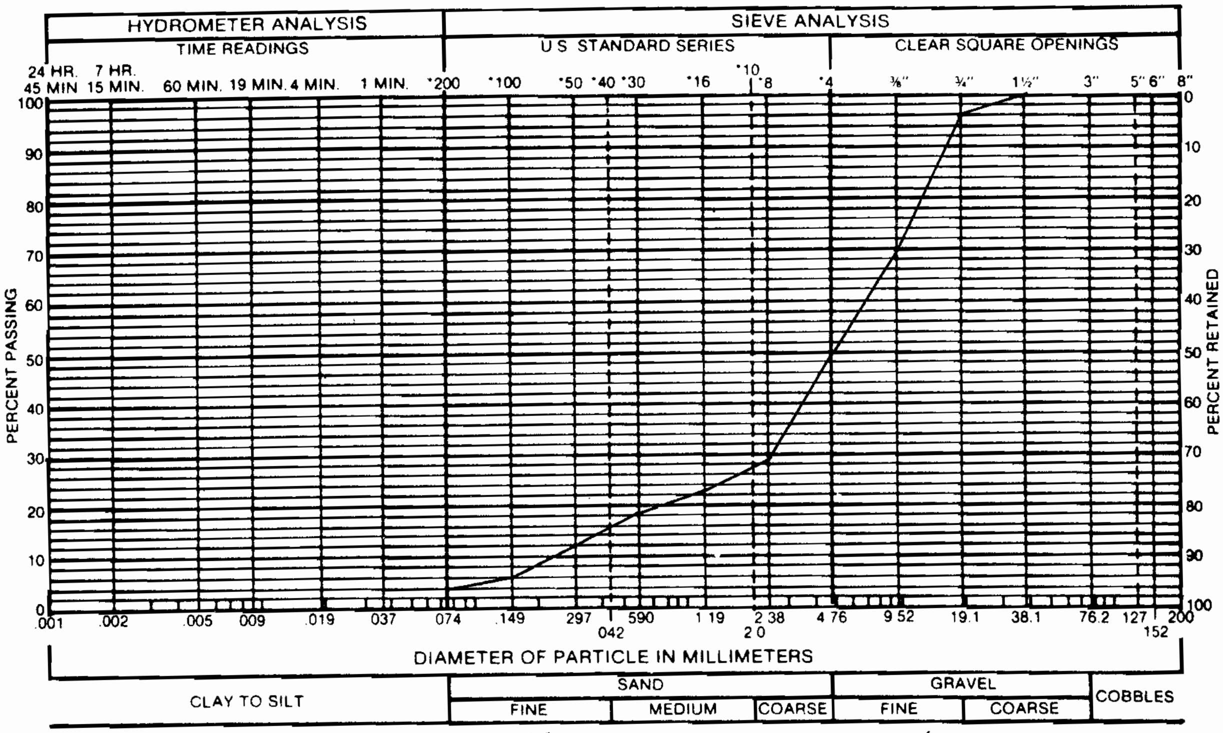
SAMPLE OF Sand and Gravel FROM Chimney Drain, Sample 2

Job No. 517689

GRADATION TEST RESULTS

Fig. 1

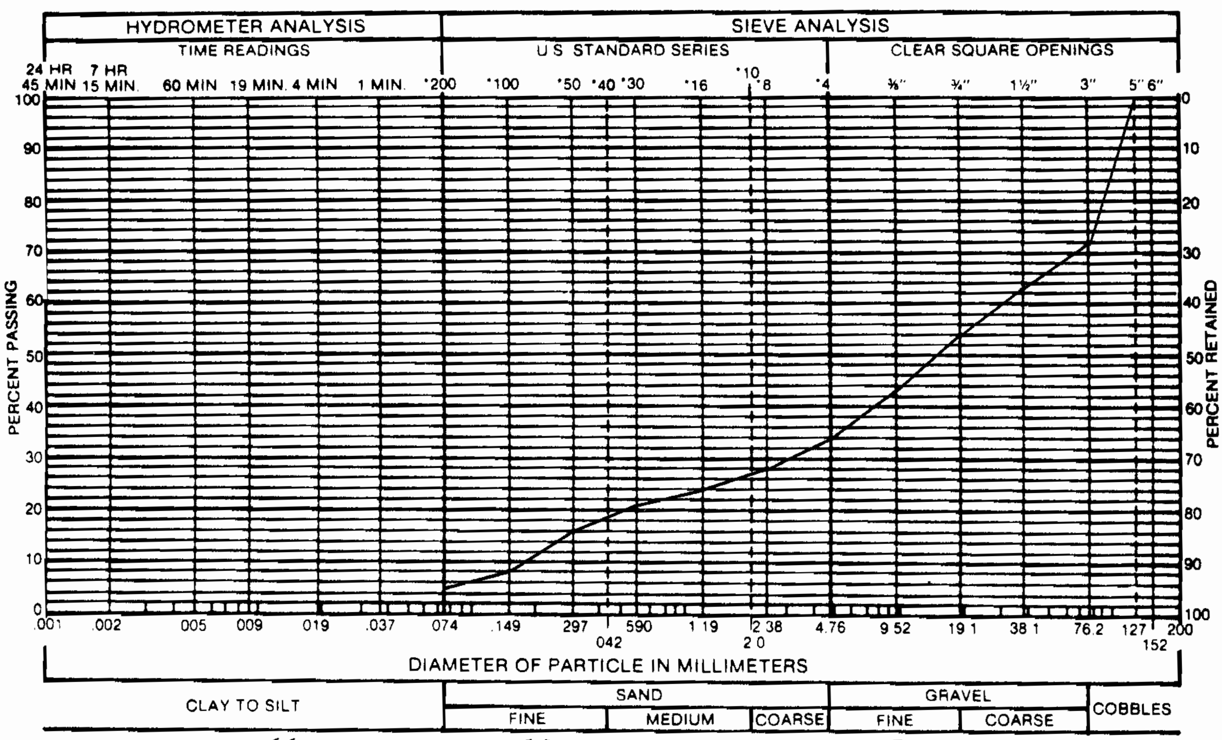
chen and associates, inc.



GRAVEL 50 % SAND 46 % SILT AND CLAY 4 %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Sandy Gravel FROM Chimney Drain, Sample 3



GRAVEL 66 % SAND 19 % SILT AND CLAY 5 %

LIQUID LIMIT % PLASTICITY INDEX %

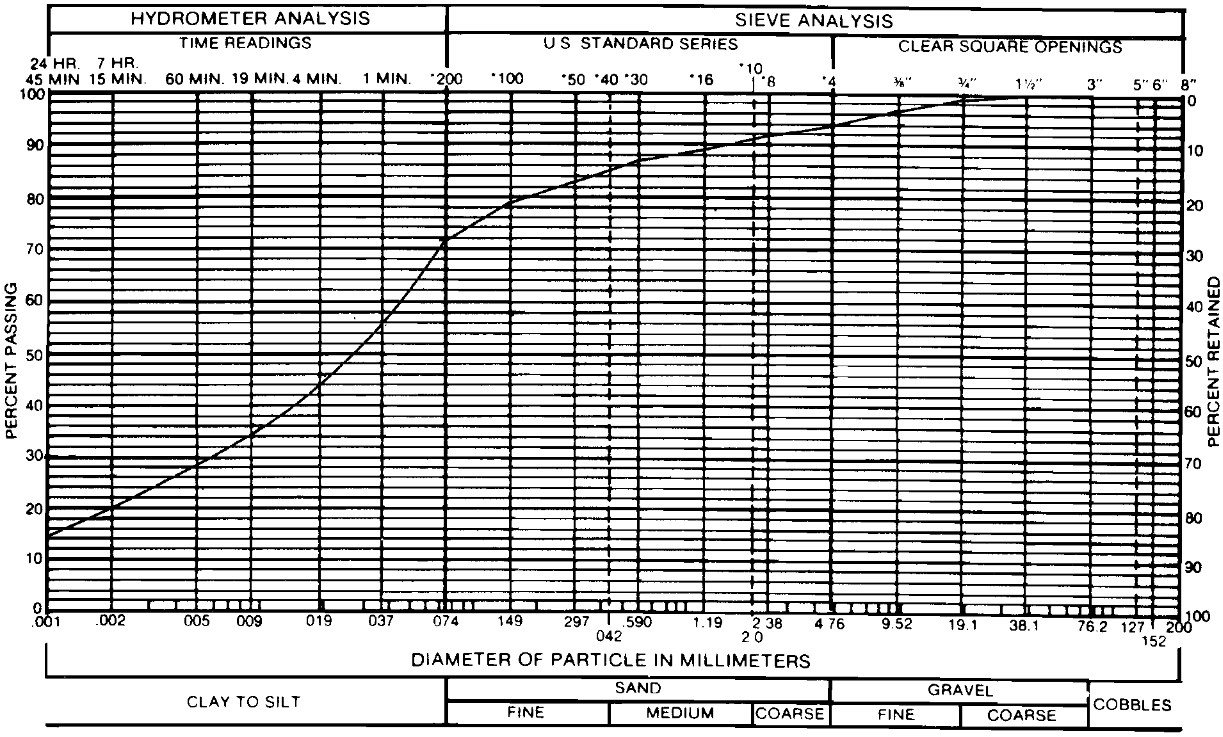
SAMPLE OF Sandy Gravel FROM Zone III, Sample 4

Job No. 517689

GRADATION TEST RESULTS

Fig. 2

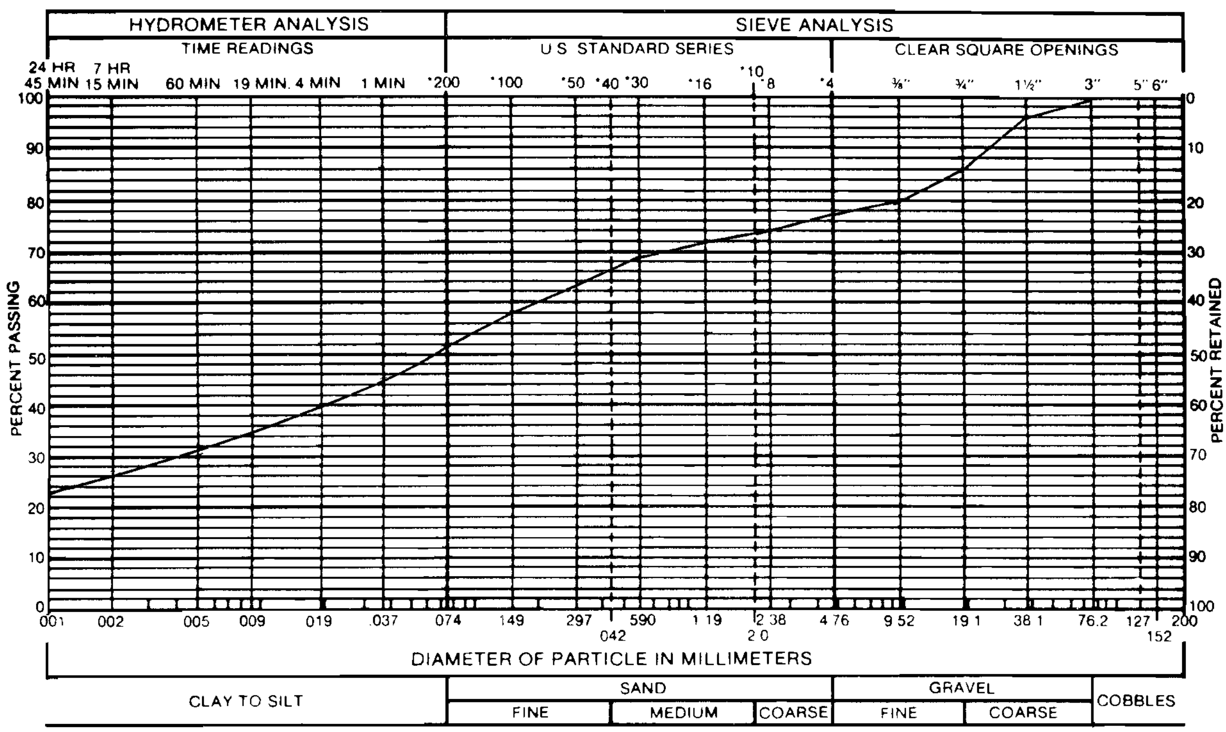
chen and associates, inc.



GRAVEL 6 % SAND 22 % SILT AND CLAY 72 %

LIQUID LIMIT 41 % PLASTICITY INDEX 20 %

SAMPLE OF Sandy Clay FROM Zone II, Sample 5



GRAVEL 23 % SAND 25 % SILT AND CLAY 52 %

LIQUID LIMIT 65 % PLASTICITY INDEX 30 %

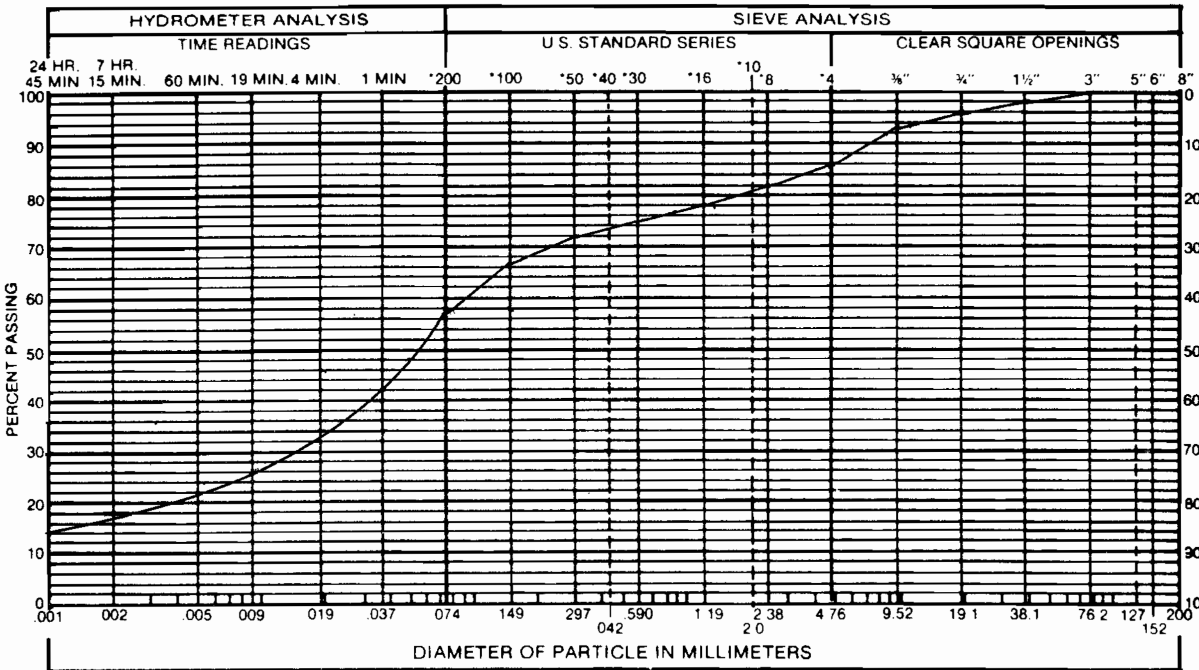
SAMPLE OF Gravelly, Sandy Silt FROM Zone II, Sample 6

Job No. 517689

GRADATION TEST RESULTS

Fig. 3

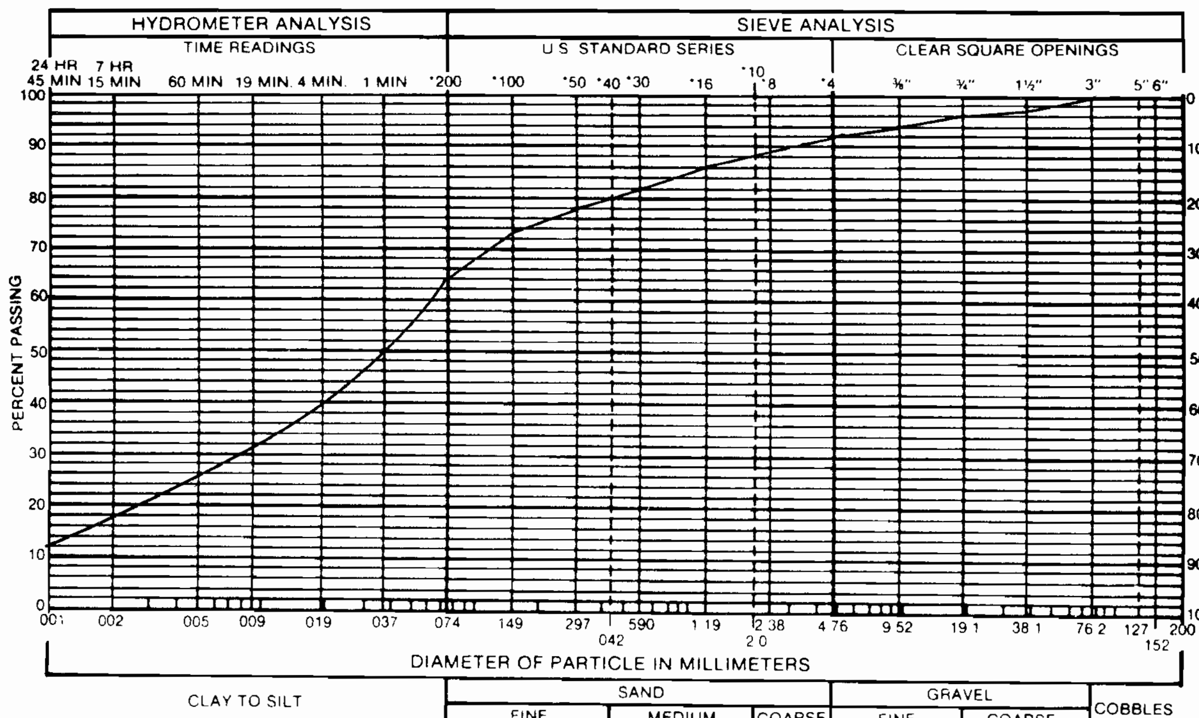
chen and associates, inc.



