GR-86-3

ANALYSIS OF UTILIZATION OF GROUT AND GROUT CURTAINS — KORTES DAM

February 1986
Engineering and Research Center

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16. ABSTRACT

The foundation grouting program at Kortes Dam was one of six large Bureau of Reclamation dams which was reviewed and analyzed. The purpose of this program was to analyze the use of foundation grouting in Bureau structures to determine the effectiveness of the grout over the service life of the dams to date. Special attention was given to preconstruction geological conditions and changed or unexpected geological conditions discovered during the grouting activities.

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ANALYSIS OF UTILIZATION OF GROUT AND GROUT CURTAINS– KORTES DAM

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UNITED STATES DEPARTMENT OF THE INTERIOR ★ BUREAU OF RECLAMATION

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ANALYSIS OF UTILIZATION OF GROUT AND GROUT CURTAINS

KORTES DAM

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ANALYSIS OF UTILIZATION OF GROUT AND GROUT CURTAINS

KORTES DAM

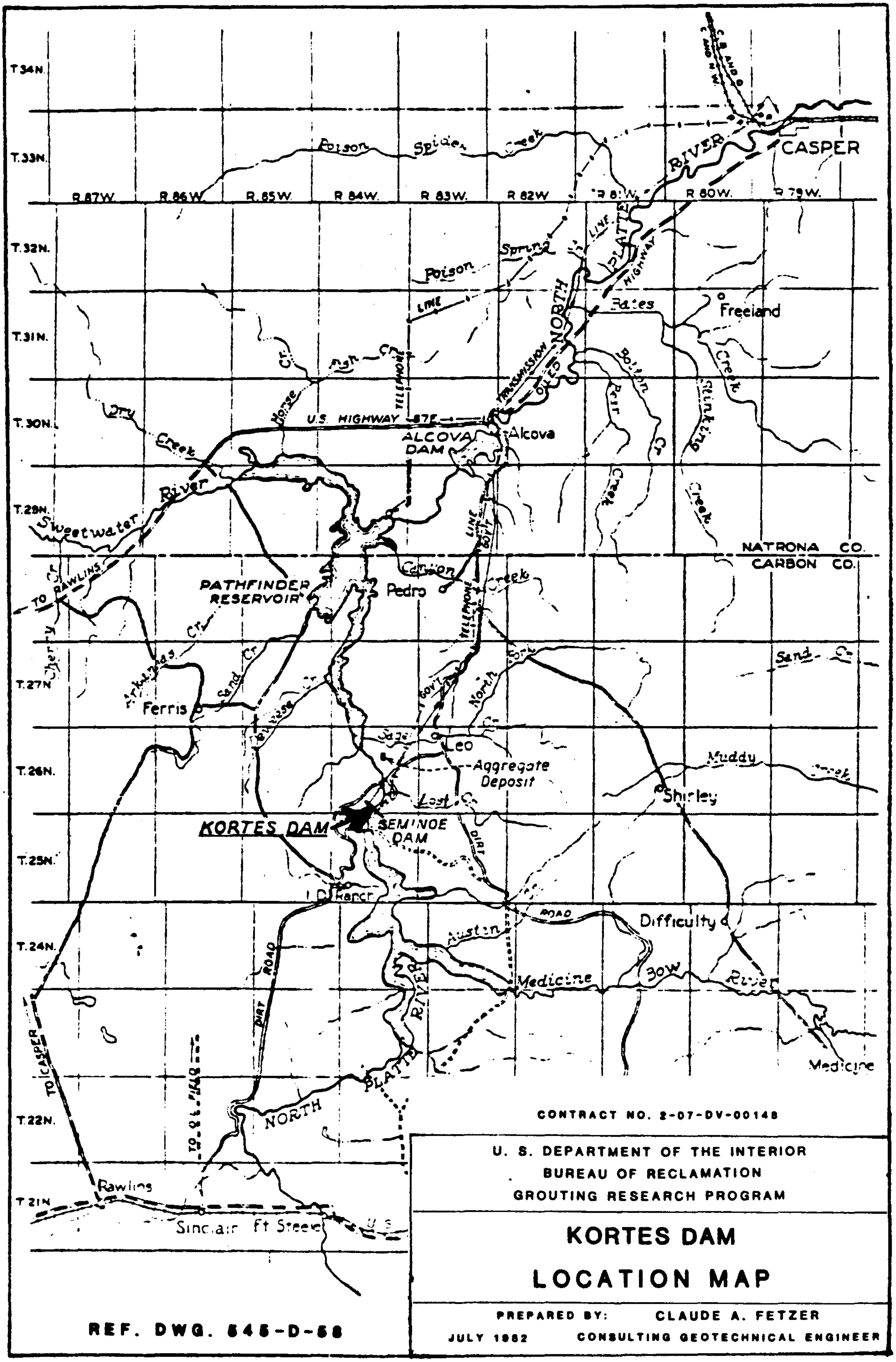
I. INTRODUCTION

1. General description. Kortes Dam is located in Black Canyon on the North Platte River in Southeastern Wyoming. Kortes Dam is located between Seminoe Dam and Pathfinder Reservoir, and its main function is to generate power from Seminoe Dam releases. It is located approximately 60 miles southeast of Casper, Wyoming. See figure 1.

The dam is a concrete-gravity structure, and it has a maximum structural height of 244 feet above the lowest point in the foundation. The crest is at El. 6,169 feet, and the crest length is 440 feet. The maximum base width of the dam is 193 feet and the crest width is 24 feet. See figure 2.

There are no river-level outlets, and normal releases are made through the power plant located at the downstream toe of the dam. Water is transmitted from the reservoir to the powerplant through three 108-inch-diameter steel penstocks sloping through the center of the dam.

The spillway is an uncontrolled overflow type discharging through a tunnel in the right abutment. The tunnel discharges into the tailrace channel about 400 feet downstream from the power plant. The spillway crest is at El. 6,142 feet; and although the reservoir water surface is normally held near El. 6,140, spillage through the spillway is very infrequent.