GRAVEL 14 % SAND 29 % SILT AND CLAY 57 %

LIQUID LIMIT 23 % PLASTICITY INDEX 2 %

SAMPLE OF Gravelly, Sandy Silt FROM Zone I, Sample 7



GRAVEL 8 % SAND 28 % SILT AND CLAY 64 %

LIQUID LIMIT 46 % PLASTICITY INDEX 26 %

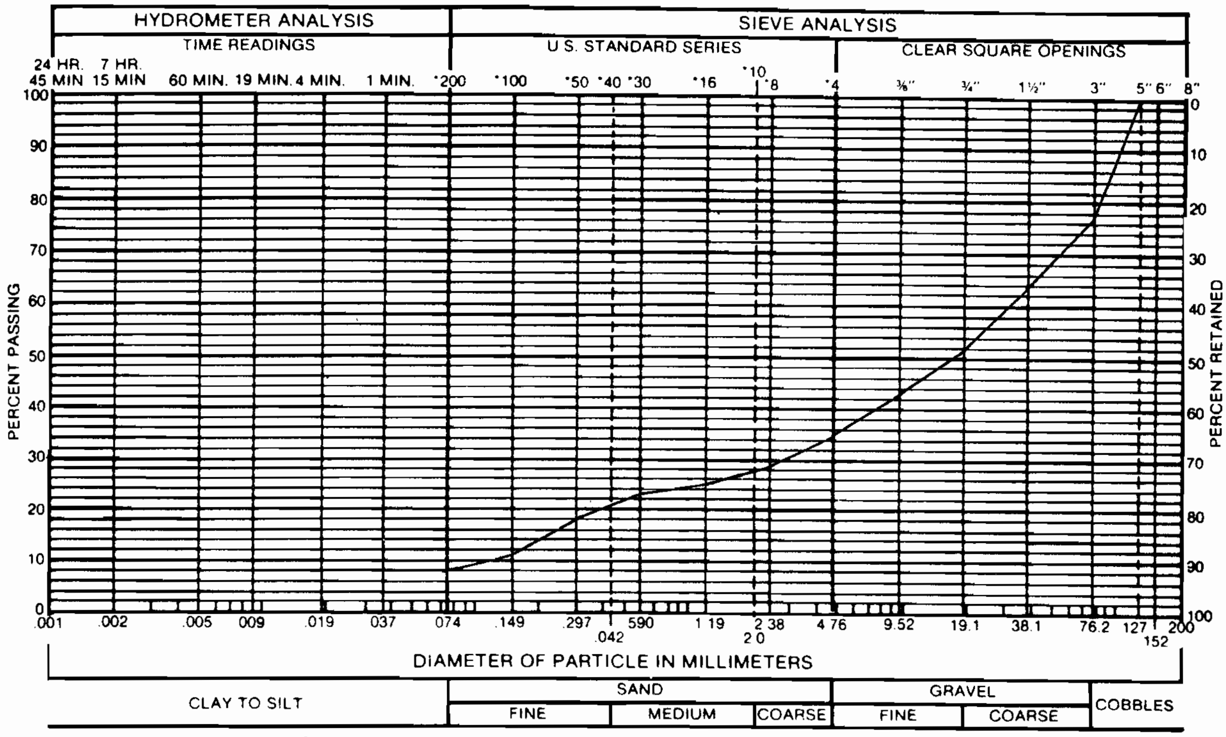
SAMPLE OF Sandy Clay FROM Zone I, Sample 8

Job No. 517689

GRADATION TEST RESULTS

Fig. 4

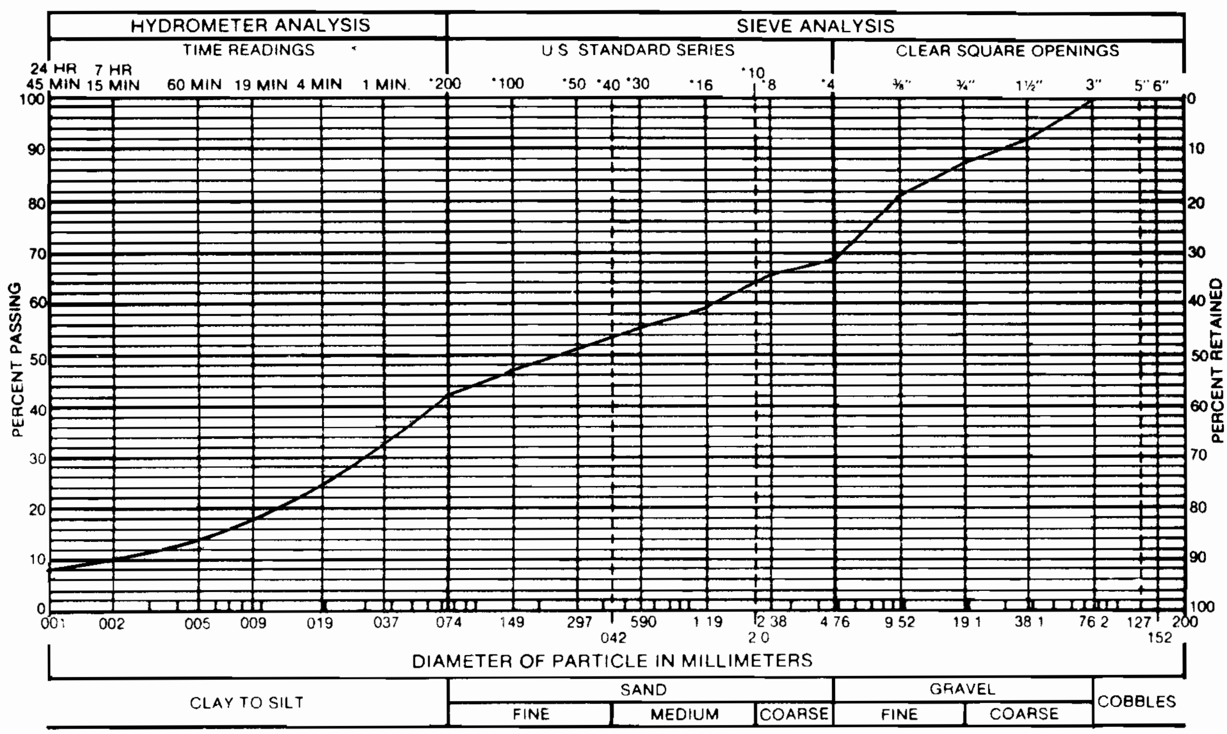
chen and associates, inc.



GRAVEL 65 % SAND 27 % SILT AND CLAY 8 %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Sandy Gravel FROM Zone III, Sample 9



GRAVEL 31 % SAND 27 % SILT AND CLAY 42 %

LIQUID LIMIT 34 % PLASTICITY INDEX 6 %

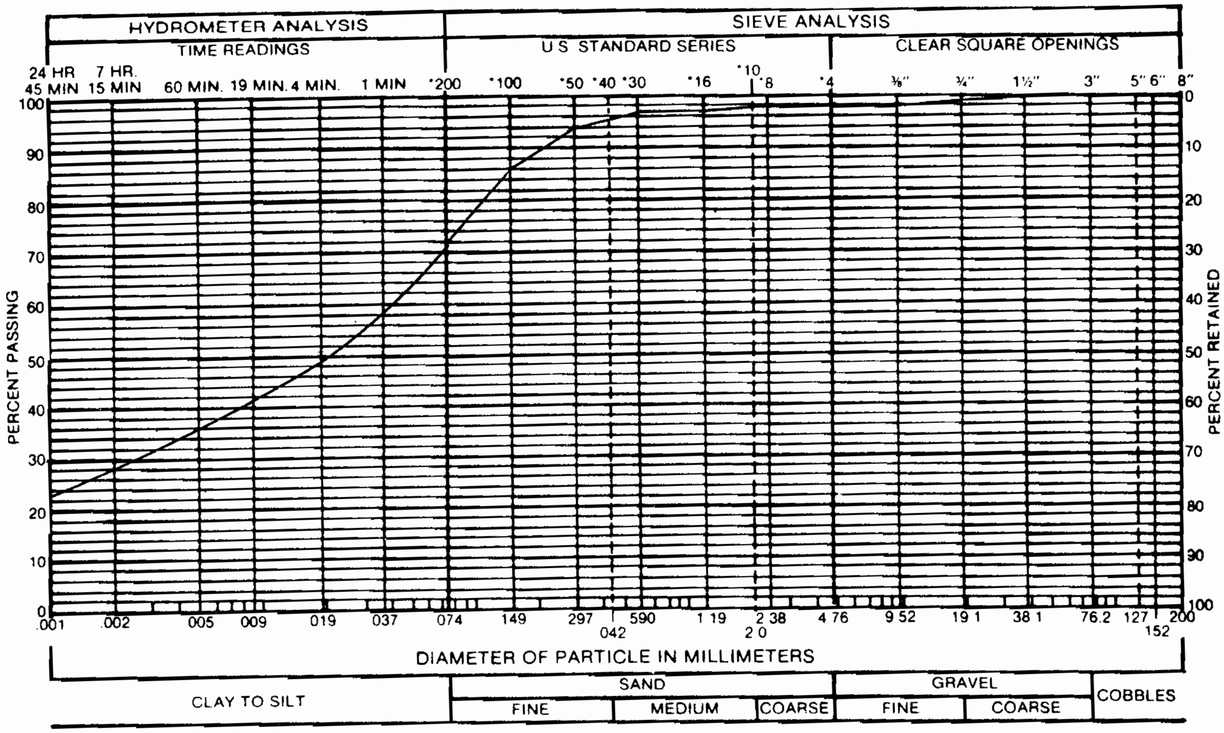
SAMPLE OF Silty Sand and Gravel FROM Zone II, Sample 10

Job No. 517689

GRADATION TEST RESULTS

Fig. 5

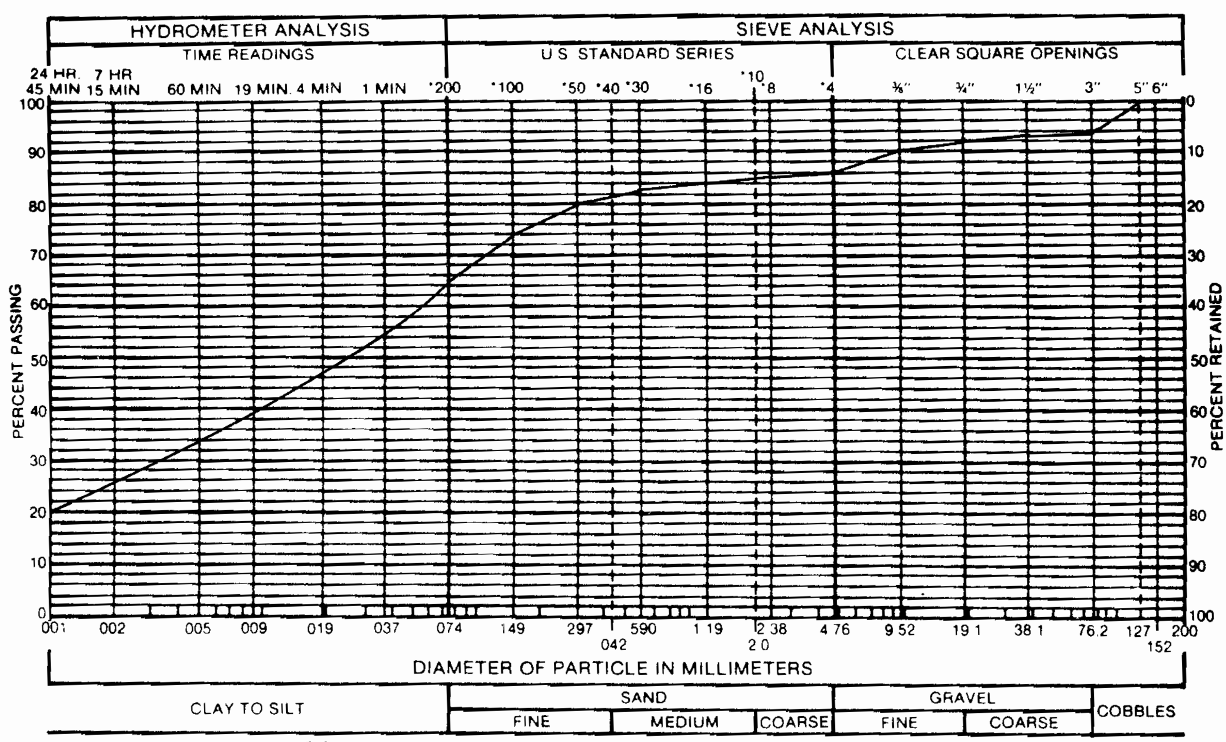
chen and associates, inc.



GRAVEL 2 % SAND 26 % SILT AND CLAY 72 %

LIQUID LIMIT 54 % PLASTICITY INDEX 23 %

SAMPLE OF Sandy Silt FROM Zone II, Sample 11



GRAVEL 14 % SAND 21 % SILT AND CLAY 65 %

LIQUID LIMIT 56 % PLASTICITY INDEX 22 %

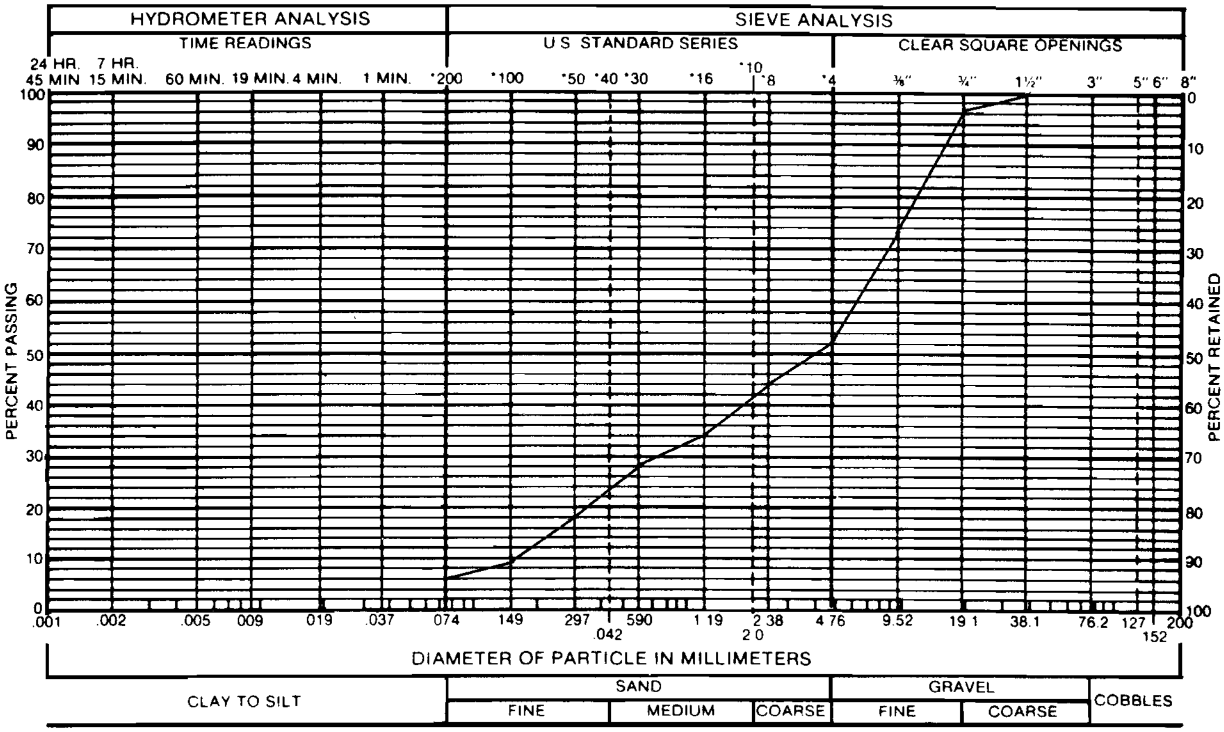
SAMPLE OF Gravelly, Sandy Silt FROM Zone II, Sample 12

Job No. 517689

GRADATION TEST RESULTS

Fig. 6

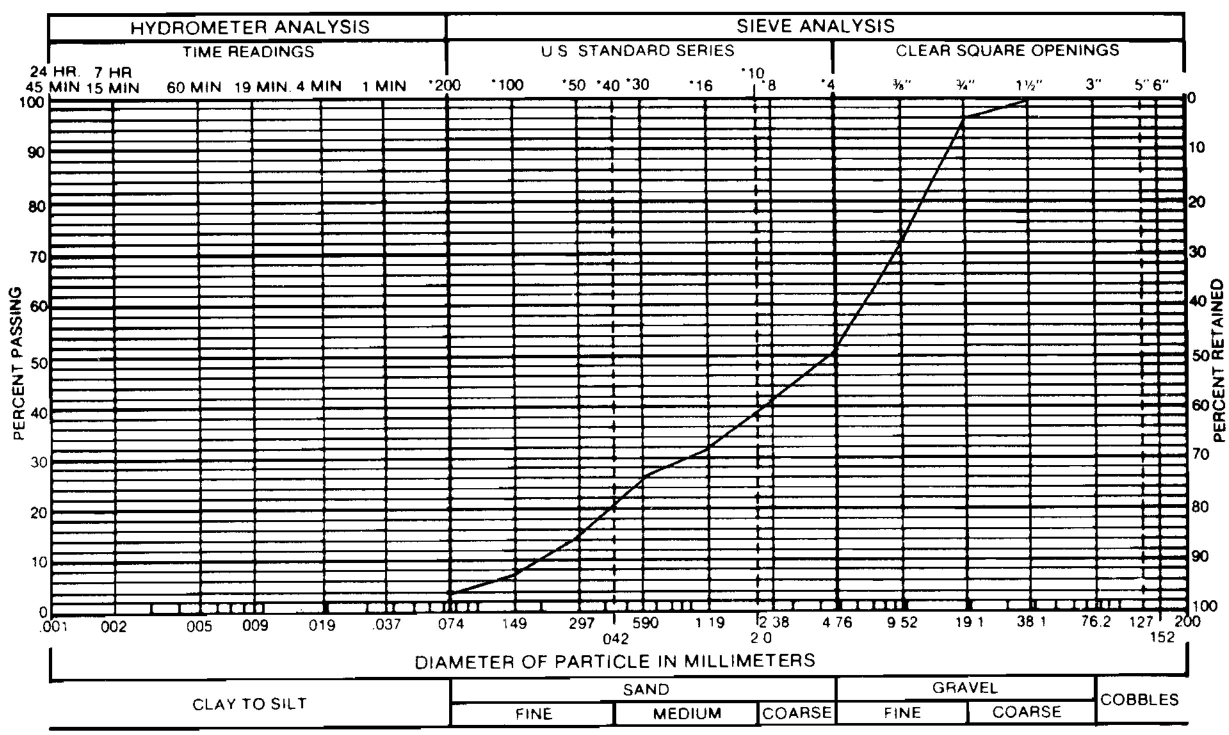
chen and associates, inc.



GRAVEL 48 % SAND 46 % SILT AND CLAY 6 %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Sand and Gravel FROM Chimney Drain, Sample 13



GRAVEL 49 % SAND 47 % SILT AND CLAY 4 %

LIQUID LIMIT % PLASTICITY INDEX %

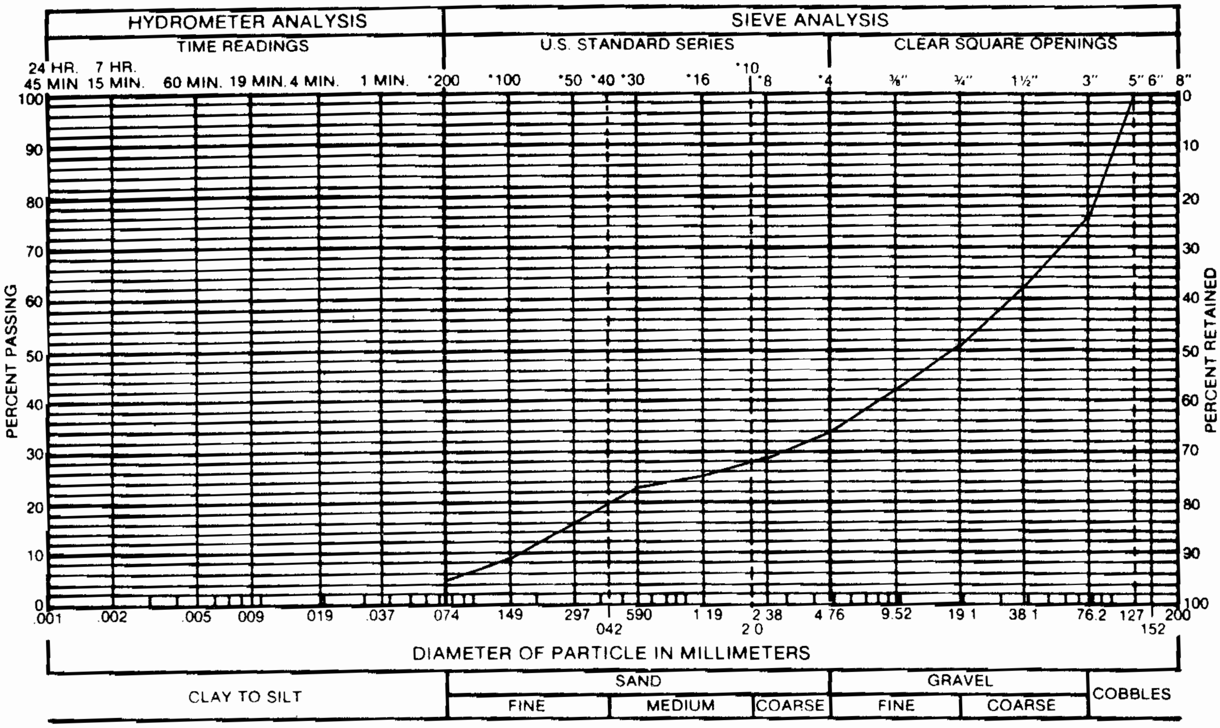
SAMPLE OF Sand and Gravel FROM Chimney Drain, Sample 14

Job No. 517689

GRADATION TEST RESULTS

Fig. 7

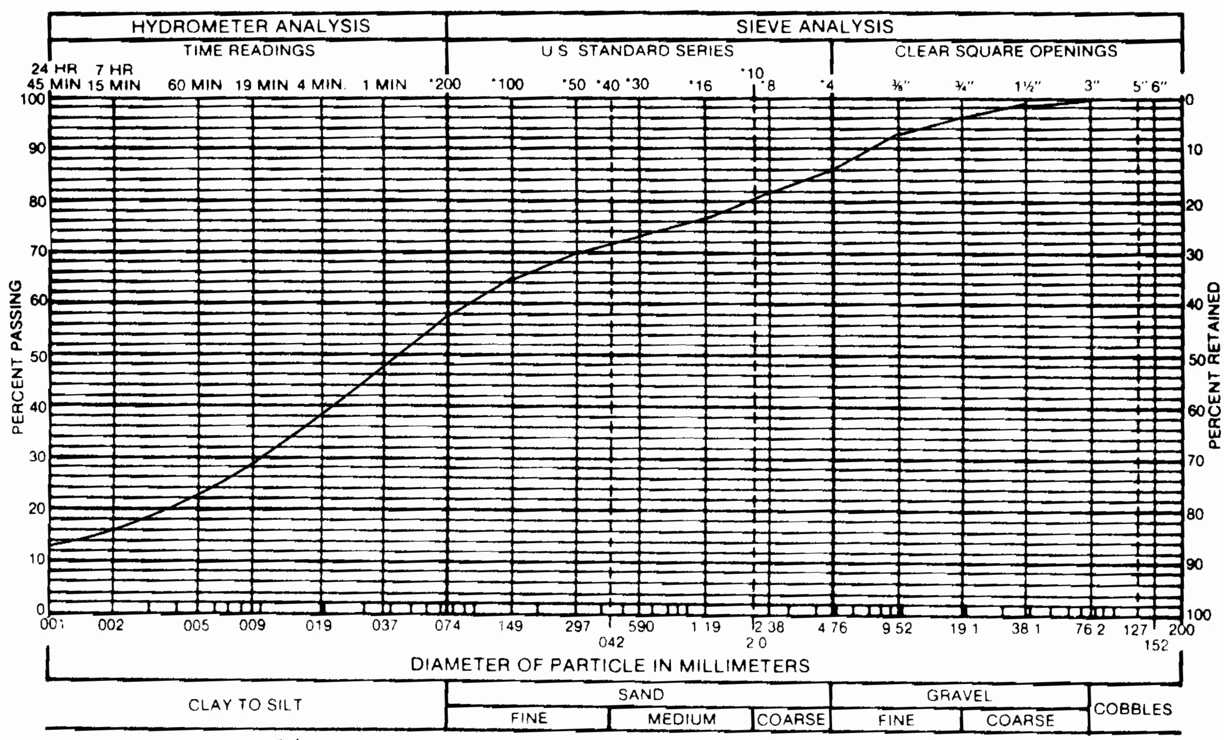
chen and associates, inc.



GRAVEL 66 % SAND 29 % SILT AND CLAY 5 %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Sandy Gravel FROM Zone III, Sample 15



GRAVEL 14 % SAND 28 % SILT AND CLAY 58 %

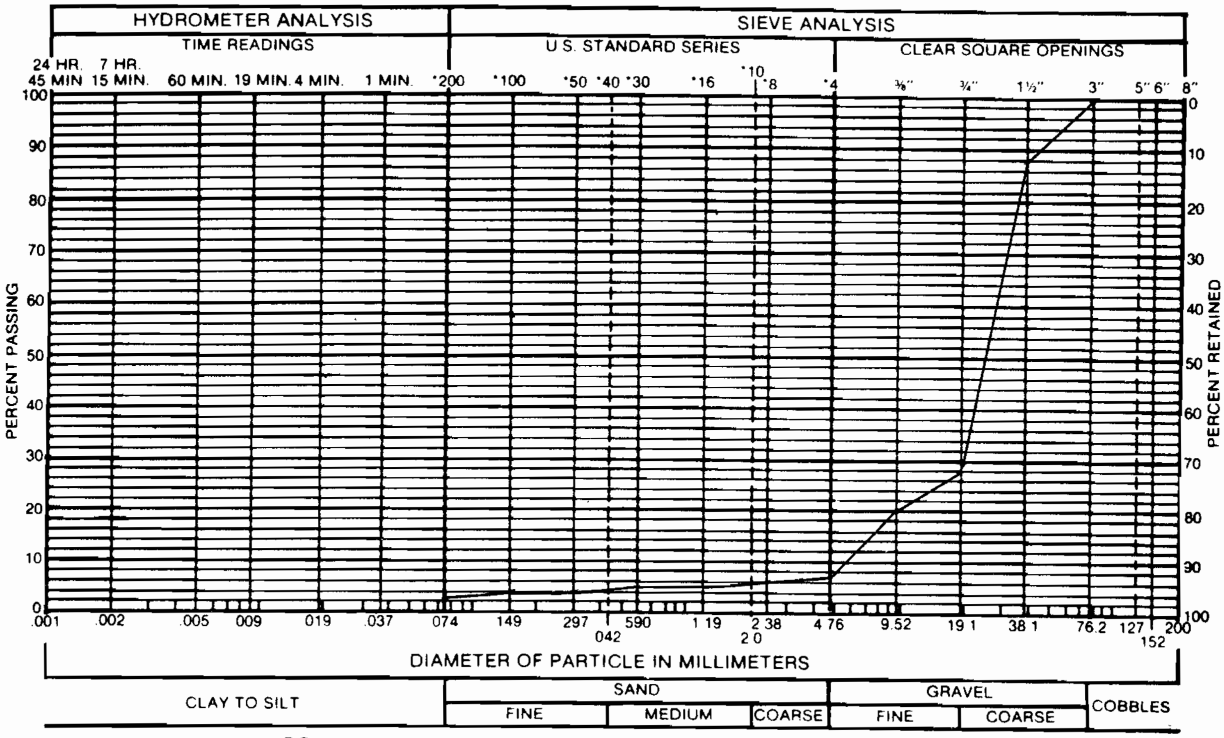
LIQUID LIMIT 42 % PLASTICITY INDEX 15 %

SAMPLE OF Gravelly, Sandy Silt FROM Zone I, Sample 16

Job No. 517689

GRADATION TEST RESULTS Fig. 8

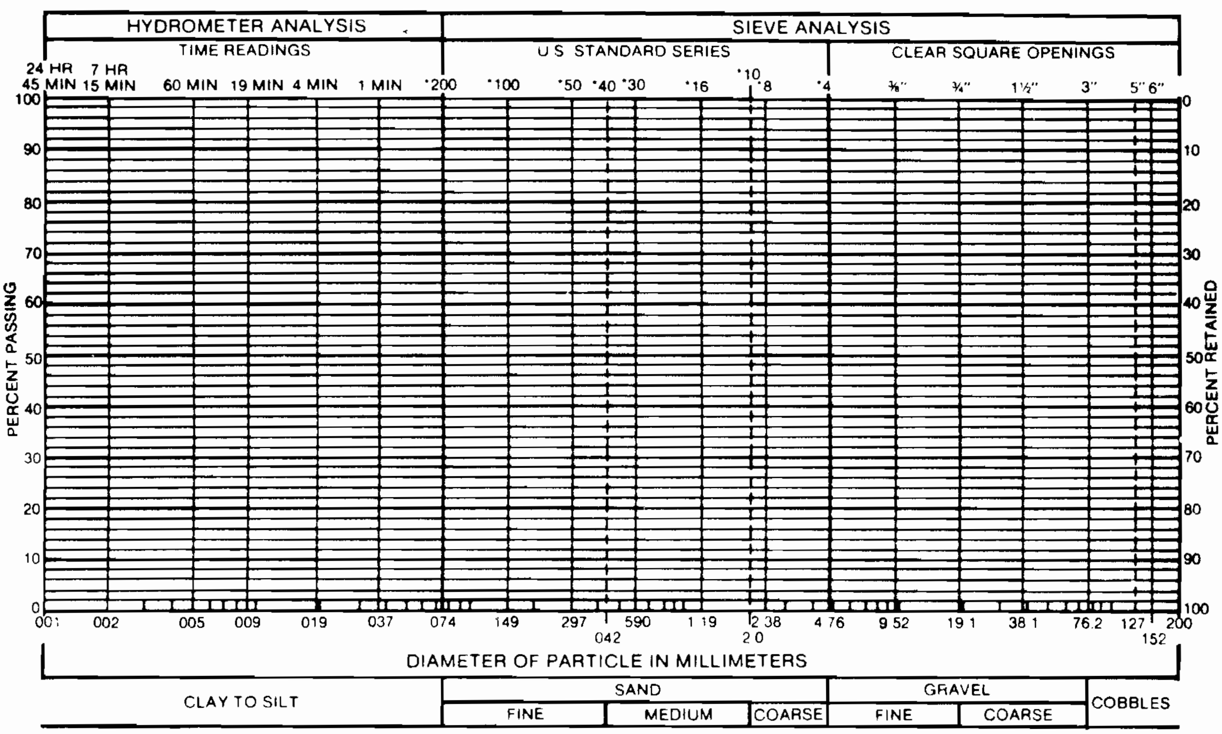
chen and associates, inc.



GRAVEL 93 % SAND 4 % SILT AND CLAY 3 %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Gravel FROM Sewer Rock, Sample 17



GRAVEL % SAND % SILT AND CLAY %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF FROM

Job No. 517689 GRADATION TEST RESULTS Fig. 9

TABLE II

Location	Dry Density pcf	Moisture Content %
Upstream cutoff-Zone II; Westside of Breach	115.6	19.1
Clay Core-Zone II; East Side of Breach	95.3	28.9
Zone I-Approximately 90 ft. South of Section Centerline; East Side of Breach	122.6	13.0
Zone I-Approximately 125 ft. South of Chimney Drain and 130 ft. East of Breach Centerline; East Side of Breach	117.7	15.7
Zone I-Approximately 125 ft. South of Chimney Drain and 160 ft. East of Breach Centerline; East Side of Breach	117.6	15.6

CHEN AND ASSOCIATES

Job No. 517689

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

SAMPLE LOCATION		NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (PCF)	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		Total Soluble Salts (%)	Dispersive Classification	SOIL OR BEDROCK TYPE
Sample No.	Zone			GRAVEL (%)	SAND (%)		LIQUID LIMIT (%)	PLASTICITY INDEX (%)			
1	III			57	37	6					Sandy Gravel
2	Chimney			48	47	5					Sand and Gravel
3	Chimney			50	46	4					Sandy Gravel
4	III			66	19	5					Sandy Gravel
5	I			6	22	72	41	20	2.22		Sandy Clay
6	II			23	25	52	65	30	1.33		Gravelly Sandy Silt
7	I			14	29	57	23	2	1.24		Gravelly Sandy Silt
8	I			8	28	64	46	26	2.11		Sandy Clay
9	III			65	27	8					Sandy Gravel
10	II			31	27	42	34	6	1.70	ND- 3*	Silty Sand & Gravel
11	II			2	26	72	54	23	0.91	ND- 3	Sandy Silt
12	II			14	21	65	56	22	0.90	ND- 3*	Gravelly Sandy Silt
13	Chimney			48	46	6					Sand and Gravel
14	Chimney			49	47	4					Sand and Gravel
15	III			66	29	5					Sandy Gravel
16	I			14	28	58	42	15	2.08		Gravelly Sandy Silt
17	Sewer Rock			93	4	3					Gravel

* Pinhole plugged during test, preventing flow.

** The macrostructive of undisturbed soils may affect pinhole test results as much as the presence of dispersive clays (1988 Annual Book of ASTM Standards D-4647 P858)

Ford Chemical
LABORATORY, INC.
Bacteriological and Chemical Analysis
40 WEST LOUISE AVENUE
SALT LAKE CITY, UTAH 84115
PHONE 466-8761

DATE: 02/06/89

CERTIFICATE OF ANALYSIS

CHEN - NORTHERN
350 W. 2700 SO.
SALT LAKE CITY, UTAH
84115

89-000002

SAMPLE: SOIL SAMPLES FROM QUAIL CREEK JOB #5-176-89 COLLECTED 2-1-89 RECEIVED 2-1-89 FOR ANALYSIS STARTING AT 5 P.M. UNDER P.O. #35527

		TOTAL SOLUBLE SALTS %
1	SAMPL #5 ZONE 1	2.220
2	SAMPLE #6 ZONE 2	1.330
3	SAMPLE #7 ZONE 1	1.240
4	SAMPLE #8 ZONE 1	2.110
5	SAMPLE #10 ZONE 2	1.700
6	SAMPLE #11 ZONE 2	.910
7	SAMPLE #12 ZONE 2	.900
8	SAMPLE #16 ZONE 1	2.080

FORD CHEMICAL LABORATORY, INC.

APPENDIX C-2

PETROGRAPHY OF TRIASSIC SHNABKAIB MEMBER, MOENKOPI FORMATION, AT THE QUAIL CREEK DIKE

M. Dane Picard
(Consulting Geologist)

Department of Geology and Geophysics
University of Utah
Salt Lake City, Utah 84112

INTRODUCTION

The failure of the Quail Creek Dike in Section 35, T41S, R14W, Washington County, Utah, on January 1, 1989, which sent a 12-foot wall of water down the Virgin River, and caused an estimated $12 million damage in Utah, Arizona and Nevada, has led to detailed studies of the remains of the dike, the foundation rock, and the geology of strata nearby in the reservoir basin and downstream below the dike. This report summarizes: 1) the petrography of rocks at the site (Appendix 1); 2) X-ray diffraction studies of two samples from the traverse to collect petrographic samples (Appendix 2); and 3) X-ray diffraction studies of joint fillings (Appendix 3). These were the principal tasks I was asked to undertake. In addition, as the work progressed, two additional requests were made: 1) to determine the clay mineralogy and general mineral composition of Zone II material used in the Quail Creek Dike (see Appendix 4); and 2) to determine the composition by X-ray diffraction of unconsolidated material collected at the toe of the dike and from the inside of a pipe (see Appendix 5).

I talked to members of the Quail Creek Dike Review Board about preliminary petrographic results on February 5, 1989, and went with them to the field twice the next day. I met again with several members of the panel on February 24, 1989, both in the field and at the Holiday Inn, St. George, Utah.

This report summarizes the results of the various studies and includes additional observations that may be of interest and are complimentary to other geological and engineering investigations of the Quail Creek Dike.

GENERAL STRATIGRAPHY

The Moenkopi Formation (Triassic) in southwestern Utah is subdivided into six members. From oldest to youngest, these are: Timpoweap (80-230 feet thick), lower red (220-310 feet), Virgin limestone (8-116 feet), middle red (436-520 feet), Shnabkaib (216-376 feet), and upper red (440-564 feet). All of the members are easy to recognize at a distance though they are gradational into overlying and underlying members (Gregory, 1952; McKee, 1954). All of them are present in outcrop throughout the area.

McKee assigned Triassic strata to a complex mixture of environments: streams, lagoons, playas, floodplains or tidal flats, shallow sea floors, and others. Evidence from the flora, fauna and primary sedimentary structures indicate there was a semiarid to arid climate present during their deposition (McKee, 1954, p. 75).

The Quail Creek Dike was set on the Shnabkaib Member, which contains the most abundant gypsum deposits in the formation, though there is gypsum in all of the Moenkopi red bed members. The Shnabkaib is 630 feet thick at Harrisburg Dome (McKee, 1954, p.16) where the dike was built. The Moenkopi Formation is 2035 feet thick there, not including the Timpoweap Member, which Thomas (1952, p. 59) indicates is absent in the subsurface. There are at least 300 feet of the gypsiferous Shnabkaib Member stratigraphically below the position of the dike and 435 feet of the middle red member which is locally gypsiferous.

PETROGRAPHY OF SHNABKAIB MEMBER

Gypsum - Gypsum is a common primary constituent, ranging from a few percent of some dolomicrite or dolomicrospar to beds of gypsum (F 1), up to 5 or 6 feet thick. It is the cement of red siltstone found in the sequence (F 2). It occurs as lenticular to tabular crystals within all rocks in the section. Nodular bedded and nodular mosaic gypsum is a conspicuous feature within the red beds (Table 1) and, also, interbedded with laminated dolomite. Gypsum fills a large variety of polygonal shrinkage cracks.

Gypsum is also present in a variety of secondary ways: as thin layers parallel or nearly parallel to bedding, as lenticular seams, as crosscutting veins (F 3), and as the filling material in joints.

The abundance of gypsum in the member was greatly underestimated in the design report for the Quail Creek Dike. Appendix 1 gives measurements and estimates of amounts of gypsum in the petrographic samples.

The environmental setting for the parts of the sequence dominated by gypsum is interpreted to have been that of a marine coastal sabkha, the Arabic word for salt flat. In sabkhas, displacive and replacive evaporite minerals form in the capillary Zone above a saline water table (Warren, 1989, p. 38). Characteristic and distinguishing features include the following:

1. The gypsum units are matrix dominated.

2. The gypsum units (supratidal) are thin (<3-6 feet).

3. Displacive and replacive nodular and enterolithic textures are common to abundant.

4. Evaporite crystals are diagenetic. In the Shnabkaib there has been, however, some mechanical deposition of evaporites as indicated by primary sedimentary structures--cross-stratification, wave ripple marks, rip-up breccias, graded beds.

5. Rare sabkha teepees are present. There are indications of plane bedded and polygonal algal mats.

6. The facies are laterally extensive.

The foregoing features correspond closely with those presented by Warren (1989, p. 133) for sabkha facies in contrast to features characteristic of shallow-water facies. Warren's model is based on studies by many other people.

Siltstone - These rocks, which generally are clayey and gypsiferous (F 4-8), split into thin layers between a quarter- and a half-inch in thickness. They are dominantly pale red and grayish red but some are pale olive, light olive gray, and grayish olive. Three primary sedimentary structures or bedding types are characteristic: horizontal lamination, wave-formed ripple marks (and ripple-stratification), and polygonal shrinkage cracks that range greatly in size. It is postulated that these are diagenetic red beds--the original sediment was non-red and became red during diagenesis as iron-rich silicates and ferrous oxide changed to ferric oxide.

Linear asymmetric ripple mark (High and Picard, 1977, p. 61) are the dominant type. The crest pattern is straight and parallel. There are foreset laminae inclined in the direction of flow.

Paleocurrent directions measured from the linear ripple marks in unit #7 are given in Table 2. Based on 19 measurements the dominant direction was S73°W (mean) with a standard deviation of 11.6°. A minor direction of N35°W was noted. There were few measurements (4), but the standard deviation is small (6.0°). The major direction is interpreted to be the offshore direction, approximately perpendicular to the average shoreline. The minor direction (N35°W) may be an oblique direction. These ripple marks formed at about the same time as the others.

Laminae in the siltstone is sometimes contorted and locally cut by microfaults with several mm of displacement (F 9). Deformation is probably related to syn-depositional dissolution of salts and collapse of overlying beds. Gypsum in fractures (veins) is deformed (twinned, bent). Gypsum veins locally enclose small fragments of wall rock siltstone. Siltstone is commonly cemented by gypsum and by small amounts of micrite.

The siltstone is interpreted to be the result of deposition on tidal mud-flats. The abundant polygonal shrinkage cracks and casts of salt cubes attest to subaerial conditions. The abundant linear ripple marks and less common interference ripple marks reflect tidal currents across the flats.

Carbonate rocks - Carbonate rocks are fine-grained, principally dolomicrite or dolomicrosparite. They are pale olive, yellowish gray, light olive gray, and grayish red. They are laminated. Some bedding is wavy and appears to be the result of algal structure, now dolomitized. The carbonate rocks are interbedded with nodular bedded and nodular mosaic gypsum. Veins of gypsum (F 10) are common. Laminae of siltstone are also common, some of which are graded (F 11) or reversely graded.

Rarely, the micrite and microsparite is cross-stratified. Linear ripple marks are common, as are polygonal shrinkage cracks in intervals of gypsum and carbonate.

JOINTS AND JOINT FILLINGS

Results of X-ray diffraction studies of joint fillings are given in Appendix 3. The three samples were collected about 615 feet downstream of the centerline (QC-13, QC-14) and near the centerline of the dike (QC-18). Based on all three samples, gypsum and calcite are the most abundant minerals in the joint fillings. The quartz is in silt-size grains that apparently were derived primarily from the Shnabkaib Member and deposited in the joints. I was careful to sample only the fillings of the joints and not any of the wall rock.

In the clay separates, the most abundant mineral is illite, followed by kaolinite and smectite. In color, the joint fillings are pale red and grayish red (because of hematite pigmentation) and pale olive.

I believe the joints were originally mostly closed and filled with gypsum (primarily) and calcite (to less extent than gypsum). Now, many of them are open or partly filled with the minerals noted in Appendix 3. Gypsum is highly soluble in water (see Blount and Dickson, 1973), several hundred times more soluble than calcite. In gypsum-carbonate terrains, the gypsum goes into solution, leaving a karstic topography of cavernous and pillared limestone and/or dolomite.

I measured the orientation of 65 joints (Table 3). These were all near the centerline of the dike and downstream for 600 feet. This data is supplementary to measurements by Mr. Chad Gourley and others that gathered data for him.

There is a systematic joint pattern in the area. One set has an orientation of N14°W (mean with standard deviation of 7.8°), based on 25 measurements. These joints most frequently contain the joint fillings (mineralogy in Appendix 3) and they contain relicts of the grout injected into the dike after its construction. Locally, all three of the joint sets may contain joint fillings (the red and olive material) and grout, but this set most frequently contains them.

The second set is oriented about N78°W (mean with standard deviation of 7.8°), based on 24 measurements. The scatter of measurements is greatest for this set. These joints (fractures) form conjugate fractures with the N14°W joints. In these instances, one could interpret the principal stress direction to be about N46°W with the two fractures intersecting in the intermediate stress direction. (By coincidence, the Quail Creek Dike is oriented at N41°W).

The third joint set is oriented at N29°E (mean with standard deviation of 3.7°), based on 16 measurements. This set is most spectacularly exposed in the small, southeast-dipping hogbacks downstream southwest of the dike. Frequently, red and olive gypsum crystals and calcite occur on these joint faces, marking them so they are easily seen at a distance.