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KORTES DAM
LOCATION MAP

PREPARED BY: CLAUDE A. FETZER
JULY 1982 CONSULTING GEOTECHNICAL ENGINEER

FIGURE 1

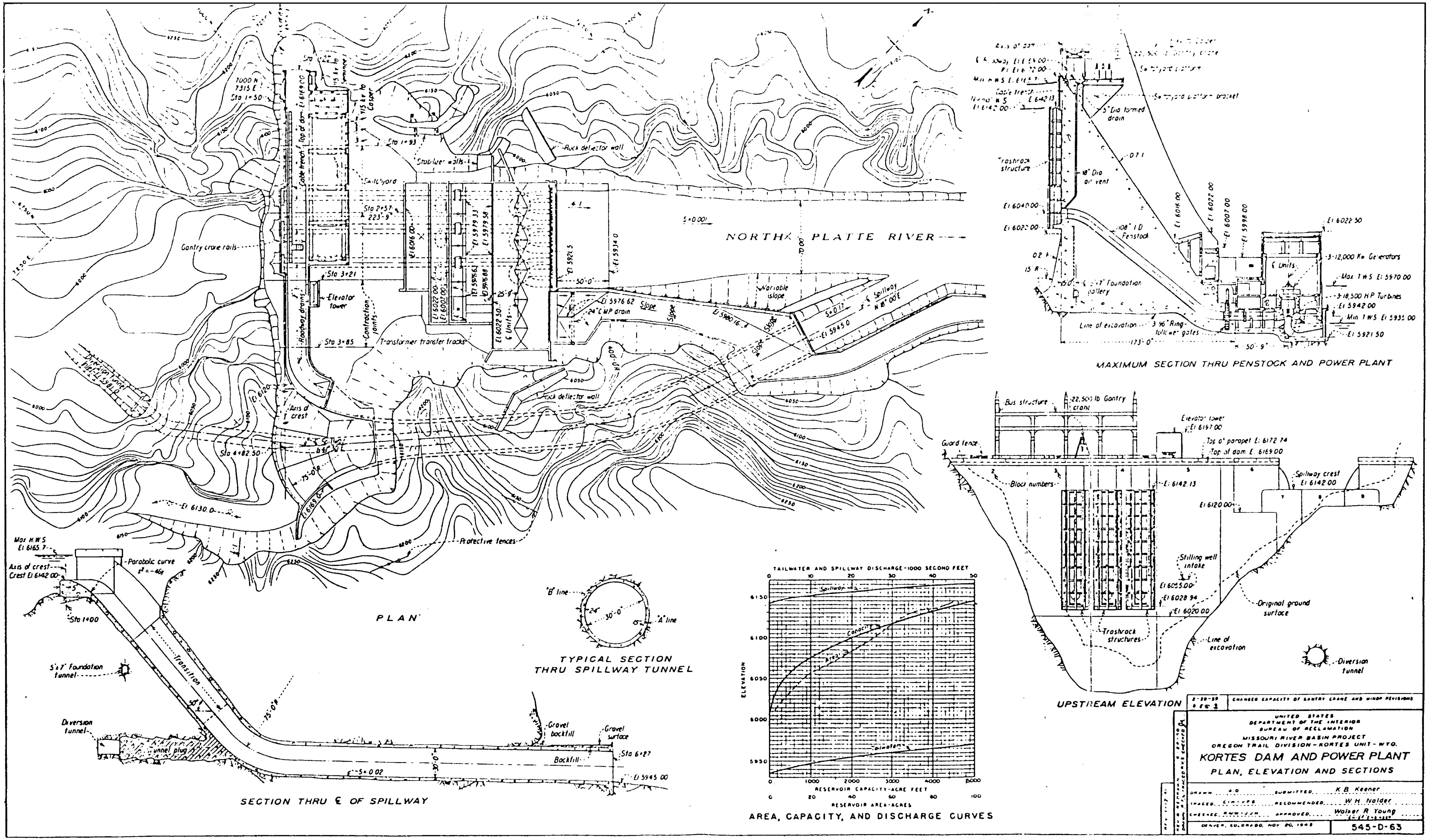

NORTH PLATTE RIVER
PLAN
Gantry crane rails
Elevator tower
Contraction joints
Transformer transfer tracks
Rock deflector wall
Stabilizer walls
Switchyard
Protective fences
Axis of crest
6 Units
TYPICAL SECTION THRU SPILLWAY TUNNEL
"B" line
"A" line
SECTION THRU ℄ OF SPILLWAY
Max H.W.S El 6165.7
Axis of crest
Crest El 6142.00
Parabolic curve
Sta 1+00
5'x7' Foundation tunnel
Diversion tunnel
Transition
Gravel backfill
Gravel surface
Backfill
Sta 6+27
El 5945.00
MAXIMUM SECTION THRU PENSTOCK AND POWER PLANT
Trashrack structure
18" Dia air vent
108" I.D. Penstock
3-12,000 Kw Generators
3-18,500 H.P. Turbines
Max T.W.S. El 5970.00
Min T.W.S. El 5935.00
Line of excavation
UPSTREAM ELEVATION
Bus structure
22,500 lb Gantry crane
Elevator tower
Guard fence
Top of parapet El 6172.74
Top of dam El 6169.00
Spillway crest El 6142.00
Block numbers
Stilling well intake
Original ground surface
Trashrack structures
Line of excavation
Diversion tunnel
AREA, CAPACITY, AND DISCHARGE CURVES
TAILWATER AND SPILLWAY DISCHARGE-1000 SECOND FEET
RESERVOIR CAPACITY-ACRE FEET
RESERVOIR AREA-ACRES
ELEVATION
Spillway
Capacity
Area
Tailwater
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MISSOURI RIVER BASIN PROJECT
OREGON TRAIL DIVISION—KORTES UNIT—WYO.
KORTES DAM AND POWER PLANT
PLAN, ELEVATION AND SECTIONS
SUBMITTED K.B. Keener
RECOMMENDED W.H. Nalder
APPROVED Walker R. Young
DENVER, COLORADO, NOV. 20, 1948
545-D-63