The various rocks of the Shnabkaib Member do not joint uniformly. The gypsiferous micrite and microspar of unit #4, for example, are brittle and there are many joints in a small area. At five places near the centerline of the dike and downstream for 200 feet, I measured the following joint frequencies within 10 feet (noted approximately parallel to strike and for a stratigraphic thickness of about 2 feet): 46 (joints), 28, 30, 26, 18. These are of the same order as the number of joints per unit area (10 ft. x 2 ft.) I found in grayish red, silty (thin laminae of dolomitic siltstone) micrite and microspar in unit #6: 22 (joints), 24. In contrast, at three places in unit #5, where the rock is silty gypsum or interlaminated gypsum and siltstone, there are fewer joints in a similar measured area: 4 (joints), 10, 8.

In an attempt to get some feeling for the frequency of jointing by volume in unit #4, I noted the number of joints in a block 10 feet wide (parallel to strike), 6 feet high (stratigraphic thickness), and 10 feet deep (down the dip). On the exposed stratigraphic face there 30 joints (top), 25 (middle), and 19 (bottom). On the dip slope (top surface of block), there are 33 joints. The base of the block or the sides are not exposed. There are two particularly large joints. One of these, whose orientation is N75°W, is partly open with joint width up to 9 inches. The other one, which is oriented at N74°W, has open widths to 1.2 inches. Numerous very small joints are closed and narrow (0.1-0.2 inches wide). Two joints that cut across the top are up to 0.5 inches wide and open.

Many large joints are persistent for tens of feet or more. But it is not possible to make detailed measurements because of the nature of the outcrops.

I have not determined the number of intersections per unit area, an important consideration. Many of the larger joints do intersect and are conjugate. There are places where the three main joint sets (Table 3) intersect and contain the pale red and pale olive joint fillings (Appendix 3).

TABLE 1. <u>Typical Cycles of red siltstone (gypsum cement) and nodular bedded and nodular gypsum in Shnabkaib Member</u>

	Unit #6				Unit #7			
Nodular Gypsum	2.3	1.5	1.2	2.3	1.8	5.0	3.5	3.5
Red Siltstone	4.7	3.0	2.4	1.5	2.9	7.5	6.0	2.7
Total Thickness (in.)	7.0	4.5	3.6	3.8	4.7	12.5	9.5	7.2

TABLE 2. <u>Paleocurrent directions measured from ripple marks in Unit #7. Measurement is of direction of flow of current that formed the ripple mark set.</u>

Major Direction	Minor Direction
S65°W	N41°W
S89°W	N37°W
S75°W	N33°W
S81°W	N27°W
S68°W	
S79°W	
S84°W	
S62°W	
S63°W	
S57°W	
S73°W	
S73°W	
S73°W	
S85°W	
S90°W	
S83°W	
S79°W	
S72°W	
S51°W	

TABLE 3. Measurements of Joints in Quail Creek Dike Area

Set #1 (N=25)	Set #2 (N=24)	Set #3 (N=16)
N12°W	N75°W	N33°E
N13°W	N87°W	N29°E
N15°W	N90°W	N36°E
N16°W	N74°W	N34°E
N21°W	N86°E	N28°E
N12°W	N82°W	N28°E
N15°W	N72°W	N33°E
N16°W	N75°W	N27°E
N19°W	N78°W	N30°E
N13°W	N75°W	N22°E
N18°W	N74°W	N30°E
N 7°W	N77°W	N26°E
N18°W	N75°W	N24°E
N 1°W	N90°W	N27°E
N 3°W	N76°W	N28°E
N18°W	N73°W	N30°E
N20°W	N76°W	
N11°W	N89°W	
N20°W	N85°E	
N 9°W	N70°W	
N19°W	N72°W	
N18°W	N68°W	
N10°W	N75°W	
N 8°W	N74°W	
N18°W		

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APPENDIX 1

DESCRIPTION OF PETROGRAPHIC SAMPLES
SHNABKAIB MEMBER OF MOENKOPI FORMATION
(see the stratigraphic column prepared by Chad Gourley for thicknesses of units and general descriptions)

QC-1: From unit #1; about Sta. 7+00; abundant joints (mostly closed); common gypsum veinlets; shrinkage cracks.

DOLOMICRITE: Rock is about 80 percent pale olive (10Y 6/2) and 20 percent grayish red (5R 4/2 and 10R 4/2) in color; very thinly laminated; micaceous; some mica is biotite; there are thin laminae of siltstone; soft-sediment deformation.

QC-2: From base of unit #2; sample from Sta. 6+40; 200 feet downstream of dike.

A fault of small displacement (less than 1 ft) cuts across unit #1 and into unit #2; fault Zone is 2 to 14 inches wide.

Nodular gypsum that is about 4" thick occurs in the uppermost foot of unit #2; gypsum has moved into the fault Zone as well.

GYPSUM (50%) and DOLOMICRITE (50%): pale olive and yellowish gray (5Y 7/2); dolomicrite contains very thin siltstone laminae; thin stringers of dolomicrite (tiny lenses and "plates") parallel with bedding; dolomicrite is slightly micaceous.

QC-3a: From top of unit #3; near 3-4 contact; Sta. 6+30, 200 feet downstream of centerline.

SILTSTONE: Very gypsiferous and dolomitic; mixed terrigenous-carbomate rock; color is pale red (10 R 6/2) dominantly but some is grayish olive (10Y 6/2); very thinly laminated and wavy bedded; micaceous; silt grains are angular to subrounded (dominantly subangular); rock classified as subarkose; there are intraformational pebbles of dark reddish brown (10 R 3/4) claystone; about 20% gypsum in rock, including gypsum cement.

Gypsum veins constitute about 10% of rock.

QC-3b: From base of unit #3; 6+50, 200 feet downstream; in the field, there appears to be at least 50% gypsum in unit #3.

DOLOMICRITE (70%), GYPSUM (30%): rock color is yellowish gray and pale olive; dolomicrite contains silty laminae and pockets of silt; gypsum occurs dominantly as cross-cutting veins; fossils in micrite.

QC-4a: From lower part of unit #4: 2 feet below 4-5 contact; 6+15; 200 feet downstream of centerline; there is much nodular gypsum in this unit; also, abundant veins of gypsum.

DOLOMICRITE: pale olive, gypsiferous, silty laminae; quartz (subangular), feldspar, and mica in siltstone laminae; about 10% gypsum in this thin section.

QC-4b: From upper part of unit #4; Sta. 6+07; 200 feet downstream of centerline.

DOLOMICRITE: light olive gray (5Y 5/2); gypsiferous; sparse laminae formed by mm-thick layers of mixed terrigenous-carbonate silt grains; quartz (subangular), feldspar, and mica in siltstone laminae; not as silty as sample QC-4a; less than 3% gypsum in thin section.

QC-5: From unit #5; Sta. 5+83; 200 feet downstream.

Silty GYPSUM: pale olive and light olive gray (5Y 5/2); silt is mostly quartz (subangular to subrounded), but there is minor feldspar, mica and opaque minerals; silt is abundant and uniformly present through the rock; it is enclosed in poikilotopic gypsum; gypsum may be recrystallized syndepositional nodules.

QC-6: From unit #6; sample from Sta. 5+80; 150 feet downstream of centerline.

DOLOMICRITE: grayish red (5R 4/2 and 10R 4/2); some of the dolomicrite is silty but most is not; there are, however, many thin laminae of dolomitic siltstone; laminae are lenticular and wavy-bedded; quartz grains are subangular to subrounded, feldspar, mica, and opaque grains in siltstone.

QC-7: From unit #6; approximate Sta. 5+80; 150 feet downstream of centerline.

Interlaminated DOLOMICRITE (40%), SILTSTONE (30%), and GYPSUM (30%); yellowish gray (5Y 7/2: dolomicrite is gypsiferous and silty; siltstone is dolomitic and gypsiferous; silt grains are quartz (subangular to subrounded), which is predominant, and feldspar, mica, opaque grains; gypsum cement; bedding of rock is lensing, wavy; perhaps total of 50% gypsum in rock.

QC-8b: From unit #8; 4+80

Dolomitic SILTSTONE and silty DOLOMICRITE; grayish red and pale red; silt grains are dominantly quartz (subangular, subrounded) with minor feldspar, mica, opaque grains; gypsum cememt; pieces of dolomicrite floating in siltstone; bedding is disturbed.

QC-15a: From top 1 foot of unit #4; sample from Sta. 6+30: 20 feet downstream from centerline of dike; compare with sample QC-4b.

DOLOMICRITE: pale olive; bedding is lensing, wavy; rare filled fractures; rock contains 20% gypsum in the thin section; there are thin laminations and lenses of dolomitic silstone; note that there is 30% gypsum here compared with less than 3% gypsum in sample QC-4b.

QC-16: 6+10, 5 feet downstream from centerline; sample is from 1.5 feet above the base of unit #5; compare with sample QC-5.

Silty GYPSUM: pale olive; thin section is remarkably similar to QC-5.

QC-17: 6+10, 5 feet downstream from centerline; sample is from 1.5 feet above the base of unit #6; compare with sample QC-6.

DOLOMICRITE: grayish red (5R 4/2 and 10R 4/2); most is very silty, micaceous, but some is not; bedding is lensing, wavy; soft sediment folding and faulting: fractures common (filled); about 70% of thin section is this rock. SILTSTONE (30%); grayish red and pale olive; dolomitic; quartz is subangular, subrounded; a lot of it is smaller than coarse silt; siltstone contains trace of feldspar; also, mica and opaque grains; cemented with gypsum and small amount of micrite.

APPENDIX 2

SUPPLEMENTAL X-RAY DIFFRACTION STUDIES

<u>QC-5:</u> Sample is from unit #5; Sta. 5+83; 200 feet downstream of centerline; rock color is pale olive (10Y 6/2) and light olive gray (5Y 5/2)

<u>Results:</u> (run of whole sample)

a. Gypsum (most abundant mineral)
b. Quartz
c. Calcite (minor)

<u>QC-6:</u> Sample is from unit #6; 5+80; 150 feet downstream of centerline; rock color is grayish red (5R 4/2) and 10R 4/2)

<u>Results:</u>

a. Dolomite (most abundant mineral)
b. Quartz (minor)
c. Gypsum (very minor)

APPENDIX 3

SAMPLES OF MATERIALS COLLECTED FROM JOINT FILLINGS

X-Ray Diffraction Studies

QC-13: Color of fill is pale red (10 R 6/2, 5R 6/2); Sample is from joint plane which becomes a fault plane with very small displacement (less than 1 foot); approximate Sta. of sample 6+70; - 615 feet downstream of centerline; from unit #2

Results: (run of whole sample)

a. Gypsum (most abundant mineral)
b. Dolomite
c. Quartz (minor mineral)
(clay separate)
a. Illite
b. Kaolinite
c. Smectite (minor)

QC-14: Color of fill is pale red and pale olive (10Y 6/2); sample is a joint filling from a joint that is 6 inches wide in places; unit #2; near QC-13

Results: (run of whole sample)

a. Calcite (most abundant mineral)
b. Gypsum
c. Quartz (minor)
d. Dolomite (minor)
(clay separate)
a. Illite (complex)
b. Kaolinite
c. Smectite (probably interstratified Illite/Smectite)

QC-18: Color of fill is grayish red (5R 4/2 and 10R 4/2); from joint in unit #4; Sta. 6+10; near centerline of dike.

Results: (run of whole sample)

a. Calcite (most abundant mineral)
b. Quartz (minor)
c. Gypsum (minor)
d. Dolomite (minor)
(clay separate)
a. Illite
b. Kaolinite
c. Smectite

APPENDIX 4

SAMPLES OF "CLAY" FROM DAM (ZONE II MATERIAL)

("THE PURPLE CLAY")

Note: Material is originally from a quarry in the Triassic Chinle Formation supposedly about 3 miles upstream on the Virgin River.

<u>X-Ray Diffraction Studies</u>

1. Dam Core (run of whole sample)

    a. Quartz (most abundant mineral)
    b. Calcite
    c. Illite (all of the peaks remaining after quartz and calcite match illite, but the 10° peak (8.85 2 0) does not appear).

Runs of clay separates (2 patterns; air dry and 250° to confirm Smectite)

    a. Smectite (almost entirely)
    b. Kaolinite (very minor)

2. Upstream Cutoff Trench

Clay separate (3 patterns; air dry, 250° to check Smectite, 550° to check Kaolinite)

    a. Smectite and Kaolinite are roughly equal

3. Cutoff Trench, centerline of dike

Clay separate (1 pattern; air dry)

    a. Illite (only)

APPENDIX 5

SAMPLES OF MATERIAL COLLECTED AT DIKE

X-Ray Diffraction Studies

1. Toe-of-Dike (unconsolidated material coating pebbles)

   a. Calcite (most abundant mineral)
   b. Quartz
   c. Gypsum (minor)

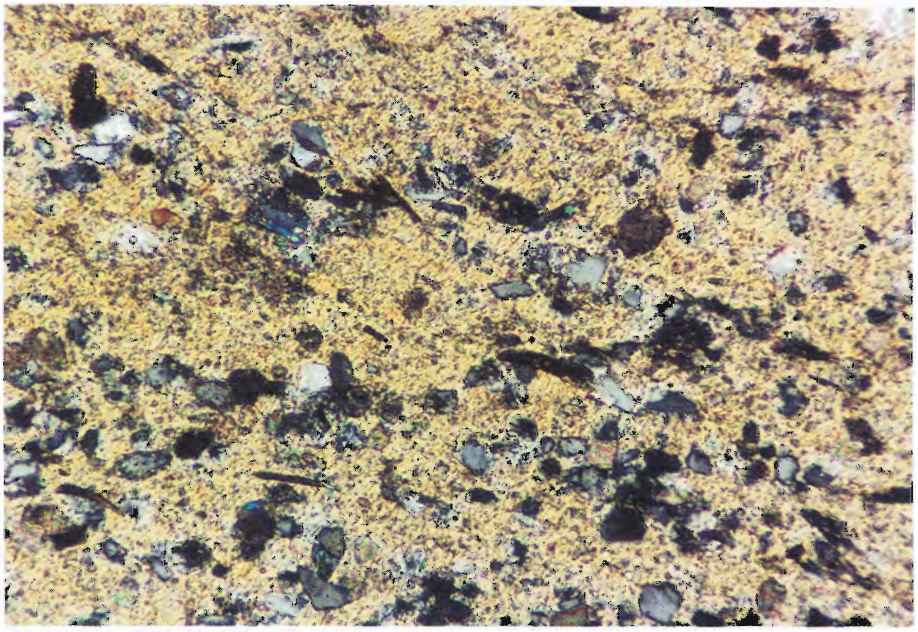
2. Pipe Sample (unconsolidated white material from inside a pipe)

   a. Quartz
   b. Gypsum
   c. Calcite (minor)

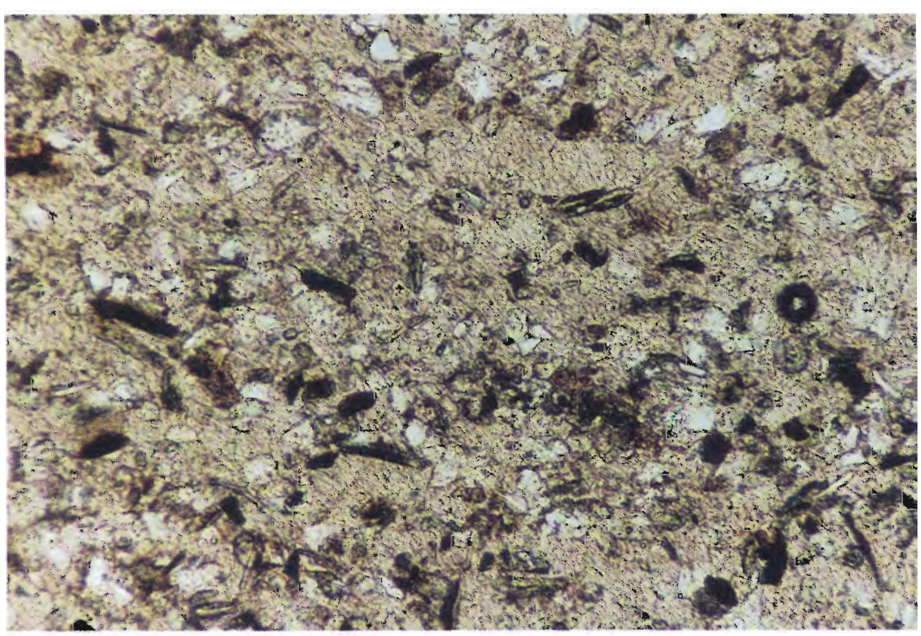
Note: Both the toe-of-dike and the pipe samples have major peaks for 9.7, 5.57, 3.82, and 2.19 A. These do not seem to match any commonminerals. The best guesses from the search manual are Paragonite and/or Ettringite, but neither of these is definite. A lot of Ettringite peaks match, especially in the toe-of-dike sample. Paragonite is questionable because the 100 intensity peak for 4.44 A is missing in both samples. Most other peaks match up.

Paragonite is a sodium mica, corresponding to muscovite in composition. The composition of Ettringite is perhaps 6CaO. Al_2O_3.350_3 33 H_2O). It is known from limestone inclusions in lava at Ettringen and Mayen in Rhineland. Also, from Tombstone, Cochise County, Arizona. Both of these minerals seem unlikely choices.

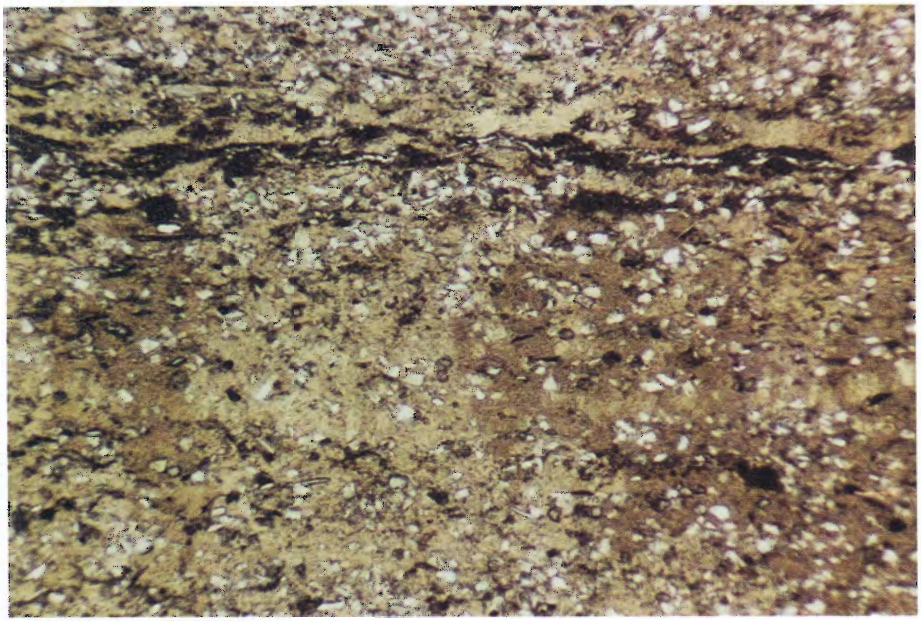
List of Photographs



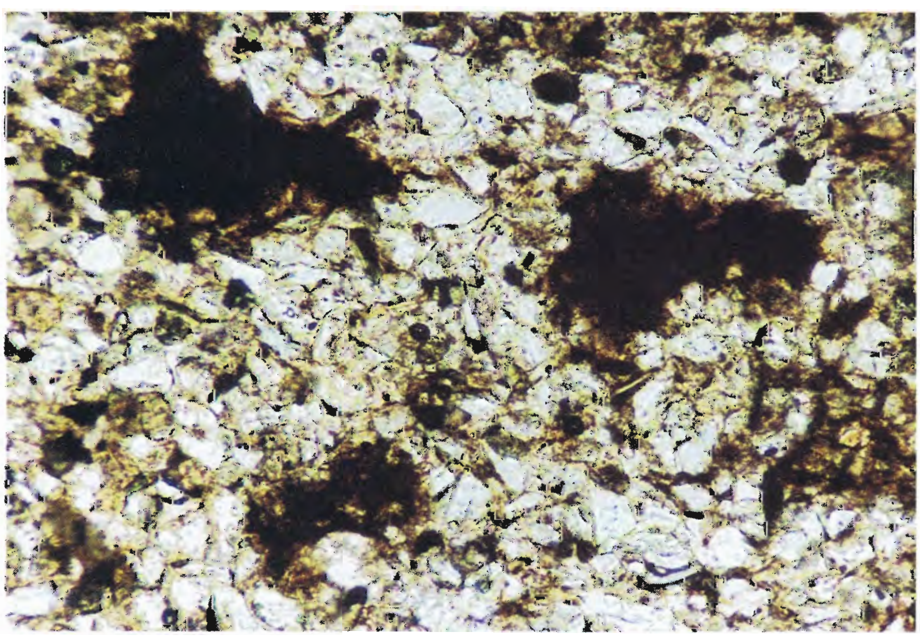
F 1. Poikilotopic gypsum, thin section QC-5.
Width of photomicrograph is 1 mm. Crossed polars.



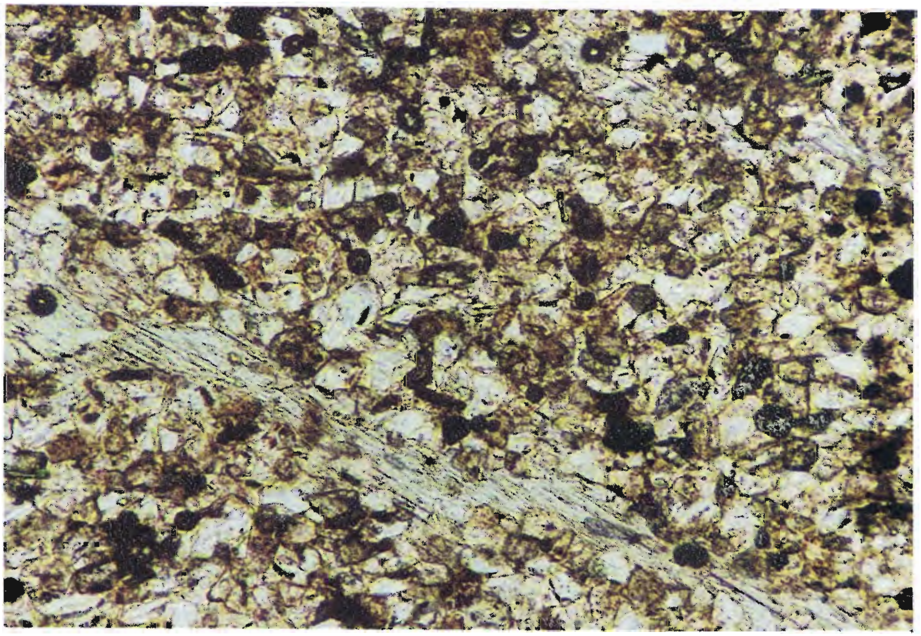
F 2. Siltstone fabric with gypsum cement. Gypsum is more abundant than silt grains. QC-16. Width is 1 mm. Plain light.



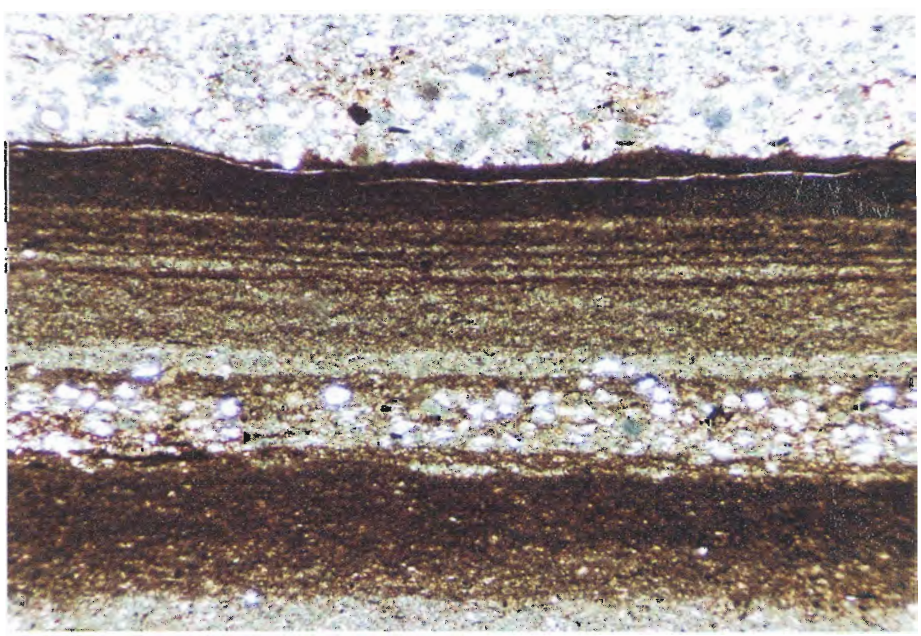
F 3. Gypsum veins and gypsiferous siltstone laminae, QC-5. Width of view is 2.5 mm. Plain light.



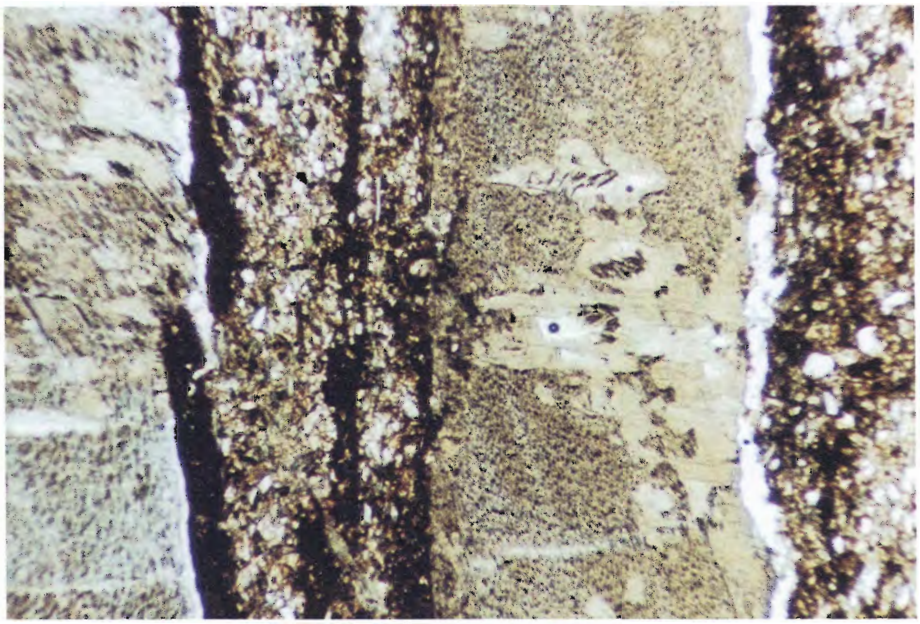
F 4. Red siltstone cemented by poikilotopic gypsum, QC-10A. Width is 1 mm. Plain light.



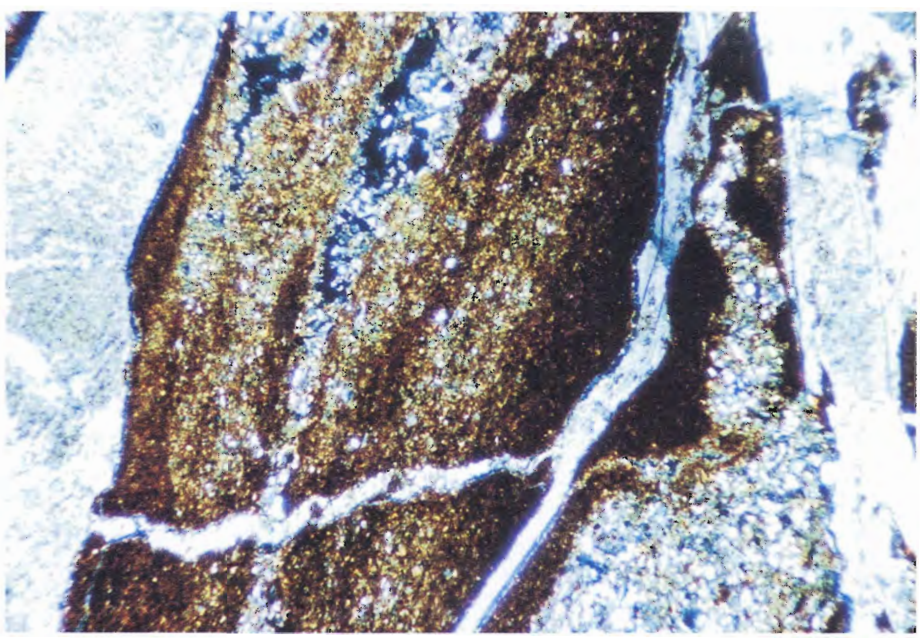
F 5. Another view of QC-10a. Note gypsum veins. Width is 1 mm. Plain light.



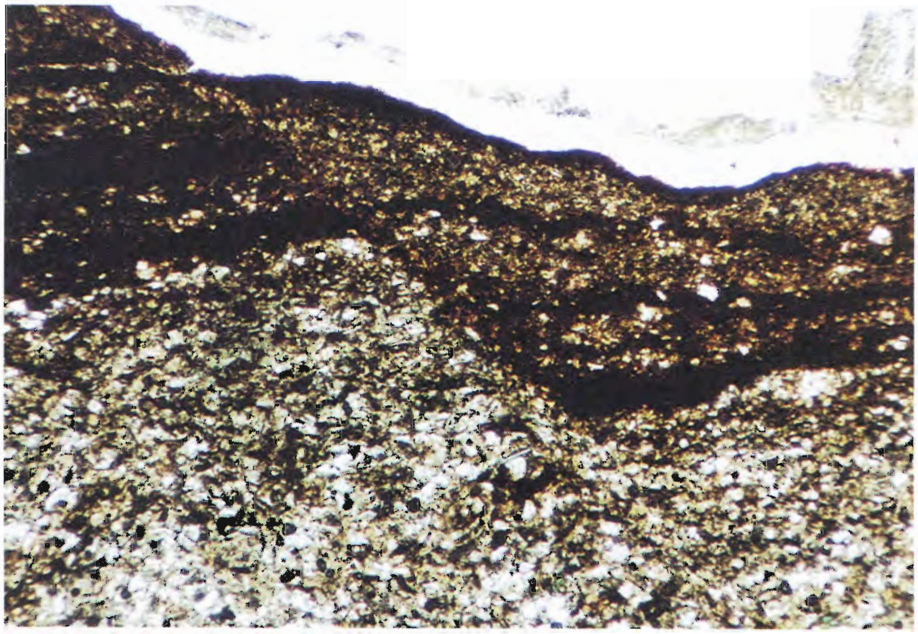
F 6. Laminae of siltstone and dolomicrite, QC-17. Width is 2.5 mm. Plain light.



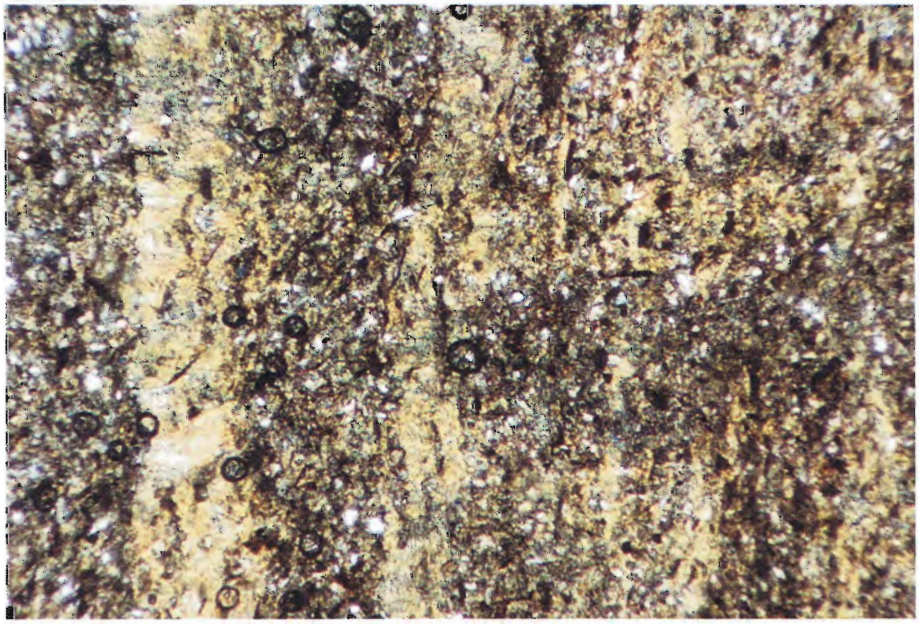
F 7. Network of gypsum veins across red siltstone, QC-10a. Width is 2.5 mm. Crossed polars.



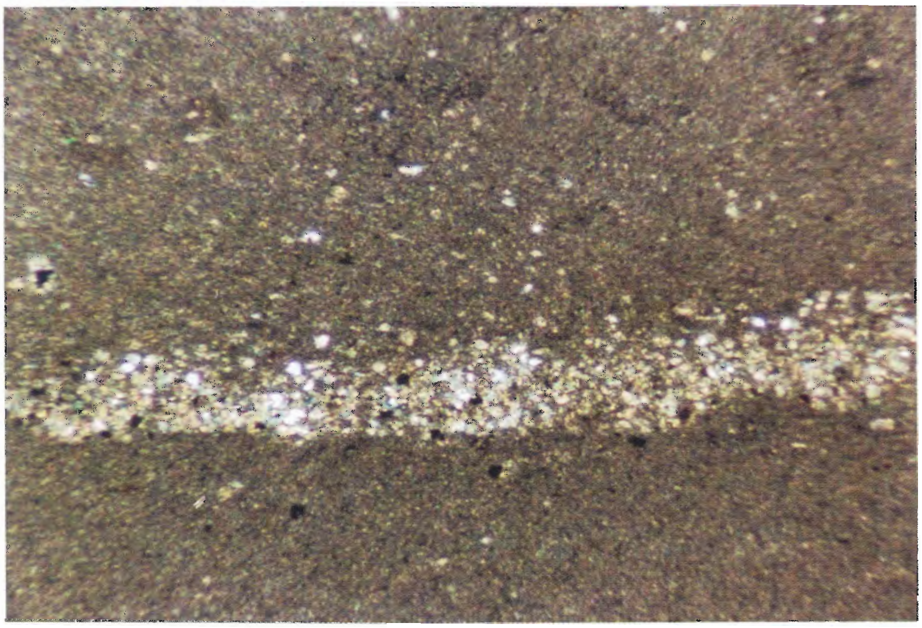
F 8. Network of gypsum veins across red siltstone, QC-10a. Width is 2.5 mm. Crossed polars.



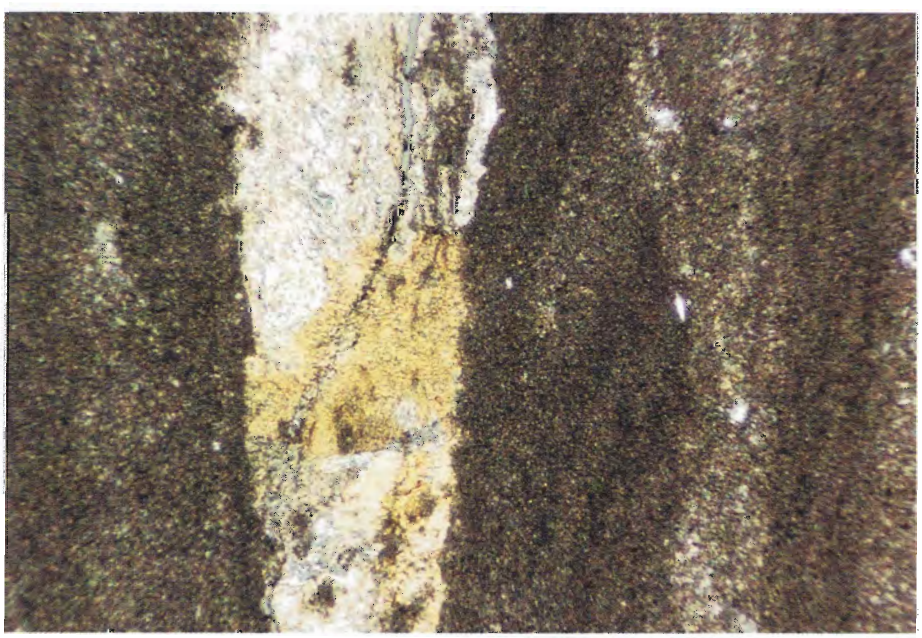
F 9. Microfault, QC-10a. Width is 2.5 mm. Plain light.



F 10. Gypsum vein in silty microspar of QC-16
from unit #5. Width of view is 2.5 mm. Plain light.



F 11. Graded siltstone laminae in dolomicrite, QC-46.
Some dolomicrosparite. Width is 2.5 mm. Crossed polars.



F 12. Gypsum vein in dolomicrite (and dolomicrosparite), QC-15a. Width is 2.5 mm. Plain light.

APPENDIX D

Ronald Thompson Memorandum

MEMORANDUM

TO: File

FROM: Ronald W. Thompson

RE: Quail Creek Dike Failure

DATE: January 2, 1989

Memorandum regarding the failure of the Quail Creek Dike on December 31, 1988. Between 10 a.m. and 10:30 a.m. December 31, 1988, I had finished a meeting with the Bench Lake Irrigation Company and invited Wayne Wilson, Chairman of the Washington County Water Conservancy District to ride with me and discuss the events of that meeting out around the Quail Creek Reservoir which is a few miles away from the location of the meeting in Hurricane, Utah.

I had passed the reservoir on the highway between Hurricane and St. George early in the morning and had watched the drain water near the highway and noticed that it was clear and there was no turbulence whatsoever in the water and I have done that just out of habit over the last several years. As we drove back, I again noticed the drain water near the highway. There was no sign of any turbulence in the water at approximately 10 a.m. I also noticed the color of the drain water near the Jones-Early road and there was no sign of any discoloration in the water.

As we approached the base of the dike on the road, I observed a flow of water which I would estimate to be two to three hundred gallons of water per minute which was carrying an amount of brownish-reddish coloring. I had never observed a flow of water in this location before and was surprised to see it. There is a drain system in that immediate area and expected any seepage to be picked up in the drain if there were any and the drains have continually run clear.

Earlier in the morning as I left the meeting met with the operator, Lloyd Jessop, who does our inspection of the dam. He gave me the piezometer and seepage reports for the prior week and had read everything on the afternoon of December 30 everything appeared to be fine with no changes in seepage or increases that he was aware of.

Wayne and I then proceeded to the base of the dike to see where the seepage was coming from and observed a flow of between two to three hundred gallons a minute seeping out around a twelve inch observation well sitting at approximately Station 620 at the base (toe) of the dike. I looked down the observation well, there was water in that drain as there has been ever since it was installed but the water that was arising above or around it was not influencing that flow. It was obvious the water in the drainage well was clear and did not appear at all to be influenced by the new water coming up. The water did not appear to be running under substantial pressure at that time.

I was concerned because we had not seen water in this location. Although, we had on a prior occasion had water in this general area and had installed the drain system and also had done substantial grouting in the summer and fall of 1987 in this area, we had not observed any seepage or any indication of any problem in this area prior to this seepage.

I immediately contacted Brad Price of Rollins, Brown & Gunnell who was involved with the design and building of the facility and has been working closely with the District dealing with the seepage problems. I called him at his home and his children indicated he had just recently left to go to his office. I then called his office and he had not arrived in his office. I proceeded to Hurricane. I contacted on the way to Hurricane Craig Stratton at Stratton Brothers and indicated that I would need some equipment and some gravel.

We had a history of building a filter system when we had seepages to make sure that we did not have any material moving with the seepage. Since this was colored I was concerned about that and so made arrangements with Interstate Rock to bring a loader and gravel.

I then recalled Rollins, Brown and Gunnell and talked to Brad Price. He had just arrived at the office and I discussed with him the seepage and where it was located and told him that we were going to observe it. I told him we were building a filter which he agreed with and told us that was what we should do. I then asked him because I felt the seepage was of significant enough quantity at that time that we would need to do additional grouting to contact the grouting company we had worked with of Boyles Brothers and that I would contact him as soon as I had gone back out to the dike and evaluated the problem.

I then called our operator, Lloyd Jessup, who was not at home and asked his wife to have him come to the base of the dike. Then I went to Interstate Rock Products where Don and Craig Stratton were down at their gravel pit. They had loaded their truck and they brought a loader over to the base of the dike.

By the time I returned to the base of the dike I noticed that the flow had changed and was moving a small amount of sands and some gravel. Although the total quantity of the water did not appear to be greater which caused more concern. Although I had seen this on a prior occasion where we had a deep foundation leak which we grouted off in the early fall of 1988, substantially downstream from this location.

We then began and did build a filter system. I left the sight at about 1:30. At about that time we had built a filter. The water was coming out the filter. There was no indication of any material moving other than a slight discoloration of the water. I instructed our operator to stay with it during the course of the day. We had access to a small tractor with a loader on it and I asked him to bring that over so that if there was any additional seepage that occurred that he would have some equipment to extend the filter if necessary. Then I had Stratton's bring an additional supply of gravel over to the base of the dike.