FIGURE 2

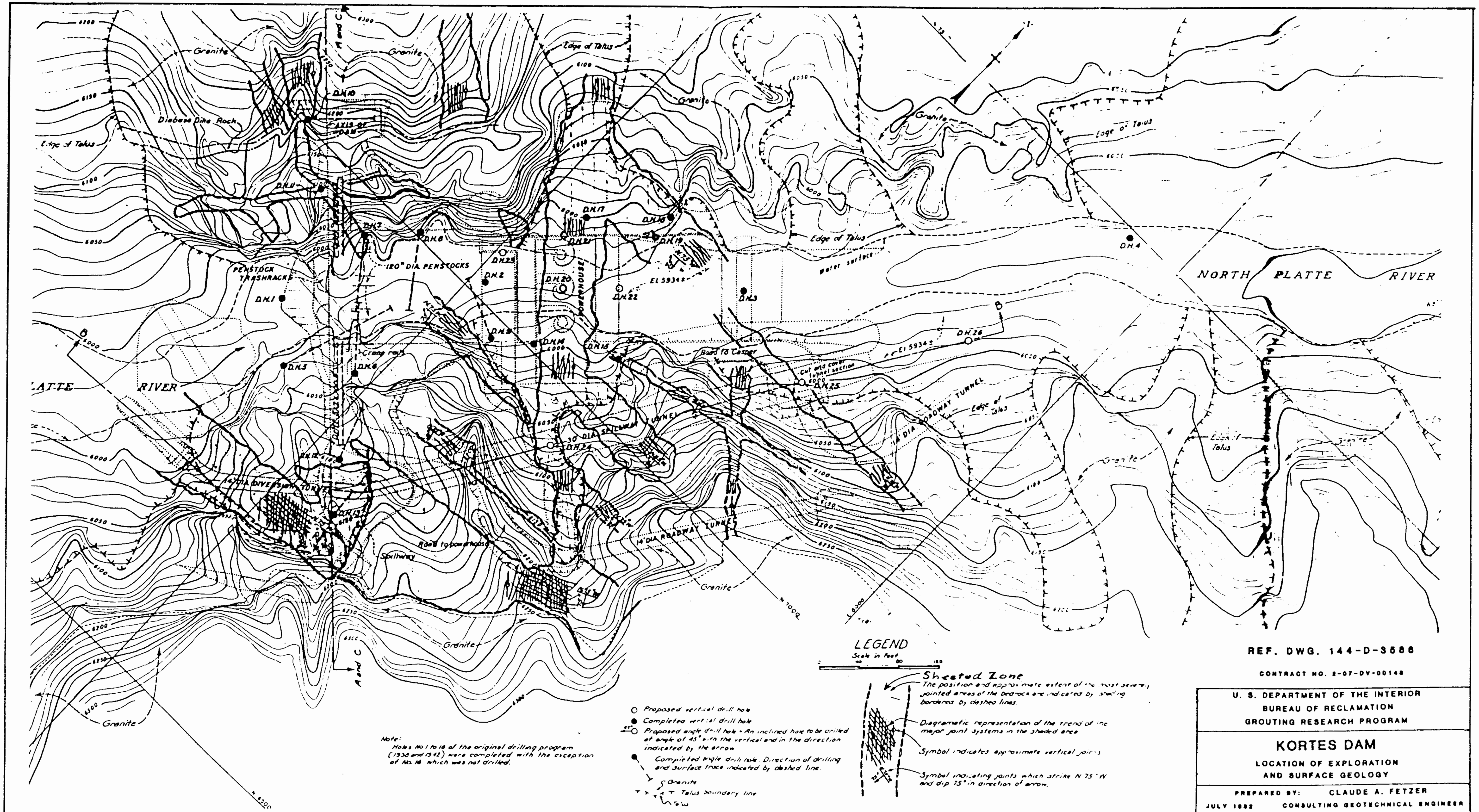
The maximum differential water head on the dam occurs with water at spillway crest and with the tailwater at the minimum, El. 5,935 -- a differential of 207 feet.

Construction of the dam was started in May 1946 and the work was completed in January 1951. The work site was protected by earthen cofferdams, and the river flows were diverted through an unlined tunnel through the right abutment. The downstream portion of the diversion tunnel was later used as part of the spillway tunnel, and the upstream portion was plugged.

II. GEOLOGY

2. Regional geology. Kortes Dam and Reservoir are located on the North Platte River in the Seminoe Mountains. The Seminoe Mountains are situated on the southern limb of the Sweetwater Arch, a broad east-west trending uplift in central Wyoming. The core or "backbone" of the Seminoe Mountains is composed primarily of Precambrian metamorphic schist and gneiss intruded by igneous granitic rocks and later by dolerite, pegmatite and aplite dikes. The rocks underwent several periods of uplift and deformation which resulted in strongly developed joint patterns. No recent faulting or movement since late Cennozoic has been reported within the Seminoe Mountains. During the early Tertiary, the region was covered by sedimentary strata and later eroded during successive periods of uplift. The North Platte River incised its present course through Black Canyon, during the most recent uplift. Kortes Dam was constructed in a deep narrow gorge of Black Canyon.