I returned to St. George for some meetings and returned to the base of the dike at approximately between 4:30 and 5:00 p.m. At that time I met with our operator and the seepage had increased slightly. When I arrived a part of the flow had moved from the filter slightly to the east. It appeared to be an increased flow. I had the operator measure the flow in the flume downstream into which we had funnelled all the water earlier in the afternoon so that we would be able to keep track of the total amount of seepage occurring at this location and what changes were occurring.

At that time the flume had increased from the night before by approximately 1.2 second feet. We extended the filter to cover this drainage water at this additional seep that was starting just two or three feet to the east of the filter we had built. I contacted Brad Price at that time and told him that it appeared to us the flow was increasing. I had also contacted him as soon as I had become aware that there was some gravel and perhaps some clay materials moving along with the water. I had contacted him and indicated that I had a lot of concern about that. He expressed concern and that we make sure the filter was there and make sure we review the conditions through the course of the night.

He also in the meantime at my request contacted Boyles Brothers and made arrangements for them to mobilize during the earlier part of this week to come down and commence a grouting operation and attempt to seal this leak. I called him back at approximately 6 o'clock and discussed the fact that it appeared to us there was an increase. In turn we were going to monitor it very carefully, but we determined that our course of action at this time would be to monitor it during the night to keep the seepage in the filter. We would observe what would happen during the night and determine what course of action we should take the first of the week when he was able to come down.

I returned to St. George and arrived at home about 6:30 p.m. At approximately 8:20 p.m., I received a telephone call from Lloyd Jessup indicating that the flow had increased substantially and that he was very concerned about it and did not have enough material or equipment in the dark to take care of the problem. I asked him to immediately contact Interstate Rock Products to get a bigger piece of equipment and to get some additional gravel on the site. I was very concerned because there had been a substantial increase in the flow in less than a two hour period. I called Brad Price at Rollins, Brown & Gunnell who had also been contacted by Lloyd Jessup our operator. He indicated that he would come immediately to St. George and was picking up Ralph Rollins. I called Ralph Rollins to discuss the problem with him and asked him if he thought there was any possibility of a failure of the dike. They indicated that they did not believe so, but that we should monitor it carefully. However, I made a decision at that time to contact Tony Hafen, Washington County Emergency Management, to indicate that I had a concern about this seepage.

I called Tony Hafen and asked him where he would be and told him that we had a problem that did not appear to be as other seepage that we had had on the dike and that I wanted to be in constant contact with him so that if we needed to take appropriate action or a possibility of a failure that we could make sure people were notified being concerned that it was dark and a holiday.

Tony agreed to meet me out at the dike. I then travelled to the Hurricane area and made arrangements with contractors to bring a large light plant so that we would be able to see what we were doing. I called L&M Construction and asked them to bring a backhoe knowing that we would need a backhoe to do anything in this area. I then attempted to call the sheriff at his home. He was out of town at Mesquite, Nevada for the evening. I also contacted several water board members. Most of whom were not available. But I did contact Winferd Spendlove and Truman Bowler. They indicated that they would come out.

I arrived back at the dike at around 8:30 p.m. Shortly thereafter Tony Hafen arrived. We looked the leak over. It was increasing. I told him I did not want to alarm anyone but this leak was not acting as leaks had in the past and asked Tony if he would contact people in case we had problem. Within the next half hour as the seepage increased we made a decision to make a more formal contact. And at approximately 10:00 p.m. I told Tony that I thought there was a definite possibility of a failure. I could see the equipment was not making any headway in containing the leakage. What he should do to make sure people downstream were contacted and to be prepared to shut off the highway between Hurricane and St. George and take appropriate action.

At that time, Tony immediately took over in notifying people that there was possibly a failure at the Quail Creek Reservoir. At approximately 10:30, I could see that the leak had substantially increased. I had fear that the equipment working under the base of the dike would put men and equipment in jeopardy so I instructed them to move back away. At that time, I honestly believed that a failure was imminent. I asked Tony Hafen to immediately contact all of the people and tell them that a failure was imminent. We made arrangements to have the highway closed and to have all people downstream contacted that Quail Creek Dike failure was immanent.

Within the next few minutes, the seepage increased several fold. Instead of boiling up as it had been previously, it showed that it had an exact direct contact with the reservoir itself. The flow leveled up and then the embankment above the flow began to crumble. It started crumbling at approximately 11:00 p.m. By approximately midnight there had been a total breach of the dike at the reservoir.

The initial embankment started to crumble as I said at approximately 11:00 p.m. By 12:45, I went over to the boat ramp and the lake had dropped at that time approximately 8 foot in the two hour period. Approximately 1:00 a.m. the light plant we had put on top of the dike collapsed into the flow of the breach. About 2:00, Steve Creamer showed up. I had been in contact or had received a phone call from Ralph Rollins at approximately 10:00 and indicated that I thought the flow was approximately 70 second foot. He and Brad were at Fillmore at the time. I think in reflecting the flow was probably greater than 70 second feet. By the time they had reached Beaver there had been a total failure of the dam. And I instructed them that there had been a failure.

I went back over to the boat ramp approximately 45 minutes later. It appeared to me that the reservoir flow had substantially increased. The lake had dropped another 8 or 9 feet in that 45 minutes to an hour. That is 12:45 a.m. to approximately 1:30 a.m. had dropped another 8 or 9 feet. By then Gerald Stoker had showed up. I had earlier tried to contact Gerald Stoker several times and told him we had some concerns but had been unable to contact him. Creamer & Noble's staff showed up. I had earlier in the evening contacted Brent Gardner at Creamer & Noble before I was aware of the amount of seepage to discuss the possibility of increasing the total amount of water we were able to release from the reservoir.

At approximately 10:00 p.m. I had sent our operator Lloyd Jessup over to the operating building at the reservoir to open a 36" valve to increase the amount of water we had turned out of the reservoir. By the time he returned he was unable to come back and found himself stranded on the east side of the breach and he was not able to get back across the river until approximately 4:00 a.m. when some people came in on four-wheelers across from the Stratton's gravel pit to find him and bring him out. His pickup is still stranded on the east side of the breach.

Shortly after 1:00 a.m. Gerald Stoker arrived. We discussed the situation. He went over to the boat ramp to look where we were able to measure the amount of water that was escaping in terms of the flow rates. We determined by then that the flow rate over the past hour had been approximately 60,000 cfs. We then contacted Robert Morgan, State Engineer, indicating to him the events of the evening. He indicated that he would be coming down with the Governor early in the morning.

We secured the area to make sure people stayed back away so that there was no one who would be injured. The only close event we had was a kid with a camera who went up on top of the dike and got out close as the embankment started to fall. Fortunately he was able to get away before the total embankment fell. That was the only close encounter I am aware of in terms of the loss of human life that evening.

Approximately 2:00 a.m., the area was secured so that people could not drive vehicles close to it either across the top of the dike or at the base of the dike so that people were required to stay back away from the facility. We then proceeded with Gerald Stoker to observe what damage was occurring downstream. By then St. George City had sealed off everything. We proceeded to see what damage had occurred. The height of the water when we arrived at the St. George City at approximately 3:00 - 3:30 a.m. had not reached the freeway. We crossed the freeway bridge and observed the amount of water that was clearly coming up. We then travelled around. The crest of the water by the time we arrived at the river bridge south of St. George which we refer to as the twin bridges had increased substantially. They were still in tact at that time. We then proceeded back around towards Ray Schmutz' farm. At that sight I saw Jim Raeburn from the city keeping people away. We proceeded down and observed the flow of the river which was at a total peak at that time in that area. Jim indicated to us that there had been a failure of the river bridges south of St. George that connect St. George and Bloomington Hills.

We proceeded to Washington. That bridge was still in tact and it appeared that the river was starting to receed. We then drove from there to the Washington Fields Diversion. It was dark and we could see very little. It appeared that the water had covered most of the fields below the Washington Fields Diversion but we were not able to observe any noticeable damage to the canal or to the diversion dam but it was too dark to tell the extent of the damage to the Washington Field Diversion Dam at that time.

We then returned to the St. George area by the same route. When we arrived at the bridge at the Interstate, the water had come up within three or four feet of the bottom of the bridge. We proceeded across because we had set up a meeting with Gary Esplin to discuss how we were going to handle the events of the day at the city. As I met with Gary we determined that the appropriate course would be to set a news conference for 11:00 a.m. We had been notified the governor would be here at 7:00 a.m. and would take a tour of the damage. He was bringing emergency relief people with him. We would then make decisions as to what ought to happen. I then proceeded back to discuss with the engineers a little bit of what had happened. I should also indicate that prior to that Dale Gubler had come. He got in the car with me and Truman Bowler had left in his own vehicle although he was back out at the breach at approximately 5:30 or 6:00 a.m. when I returned.

I talked to the engineers and indicated that I was going to be with the governor and asked them to continue their investigation. I then proceeded to the airport with Dale Gubler and made arrangements with Commissioner Jerry Lewis who had also been out there to be up to the airport. The governor's plane did not arrive until approximately 7:30. We proceeded to the bridge on the freeway at the south interchange with the State people and looked at the damage and then crossed the bridge to the Man of War Bridge. It was obvious there had been water leave the banks of the river back towards Dr. Capel's home and some damage to several homes in that area although the water had receded by the time we were there and it appears that that bridge is in tact without damage or significant damage.

We then proceeded with the governor to the reservoir site and observed the breach. It appeared at that time which is the first time I had been back there in the daylight that the width of the breach was approximately 300 feet and then with the governor's staff proceeded back to St. George City where we held a news conference discussing what had happened, the extent of the damage and what we would do. I then had a meeting with our water board, those who had not been present and discussed the impacts of the day.

We determined that we would immediately assess the damages. In terms of direct impact to the district, the damages we were aware of at that time is that we had lost all power facilities at Quail Creek Reservoir. We had lost all the telephone connections. We have lost the water and sewer lines. Of course, lost the dike. There was no damage we could see to the Quail Creek Water Treatment Plant, however. We have also discussed with Rollins, Brown & Gunnell that we needed go check the main dam to make sure that that immediate draw down had not caused any problems to the dam which they proceeded to do. I instructed Lloyd Jessup to turn off the 36" release valve so that we would conserve in the remaining pool of water whatever water would be possible to conserve which would be 10-11,000 acre foot of water.

I then proceeded to meet with Fred Finlinson, Jim Holbrook of the law firm Callister, Duncan, & Nebeker who we have retained to assist us in this matter and indicated that we would follow a course of action to determining what went wrong which will be done between us and water resources and dam safety people.

We then determined that we would take an airplane ride to view the amount of damage. Creamer & Noble arranged to have their airplane available to us. As we flew, we flew to the dam. The breach was as I have previously indicated. The highway U-9 has been totally washed out from approximately to where the gate in the Jones-Early road was down to the bridge. The bridge appeared to be intact but substantially damaged. There is a tremendous amount of fill quantity in the river which has caused the river to back up towards the Berry Springs ranch. We then observed the course of the road is completely torn out. It also took out the gas line which had been recently been installed on the other side of road. We proceeded down the river pretty much. The water stayed in the channel of the river until the Washington Fields Diversion. This flood had totally destroyed the diversion dam it appeared from the air. I have not been on sight yet to investigate either the amount of fill or it has washed that structure out.

We then proceeded down the river. The river had washed farm land and covered farm lands on the both sides of the river. There is some equipment and livestock loss. It did not appear except right at the vicinity of the dam that there was any damage to the Washington Fields Canal. As we proceeded downstream towards the Washington Fields Bridge, the water had covered the fields on the north side of the river washing out the north approach to the Washington Bridge. It also immersed most of the fields along the south edge of the river covering them with some degree of water.

The flood channels appear to be open. It appears that there is some damage at the Johnson Diversion Dam and some farm land. It had totally overtopped the fields at the Foremaster fields. Much of the old Shirtliff Schmutz fields. The twin bridges had totally washed out. The water had covered all of the low line flood planes in that area and had gone out along the banks of the Boots Cox farm. The water had come up into the Riverside Apartments and covered a good share of the land down to the riverside along Riverside Drive although it did not come up to the two convenience stores on Riverside drive. It did put a lot of water around that storage complex and the tire store. The water did not overtop or if it did, barely overtop Man of War bridge, but the water did go around the north abutment and covered the park. Water was put into approximately 30 homes in the Bloomington area.

As we flew downstream, we observed at approximately at 3:00 p.m. the flood waters residing. The diversion dam we built for the protection of the wound fin minnows appears to be in tact from the air. There may be damages to the irrigation structures. We will have to investigate that later in the week.

We then proceeded to the Littlefield area. The water had covered most of the farms on the left hand side as we were going downstream. It did not appear to get up into the town of Littlefield much except for possibly some small stone houses along the edge of the river. I am not sure of the nature of those houses since we were flying in the air.

As we proceeded down to the Mesquite area you could see that the water had spread out. We were not able to tell that any structures had been damaged. However, the crest of the water at 3:00 p.m. was just below Mesquite. As we proceeded further down the river it was clear that the head of the flood had just barely passed the Mesquite area. Within a few miles we were to where most of the debris coming into the flood was. The flood had not yet reached Lake Mead. It was still several miles away from the lake and it was difficult to tell what damage if any would be done to structures in that area.

I then proceeded back to St. George. Our attorneys flew back to Salt Lake. I contacted Chuck Carney, our operator, and gave him certain instructions. I contacted Lloyd Jessup with instructions to set up a time to meet with him January 2, 1989. I talked to Evan Woodbury, a member of our board, regarding the Washington Fields Diversion Dam and made arrangements to meet with him January 2, 1989. I also contacted Staf Snow from the St. George Canal Company to set up a meeting with their board at 11:00 a.m. I received a telephone call from Tony Hafen who indicated we would have a meeting this morning at 9:00 a.m. at the county offices and another one at 4:00 p.m. with the Federal Emergency People. I then contacted Brent Gardner making arrangements to meet with him to work on getting the services back around the lake and also to discuss damages. I made arrangements for a helicopter to come in and we will take certain camera equipment to see if we could do a full analysis of other impacts. I attempted to contact Clark Church but have been unable to do so at this date. I have made arrangements to meet with Omar Matthews in the morning and intent to start rigorously at 8:00 a.m. trying to assess the total amount of the damages and what course of action the district should take.

APPENDIX E

Montgomery and Everitt Trench Mapping Report

DIVISION OF WATER RESOURCES
1636 West North Temple
Room 310
Salt Lake City, Utah 84116

MEMORANDUM

January 10, 1984

TO: Lee Sim
Larry Anderson
Dennis Strong

FROM: S. Bryce Montgomery & Ben Everitt

SUBJECT: Geologic Examination of Quail Creek Dike Cutoff Trench

The cutoff trench for the Quail Creek dike was examined on December 16 and 21, 1983. On the 16th the initial look was done in the company of Brent Gardner, project engineer for Creamer and Noble; Brad Price, project engineer for Rollins, Brown & Gunnell, and Gerald Stoker, area engineer for the State Engineer, and Lee Sim, project engineer for this office.

Attached to this memorandum is a copy of the field notes at stations along the trench (attachment 1), along with a plan view map and section illustrating the noted geology (attachment 2). Of major concern is the high percentage of sulfate minerals, both calcium sulfate ($CaSO_4$) and sodium sulfate (NA_2SO_4) within the bedrock of the cutoff trench and within the residual soil on both sides which will be beneath the dike embankment. The soluble salt content of the foundation rock is much higher than it appeared to be from the drill hole logs.

Except for the yellow-buff weathering, fine grained sandstone in the left abutment, all of the bedrock in the cutoff trench is the Shnabkaib member of the Moenkopi Formation of Triassic age, as mapped by Rollins, Brown & Gunnell, Inc., from surface exposures and drill hole data. The Shnabkaib member is mostly a dolomite or dolomitic siltstone containing a high percentage of intercalated laminae of sulfate rock (described as gray claystone and siltstone in the drill hole logs, D-1 through D-7). There are also beds of dark brown siltstone thinly laminated with sulfate. The sulfate laminae range in thickness from very fine (2 to 5 per millimeter with the intercalated dolomitic siltstone) to 3/8-inch thick beds. Within 10 feet of the surface, some of the sulfate laminae have expanded by recrystallization and hydration, creating voids and channels and a fluffy residual soil.

Chemical analyses of leachates of both the densely laminated dolomitic, gypsyferous siltstone and the expanded weathered rock were conducted by Ford Laboratories (attachment 3). The analyses show that the unweathered rock is at least 9% soluble salt, mostly sulfates of sodium and calcium in roughly equal proportions by weight. Moisture, assumed to be mostly water of hydration bound in the sulfate minerals, was determined at 3 1/2% by measuring the loss in weight during heating.

The minerals composing the soluble fraction are not known, but probably are a mixture of some of the following minerals common in marine evaporitic rocks:

			INDICES OF REFRACTION		
			α	γ	β
Gypsum	-	$CaSO_4 . 2H_2O$	1.520	1.530	1.523
Anhydrite	-	$CaSO_4$	1.570	1.614	1.575
Glauberite	-	$Na_2Ca(SO_4)_2$	1.515	1.536	1.535
Thenardite	-	Na_2SO_4	1.474	1.484	1.477
Mirabilite	-	$Na_2SO_4 . 10H_2O$	1.393	1.397	1.395

The optical properties of gypsum and glauberite are very similar, and we suspect that some of what was identified in Rollins, Brown, & Gunnell's Design Report (p. 7-21) is actually glauberite.

Analysis of the weathered rock showed a lower soluble salt content, lower sodium to calcium ratio, and a higher moisture content than the unweathered rock. This is expected from leaching and hydration of sulfate minerals during weathering.

Over most of the excavation hard impervious rock has been reached at a depth of 10 feet or less. Fractures appear tightly sealed with sulfate. This confirms the drill hole tests, most of which show excellent recovery and low permeabilities below 10 feet. The low rock quality shown by the drill hole logs we believe is due to the failure of the rock core along the thin sulfate laminae during drilling.

Several zones of intense fracturing with gypsum-filling were observed within the fresh cut of the excavation trench. Some of these zones between Stations 17+25 and 17+70, and Stations 18+40 and 18+70 are faults. Obviously local warping and shearing of beds with associated fracturing has intensified the emplacement of gypsum within the produced openings. (This has also taken place within a 10-foot thick sandstone bed within the upper section of the yellow sandstone, in the left abutment of the dike at Station 1+20.)

As shown on the attached geologic map and section, the bedding strikes generally northeasterly with a southeast dip of 6 to 30 degrees, but with a great deal of local variation due to warping and shearing. Throughout the full length of the dike, except at the fault zone between Stations 18+40 and 18+70, all of the bedding is dipping southeastward. Thus, the crest of the major structure of the area, the Virgin Anticline, is west of the right abutment of the dike, near Station 23+50. West of here the bedding begins to gently dip to the northwest within the high hogback ridge.

Attachment 4 presents a detailed sketch of the south wall of the cutoff trench, where a deformed zone comes to the surface. Here fractured and sheared zones form soft pockets in the floor of the trench. Although soft, the rock is compact, with no open fractures. No drill holes penetrated this zone, but the Geologic Map of Rollins, Brown & Gunnell shows an area of gypsum at the surface.

This deformed zone is the weakest part of the dike foundation between 2+00 and 20+00. Fortunately it occurs high on the right abutment where the base of the dike will be under only about 15 feet of hydrostatic pressure.

CONCLUSIONS AND RECOMMENDATIONS:

The bedrock within the foundation and cutoff trench of the Quail Creek Dike contains at least 9% water-soluble salts, which have heretofore assumed to be anhydrite and gypsum but which are now shown to be as much as 50% sodium sulfate. At ordinary temperatures (20^{o}C.) sodium sulfate is 100 times more soluble than calcium sulfate.

It is imperative that no water be allowed to circulate through this material, in order to minimize the initial solution and removal of the soluble salts.

Presently the small fractures in the floor of the cutoff trench are filled with gypsum and appear to be tight. The trench crosses fault zones between Stations 17+25 and 17+70, and Stations 18+40 to 18+70, where permeable fractured rocks extend below the present grade. These areas should be excavated of soft rock as deep as possible, by hand if necessary.

It is our recommendation that the design and construction of the dike provide for later extension of the grouting from the left abutment throughout the complete foundation of the dike, as sulfate-filled fractures may become opened with time. Drainage from the dike abutments, foundation, and embankment should be monitored continuously as the reservoir is operated, as to chemical content, amount and head. By comparing the salt content of the drainage water initially upon filling the reservoir and thereafter a comparison can be made which will indicate the volume of salt being removed at various locations with time.

S. Bryce Montgomery
S. Bryce Montgomery

Benjamin L. Everitt
Benjamin L. Everitt

Attachments

cc: Creamer & Noble

Field Notes: Geologic examination of cut-off trench in Quail Creek Dike by S. B. Montgomery & B. L. Everitt, December 16 and 21, 1983

Sta 1+00 and eastward: Maroon-rust brown, thin to medium bedded shaley sandstone. Beds strike N53^{O}E, dip 30^{O}E.