3. Site geology. (a) Site investigations. Surface reconnaisance and geologic mapping revealed the rock at the damsite was severely jointed and fractured, more so, than the rock at any of the other dams investigated in this grouting research study. In several locations, jointing was very closely spaced and such areas were identified and mapped as "sheeted-zones" and are shown on figure 3, by symbol. Surface mapping and aerial photography identified talus deposits on the abutments that concealed rock outcrops in several locations. The talus deposits are also shown on figure 3, by symbol. The field studies indicated there was


NORTH PLATTE RIVER
PLATTE RIVER
PENSTOCK TRASHRACKS
120" DIA PENSTOCKS
POWERHOUSE
14' DIA DIVERSION TUNNEL
14' DIA ROADWAY TUNNEL
Diabase Dike Rock
Edge of Talus
Granite
Axis of Dam
Road to Casper
Road to powerhouse
Spillway
Crane rail
Water surface
LEGEND
Scale in Feet
Sheeted Zone
The position and approximate extent of the most severely jointed areas of the bedrock are indicated by shading bordered by dashed lines
Diagrammatic representation of the trend of the major joint systems in the shaded area
Symbol indicates approximate vertical joints
Symbol indicating joints which strike N 75° W and dip 75° in direction of arrow.
Proposed vertical drill hole
Completed vertical drill hole
Proposed angle drill hole • An inclined hole to be drilled at angle of 45° with the vertical and in the direction indicated by the arrow
Completed angle drill hole. Direction of drilling and surface trace indicated by dashed line.
Granite
Talus boundary line
Talus
Note:
Holes No 1 to 16 of the original drilling program (1938 and 1942) were completed with the exception of No. 16 which was not drilled.
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KORTES DAM
LOCATION OF EXPLORATION AND SURFACE GEOLOGY
PREPARED BY: CLAUDE A. FETZER
JULY 1982 CONSULTING GEOTECHNICAL ENGINEER

FIGURE 3

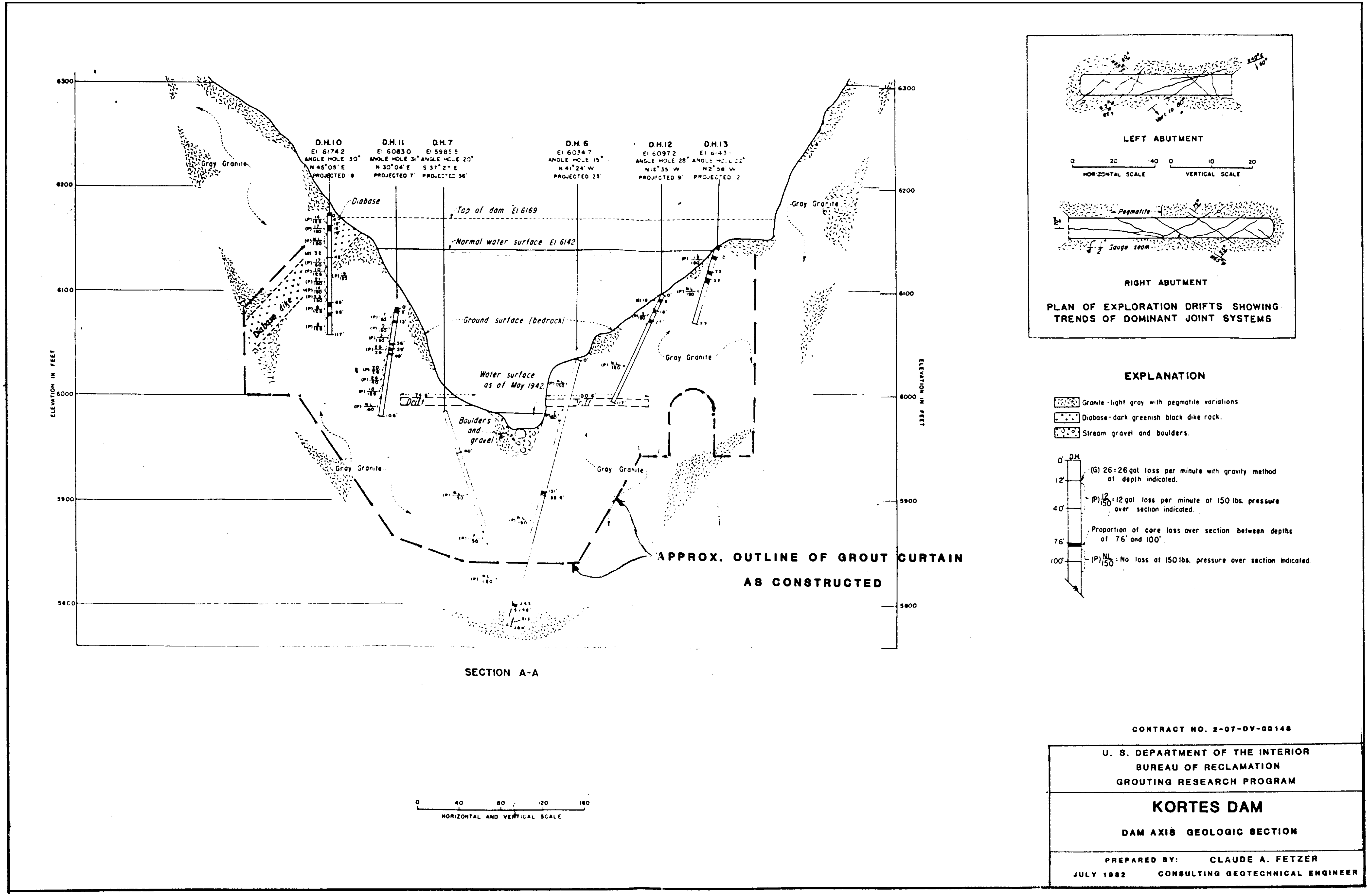
very little overburden and that there were practically no zones of weathered or decomposed rock.