Sta 1+100 - 1+20: Yellow-buff weathering, fine grained sandstone, moderately hard, forming a ledge.

Sta 1+20 - 1+30: Yellow-weathering fine grained sandstone containing intercalated seams of gypsum parallel to bedding up to 1/2-inch thick and joint-fracture filling across bedding up to 1/2-inch thick. Beds strike N52^{O}E, dip 30^{O}SE. Prominent joints strike N17^{O}E, dip 82^{O}NW. These joints are open to 1/2-inch wide at the weathered surface with loose sand within. This is probably the same gyppy zone penetrated in DH 1-D, 36-38'.

Sta 1+30 - 1+85: Yellow-weathering fine grained, thick-bedded, ledge-forming sandstone within slope of 62^{O} from horizontal. Beds strike N40-45^{O}E, dip 30-33^{O}SE. Sta 1+85 is 18 feet up the 62^{O} slope from the bottom of the slope and cutoff trench. The basal few feet of this interval is actually soft, yellow-weathering siltstone to very-fine-grained sandstone.

Sta 1+85 - 1+93 in steep slope and 1+93 - 2+35: Rust red-brown-maroon siltstone and silty shale containing few gypsum veinlets to 1/8-inch wide. Beds strike N40^{o}E, dip 30^{o}SE.

Sta 2+35 - 2+80: Eastward dipping, hard, rust-brown, gyppy siltstone with intercalated seams of white gypsum to 1/4-inch thick about every 2-5 inches apart.

Sta 2+80 - 3+00: Interbedded olive green-maroon-yellow brown shaley siltstone and very fine grained sandstone, soft, with gypsum seams and cross-veinlets to 3/8-inches thick and 1-4 inches apart.

Sta 3+00 - 3+19: Interbedded very fractured and bent, thin beds (1/2-1 inch thick) of maroon-gray siltstone with intercalated gypsum seams to 1/2-inch thick; apparent slippage has occurred along bedding. Beds strike N35^{o}E, dip 31^{o}SE. At Sta 3+16 gypsum has been leached-out along fractures down to depth of 10 feet (photo).

Sta 3+19 - 3+37: Maroon siltstone, thin bedded having gypsum seams to 1/2-inch thick. Joints strike N15^{o}W and vertical dip, and N77^{o}W with a 88^{o}N dip, spaced 4-12 inches apart. At Sta 3+33 beds strike N40^{o}E, dip 29^{o}SE.

Sta 3+37 - 3+60: Shaley siltstone and silty soft shale; maroon and gray with fractures filled with gypsum to 1/2-inch thick. At Sta 3+50 is shearing along bedding.

Sta 3+60 - 4+10: Hard beds of gray-maroon siltstone containing intercalated and wavy laminae of anhydrite and gypsum. Beds strike N40-60°E, dip 19°SE and wavy. Some crinkled bedding with much gypsum filling especially at Sta 3+84 - 4+10 which appears to have experienced shearing and thrusting (photo). Joints strike N20°W, dip 78°SW and N821°W, dip 74°N, spaced 4-12 inches apart, open to 1/4-inch near the surface but closed with gypsum filling in bottom of trench. At Sta 3+70 beds strike N42°E, dip 20°SE.

Sta 4+10 - 5+15: Soft gray and maroon silty shale with interbeds of white gypsum up to 3/4-inch thick and about every foot, being crumpled. Bedding is thin to 1-inch thick striking N45-50°E, dip 20°SE. At Sta 4+86 is a softer maroon, silty shale bed about 5 feet thick.

Sta 5+15 - 6+12: Hard beds of rust brown siltstone with interfingering seams and veinlets of white gypsum that cross bedding planes, progressing westward into gray and maroon-brown hard, dolomitic siltstone with numerous interlaminae to 1/4-inch thick of anhydrite and gypsum, which forms a ledge-ridge. Much white gypsum in veinlets to 1/2-inch thick exists within the rust-brown, underlying siltstone. Beds strike N45-48°E, dip 19-20°SE. Joints N80°W, dip 88°N.

Sta 6+12 - 6+24: Soft, maroon-rust brown shaley siltstone, fractured; weathers blocky to 1-4 inch pieces.

Sta 6+24 - 6+62: Hard gray, thin-bedded, dolomitic siltstone with numerous interlaminae and seams of anhydrite and gypsum to 1/8-inch thick. Beds break into tabular blocks. Bedding strike N50°E, dip 20°SE. Joints are N87°W and vertical.

Sta 6+62 + 6+80: Thinly bedded rust-brown and green-gray shaley siltstone, soft with intercalated gypsum seams to 1/8-inch thick spaced 1-2 inches apart, and some interbeds of very fine sandstone.

Sta 6+80 - 7+14: Gray hard, dolomitic siltstone with numerous intercalated anhydrite and gypsum laminae and seams to 1/8-inch thick; forms a ledge ridge. Bedding thickness to 4-inches but with numerous inter laminae, striking N36°E, dip 15°SE and at Sta 7+00 N58°E, dip 19°SE. Joints strike N75°E and vertical, and N15°W and vertical, with gypsum filling to 1/8-inch wide.

Sta 7+14 - 8+64: Rust brown-maroon and gray, softer, shaley siltstone forming a strike-valley; much inter-laminations of anhydrite and gypsum and veinlets of gypsum, up to 1/4-inch wide and spaced up to 1/2-2 inches apart. Joints strike E-W with a 79°N dip, spaced 6-12 inches apart, and N-S with a west dip of 74°, spaced 24-inches apart; all open to 1/8-inch near the surface. Apparent shear fractures.

Sta 8+64 - 9+20: Hard yellowish gray dolomitic siltstone containing numerous intercalated laminae and seams of anhydrite and gypsum to 1/8-inch thick; weathers blocky eventually breaking down into thin platelets due to its laminated character; forms ridge-ledge. At Sta 9+00 the beds turn to maroon-rust brown but are similar in character to the upper gray beds. At Sta 8+85 beds strike N40°E and dip 20°SE, and at Sta 9+10 strike N45°E, dip 18°SE. Joints strike N75°W with 85°S dip, and N27°E, dip 80°SW, with gypsum filling to 1/4-inch wide.

Sta 9+20 - 10+16: Gray, weathered, soft, dolomitic siltstone with openings in weathered surface from salt leaching; contains numerous intercalated laminae and seams of anhydrite and gypsum to 1/4-inch thickness. At Sta 10+00 beds strike N53°E, dip 22°SE.

Sta 10+16 - 11+69: Harder gray dolomitic siltstone with abundant intercalated laminae and seams of anhydrite and gypsum to 1/4-inch thick; well fractured with gypsum filling. Bedding strikes N56-80°E, dip 22-24°SE; and at Sta 10+40 N50°E, dip 28°SE; and at Sta 11+00 N74°E, dip 20°SE, at Sta 11+25 N79°E, dip 21°SE, and at Sta 11+40 S50°E, dip 17°SE but warped. Joints strike N25°E, dip 81°SW and N60°W, dip 70°NE, spaced 1-2 feet apart with 1/8-1/4 inch wide gypsum filling.

Sta 11+69 - 14+23: Gray, shaley, thin bedded, dolomitic siltstone with frequent inter-laminations of anhydrite and gypsum to 3/8-inch thick; softer and more weathered than at Sta 10+16; beds are crinkled with strike N80°E, Dip 10°SE. Beds are well fractured with much veinlets of gypsum to 5/8-inch wide. Small thrusts with gyp-filled fractures at 12+10 and 12+45 strike N60°E and dip SE (photos).

Sta 14+23 - 15+82: Siltstone as above and at Sta 15+32 containing numerous small, cubic crystals of pyrite to 1mm across on some bedding planes and some pyrite nodules to 1/2-inch diameter. Beds strike N20°E, dip 6°SE here.

Sta 15+82 - 16+43: Yellow-gray, soft, shaley siltstone, very fractured and broken, weathering to pieces 1/2-3 inch size; some harder interbeds and hard in bottom of trench.

Sta 16+43 - 17+27: Same material as at 15+82 - 16-43 but with warped bedding. Beds here strike N80°E, dip 17°S, and N23°E, dip 27°SE, and at Sta 17+00 N30°E, dip 5°SE. Small thrust with drayfold exposed in south wall at 17+27 strikes N55°E, dips 30°NW (photo).

Sta 17+27 - 17+79: Broken beds of thin bedded, yellow-gray siltstone, soft, buckled with associated small fault having a plane N55°E, dipping 32°NW. This fault plane has a 1/2-inch thick gouge coating of brown clay and 1/38-inch thickness of white gypsum.

Sta 17+79 - 18+10: Here is another buckle and small fault in gray siltstone that is interbedded with gypsum. The siltstone bedding is up to 3-inches thick; very fractured. The fault has gray-green clay gouge and clear gypsum crystals to 3/4-inch thick on its plane.

Sta 18+10 - 18+28: Here is another buckle in the bedding with a prominent fault, within gray-maroon, thin-bedded siltstone with interbedded gypsum seams. The beds are very broken with gypsum veinlets. The beds are dragged down to the west striking N55°E and dipping 57°NW. To the east of this the beds are near horizontal. (Between here and Sta 19+00 the beds are wavey and broken with buckling).

Sta 18+28 - 18+62: Between Sta 18+10 and 18+62 there is a very broken fault zone with very fractured yellow-gray, thin-bedded siltstone with gypsum veinlets. At Sta 18+28 is gray and maroon, thin bedded siltstone that is very fractured containing nodules of gypsum to 1-inch thick and fracture-filling of gypsum to 1-inch thick. There are also inter-beds of gypsum to 1-inch thick spaced 4-12 inches apart within the gray siltstone. Bedding strikes N50°E, dip 22°NW.

Sta 18+62 - 18+79: At Sta 18+62 the bedding is tipped-up very steeply with many interlaced, white gypsum laminae and cross-veinlets to 1/2-inch thick. Beds here strike N50°E and dip 59°SE. At Sta 18+70 is the axis of a sharp fold, steep on the east limb to horizontal bedding on the west. (Sketch and photos)

Sta 18+79: Gray siltstone beds 1/4-2 inches thick near horizontal attitude.

Sta 19+37: Small buckle within thin, gray beds of siltstone.

Sta 19+37 - 19+51: Siltstone as above. Small fold, north wall, 19+39 (photo).

Sta 19+51 - 20+06: At Sta 19+51 is harder, gray siltstone beds to 4-inches thick with thin intercalated seams to 1/4-inch of white gypsum, and cross veins of gypsum to 1-inch thick having a strike of N10°E and vertical dip. Beds at 19+51 strike N60°E and dip 12°E.

Sta 20+06 - 20+50: Thin-medium bedded gray siltstone with fractures filled with gypsum. At Sta 20+06 beds strike N60°E and dip 2°SE. Joints here are filled with white gypsum to 1/2-inch thick, being N18°W and vertical, and N76°E and vertical.

Sta 20+50: Here is an apparent small thrust fault in gray, gyppy siltstone beds. Bedding here appears to be tilted on end in the bottom of the trench.

Sta 20+70: Beds of gyppy siltstone striking N47°E with a dip of 17°SE.

Sta 21+20 on gray siltstone beds striking N46°E, dipping 11°SE.

Sta 22+56 is end stake on center-line of dike.

Sta 23+56: Here is a recently cut haul road across the dike trend line which exposes gray siltstone with near horizontal beds.

Ford Chemical
LABORATORY, INC.
Bacteriological and Chemical Analysis
40 WEST LOUISE AVENUE
SALT LAKE CITY, UTAH 84115
PHONE 466-8761

DATE: 01/04/84

CERTIFICATE OF ANALYSIS

UTAH STATE NATURAL
RESOURCES & ENERGY
1636 W. NORTH TEMPLE
SLC, UTAH 84116

84-007892

SAMPLE: WEATHERED ROCK SAMPLE RECEIVED 12-14-83 FOR ANAYSIS UNDER P.O. #621349.

	QUAIL CREEK DIKE LEFT
Alkalinity as CaCO3 ppm	240
Bicarbonate as HCO3 ppm	292.80
Calcium Sulfate as CaSO4 %	3.286
Carbonate as CO3 ppm	<.01
Chloride as Cl %	.180
Hydroxide as OH ppm	<.01
Magnesium Sulfate as MgSO4 %	.018
Moisture %	5.65
Sodium Sulfate as Na2SO4 %	3.440
Sulfate as SO4 %	5.240
Total Soluble Solids %	6.960
pH Units SM423	7.60

FORD CHEMICAL LABORATORY, INC.

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Bacteriological and Chemical Analysis

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CERTIFICATE OF ANALYSIS

UTAH STATE NATURAL
RESOURCES & ENERGY
1636 W. NORTH TEMPLE
SLC, UTAH 84116

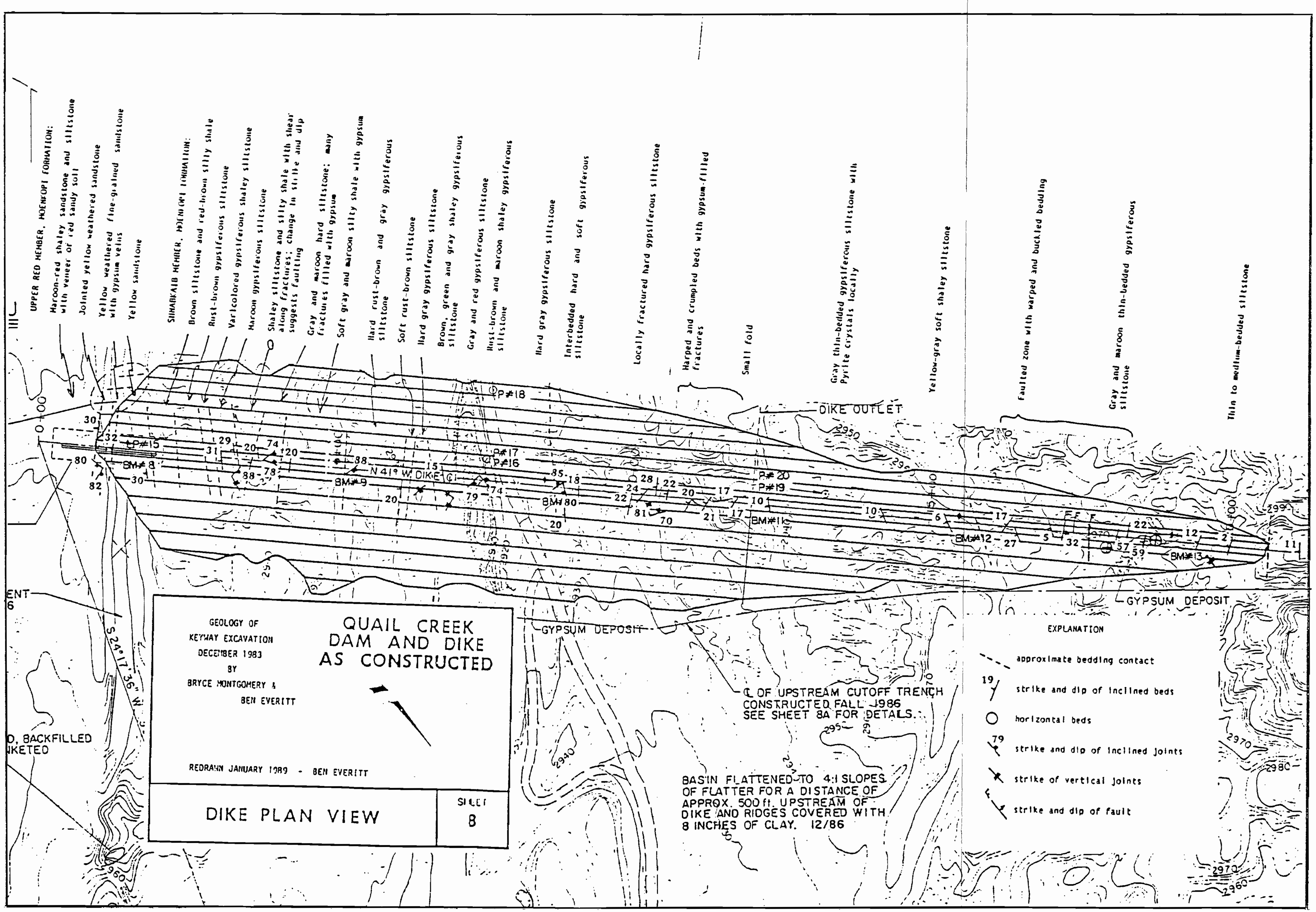
84-007891

SAMPLE: UNWEATHERED ROCK SAMPLE LABELED QUAIL CREEK DIKE RECEIVED 12-14-83 FOR ANALYSIS UNDER P.O. #621349.

RESULTS

Alkalinity as $CaCO_3$ ppm	269
Bicarbonate as HCO_3 ppm	328.30
Calcium Sulfate as $CaSO_4$ %	3.932
Carbonate as CO_3 ppm	<.01
Chloride as Cl %	.175
Hydroxide as OH ppm	<.01
Magnesium Sulfate as $MgSO_4$ %	.299
Moisture %	3.65
Sodium Sulfate as Na_2SO_4 %	4.950
Sulfate as SO_4 %	6.780
Total Soluble Solids %	8.950
pH Units SM423	7.50

FORD CHEMICAL LABORATORY, INC.



QUAIL CREEK
DAM AND DIKE
AS CONSTRUCTED
GEOLOGY OF
KEYWAY EXCAVATION
DECEMBER 1983
BY
BRYCE MONTGOMERY &
BEN EVERITT
REDRAWN JANUARY 1989 - BEN EVERITT
DIKE PLAN VIEW
SHEET
8
EXPLANATION
approximate bedding contact
strike and dip of inclined beds
horizontal beds
strike and dip of inclined joints
strike of vertical joints
strike and dip of fault
UPPER RED MEMBER, MOENKOPI FORMATION:
Maroon-red shaley sandstone and siltstone with veneer of red sandy soil
Jointed yellow weathered sandstone
Yellow weathered fine-grained sandstone with gypsum veins
Yellow sandstone
SHNABKAIB MEMBER, MOENKOPI FORMATION:
Brown siltstone and red-brown silty shale
Rust-brown gypsiferous siltstone
Varicolored gypsiferous shaley siltstone
Maroon gypsiferous siltstone
Shaley siltstone and silty shale with shear along fractures; change in strike and dip suggests faulting
Gray and maroon hard siltstone; many fractures filled with gypsum
Soft gray and maroon silty shale with gypsum
Hard rust-brown and gray gypsiferous siltstone
Soft rust-brown siltstone
Hard gray gypsiferous siltstone
Brown, green and gray shaley gypsiferous siltstone
Gray and red gypsiferous siltstone
Rust-brown and maroon shaley gypsiferous siltstone
Hard gray gypsiferous siltstone
Interbedded hard and soft gypsiferous siltstone
Locally fractured hard gypsiferous siltstone
Warped and crumpled beds with gypsum-filled fractures
Small fold
Gray thin-bedded gypsiferous siltstone with Pyrite crystals locally
Yellow-gray soft shaley siltstone
Faulted zone with warped and buckled bedding
Gray and maroon thin-bedded gypsiferous siltstone
Thin to medium-bedded siltstone
DIKE OUTLET
GYPSUM DEPOSIT
GYPSUM DEPOSIT
℄ OF UPSTREAM CUTOFF TRENCH CONSTRUCTED FALL 1986
SEE SHEET 8A FOR DETAILS.
BASIN FLATTENED TO 4:1 SLOPES OF FLATTER FOR A DISTANCE OF APPROX. 500 ft. UPSTREAM OF DIKE AND RIDGES COVERED WITH 8 INCHES OF CLAY. 12/86
N 41° W DIKE ℄
S 24°17'36" W
0+00
P#15
P#16
P#17
P#18
P#19
P#20
BM#8
BM#9
BM#10
BM#11
BM#12
BM#13
ENT
6
D, BACKFILLED
KETED

APPENDIX F

General Piping Mechanisms

GENERAL PIPING MECHANISMS

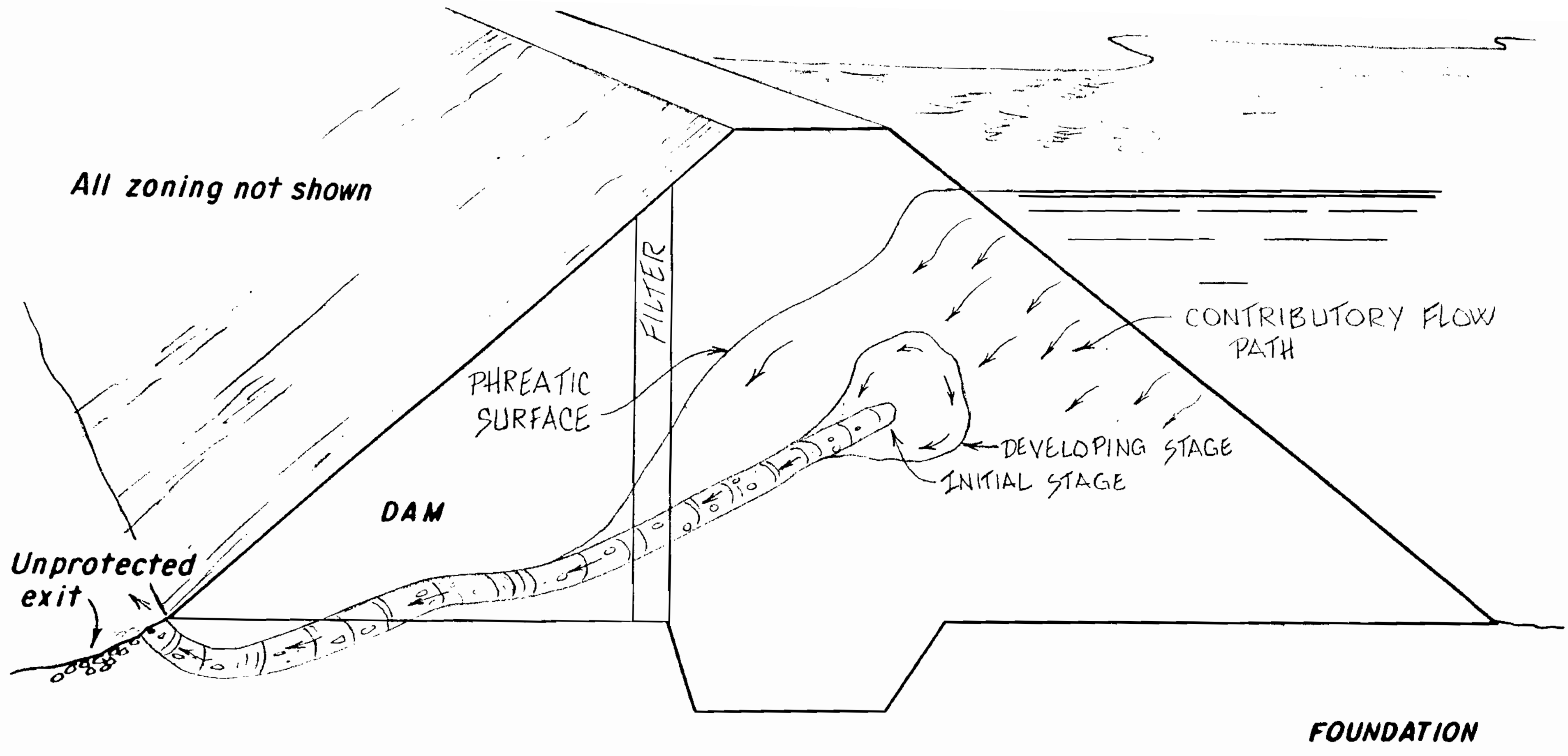
Piping may occur in or beneath a zoned embankment dam along a variety of paths. Essentially, if materials in or beneath the dam will allow piping to occur (i.e., if there are erodible materials, a roof can be supported along the exit channel and there is an unfiltered exit for the seepage), then any flow path passing through such materials will pipe. There are four general piping mechanisms for an embankment dam on a permeable foundation. These mechanisms are presented below. It should be noted that these mechanisms are not intended to represent what occurred at Quail Creek Dam but are presented to explain the process of piping and the possibilities examined.

Mechanism I. Flow through the Dam. In this mechanism piping of the embankment materials entirely through the dam occurs (see Figure F-1).

Mechanism 2. Flow from Dam to Foundation. In this mechanism piping occurs due to flow from the dam into the foundation (see Figure F-2).

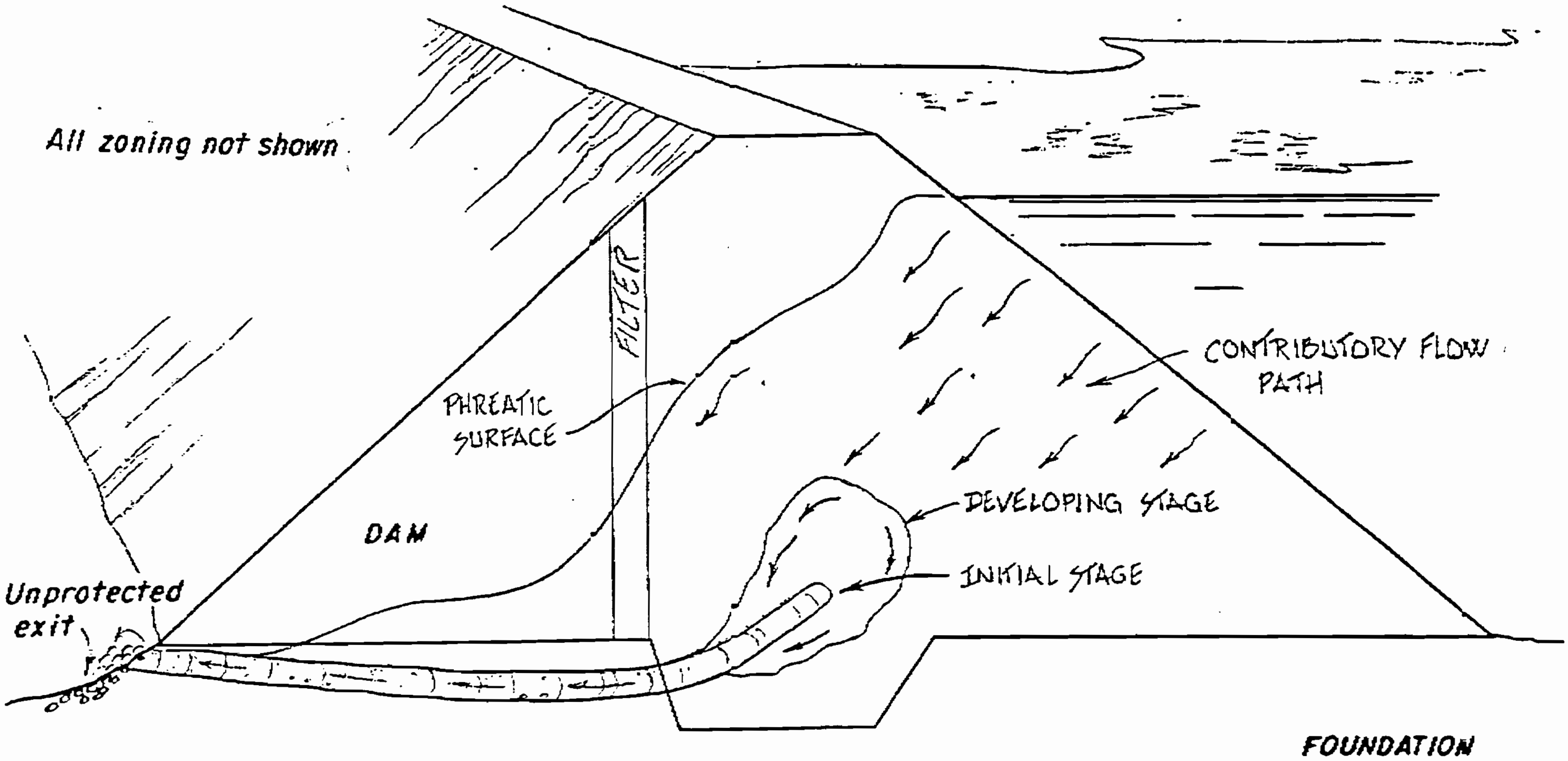
Mechanism 3. Flow along the Interface. In this mechanism piping of materials occurs along the interface of the dam and foundation (see Figure F-3).

Mechanism 4. Flow through Foundation. In this mechanism materials are piped through a conduit in the foundation (see Figure F-4).



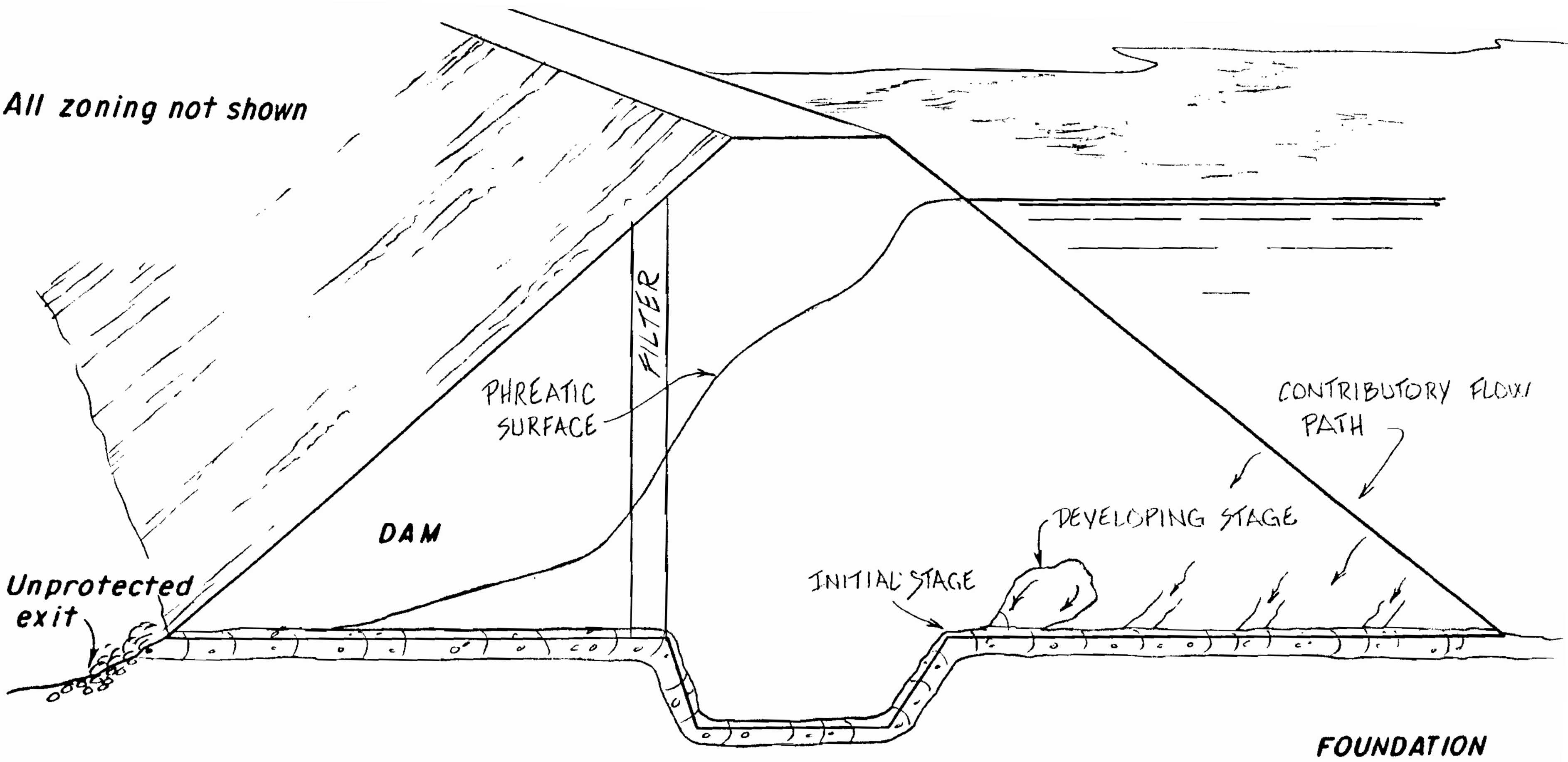
NOTE: As piping progresses upstream the hydraulic gradient increases, as the hydraulic gradient increases flow volumes and velocities increase upstream piping potential.

GENERAL PIPING MECHANISMS
MECHANISM — I
FIGURE F — I



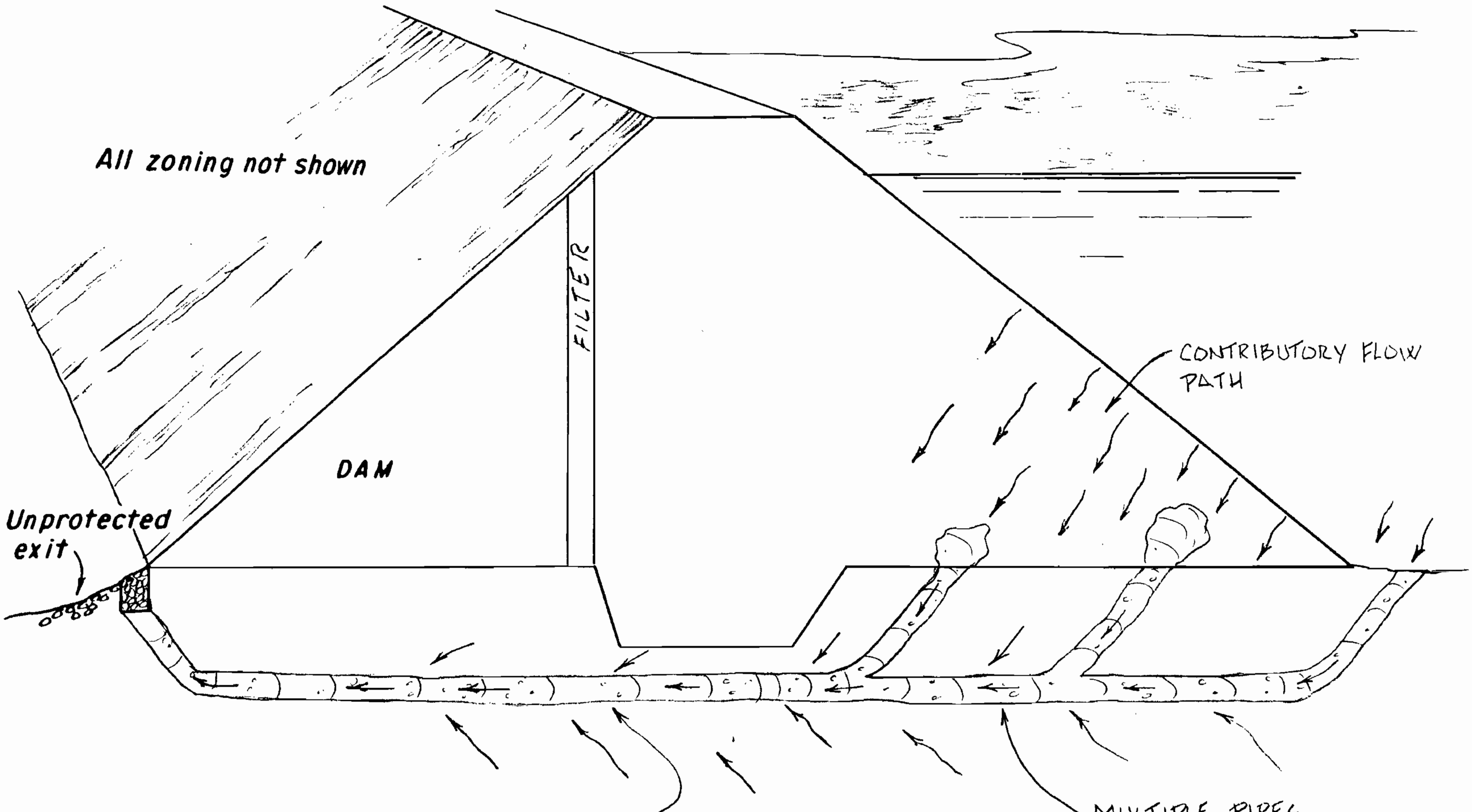
NOTE: As piping progresses upstream the hydraulic gradient increases, as the hydraulic gradient increases flow volumes and velocities increase upstream piping potential.

GENERAL PIPING MECHANISMS
MECHANISM — 2
FIGURE F — 2



NOTE: As piping progresses upstream the hydraulic gradient increases, as the hydraulic gradient increases flow volumes and velocities increase upstream piping potential.

GENERAL PIPING MECHANISMS
MECHANISM — 3
FIGURE F — 3

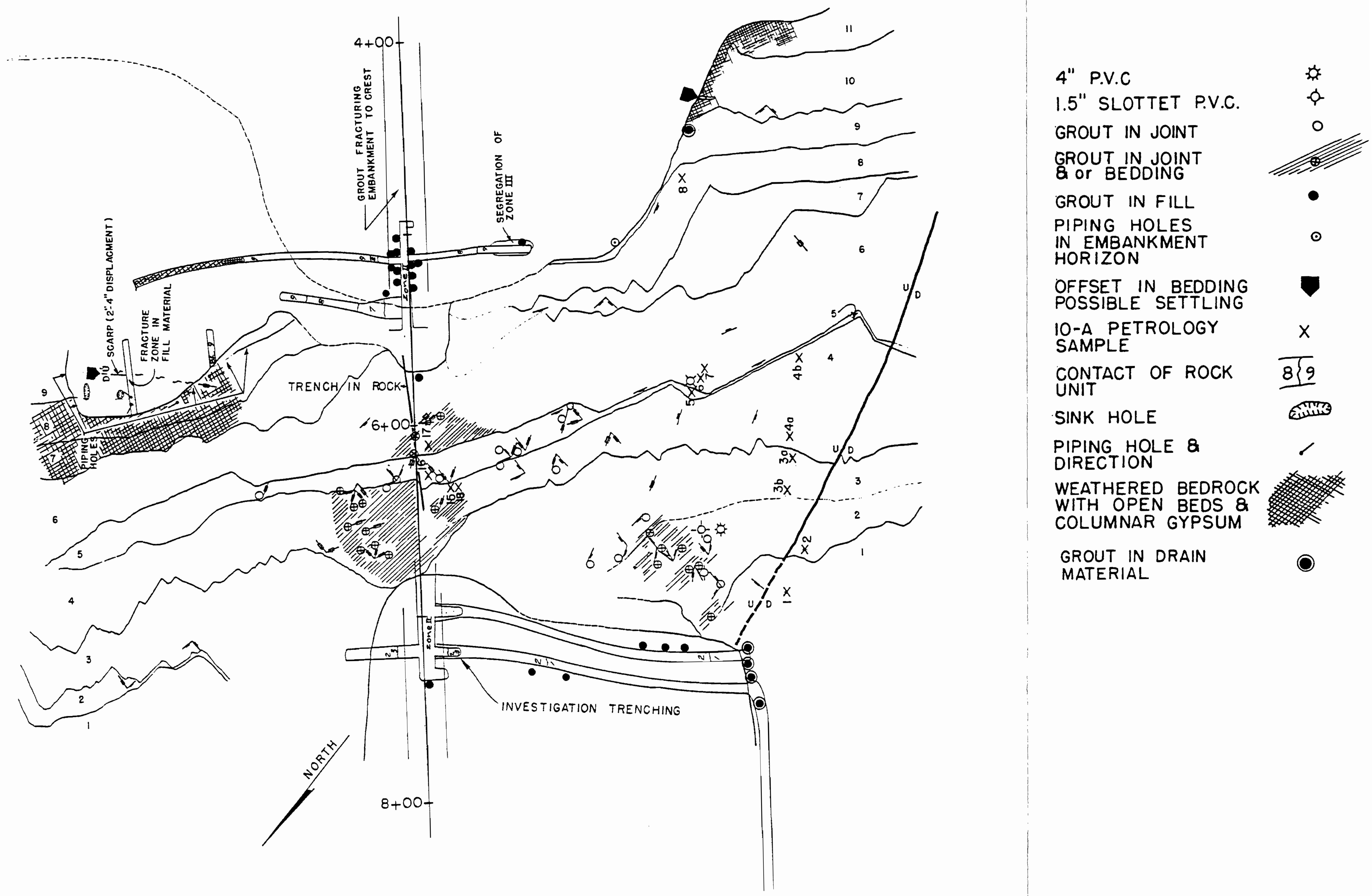


NOTE: As piping progresses upstream the hydraulic gradient increases, as the hydraulic gradient increases flow volumes and velocities increase upstream piping potential.

GENERAL PIPING MECHANISMS
MECHANISM — 4
FIGURE F — 4

APPENDIX G

Map of Significant Observations after Dike Failure


4+00
GROUT FRACTURING EMBANKMENT TO CREST
SEGREGATION OF ZONE III
SCARP (2"-4" DISPLACMENT)
FRACTURE ZONE IN FILL MATERIAL
PIPING HOLES
TRENCH IN ROCK
6+00
INVESTIGATION TRENCHING
NORTH
8+00
4" P.V.C
1.5" SLOTTET P.V.C.
GROUT IN JOINT
GROUT IN JOINT & or BEDDING
GROUT IN FILL
PIPING HOLES IN EMBANKMENT HORIZON
OFFSET IN BEDDING POSSIBLE SETTLING
10-A PETROLOGY SAMPLE
CONTACT OF ROCK UNIT
SINK HOLE
PIPING HOLE & DIRECTION
WEATHERED BEDROCK WITH OPEN BEDS & COLUMNAR GYPSUM
GROUT IN DRAIN MATERIAL