Geologic mapping revealed no evidence of major faulting at the damsite.

(b) Exploratory borings and drifts. The first test drilling at Kortes Dam site, to determine depth of alluvium in the river bottom, was completed in 1938. At about the same time, two 6-foot by 7-1/2-foot exploratory drifts were driven 100 feet and 75 feet into the right and left abutments near the proposed dam axis at an invert elevation just above river bed as shown on figures 3 and 4. A plan of the two exploratory drifts showing the trends of dominant joint systems is presented on figure 4, and a geologic section along the dam axis is also shown on figure 4.

Twenty-nine borings, totaling 2,776 linear feet of drilling, were completed at the site by 1947 and are shown on figure 3. An examination of the logs of these test borings shows they contain meager descriptive information. For example, these logs give no information on the joint orientation or spacing, or on the amount or type of staining which provides clues on ground-water circulation and on the degree of openness of the joints. Figure 5 is a photograph of the core from hole DH-10. The majority of the breaks in the core represent pre-existing joints. The "button-like" pieces are probably from one of the "sheeted-zones" where the joints are closely spaced. Core recovery was good considering the severely jointed and fractured condition of the rock.

The water pressure tests conducted in the exploratory borings indicate that in most cases the holes were tested using



PLAN OF EXPLORATION DRIFTS SHOWING TRENDS OF DOMINANT JOINT SYSTEMS
LEFT ABUTMENT
RIGHT ABUTMENT
HORIZONTAL SCALE
VERTICAL SCALE
Pegmatite
Gouge seam
EXPLANATION
Granite - light gray with pegmatite variations.
Diabase - dark greenish black dike rock.
Stream gravel and boulders.
(G) 26 = 26 gal loss per minute with gravity method at depth indicated.
(P) 12/150 = 12 gal loss per minute at 150 lbs. pressure over section indicated.
Proportion of core loss over section between depths of 76' and 100'.
(P) NL/150 = No loss at 150 lbs. pressure over section indicated.
D.H.10
El 6174.2
ANGLE HOLE 30°
N 45°05' E
PROJECTED 18'
D.H.11
El 6083.0
ANGLE HOLE 31°
N 30°04' E
PROJECTED 7'
D.H.7
El 5985.5
ANGLE HOLE 20°
S 37°27' E
PROJECTED 36'
D.H.6
El 6034.7
ANGLE HOLE 15°
N 41°24' W
PROJECTED 25'
D.H.12
El 6097.2
ANGLE HOLE 28°
N 12°35' W
PROJECTED 9'
D.H.13
El 6143
ANGLE HOLE 22°
N 2°38' W
PROJECTED 2'
Top of dam El 6169
Normal water surface El 6142
Diabase
Diabase dike
Gray Granite
Ground surface (bedrock)
Water surface as of May 1942
Boulders and gravel
Drift
Dr. II
APPROX. OUTLINE OF GROUT CURTAIN AS CONSTRUCTED
ELEVATION IN FEET
SECTION A-A
HORIZONTAL AND VERTICAL SCALE
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KORTES DAM
DAM AXIS GEOLOGIC SECTION
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JULY 1982
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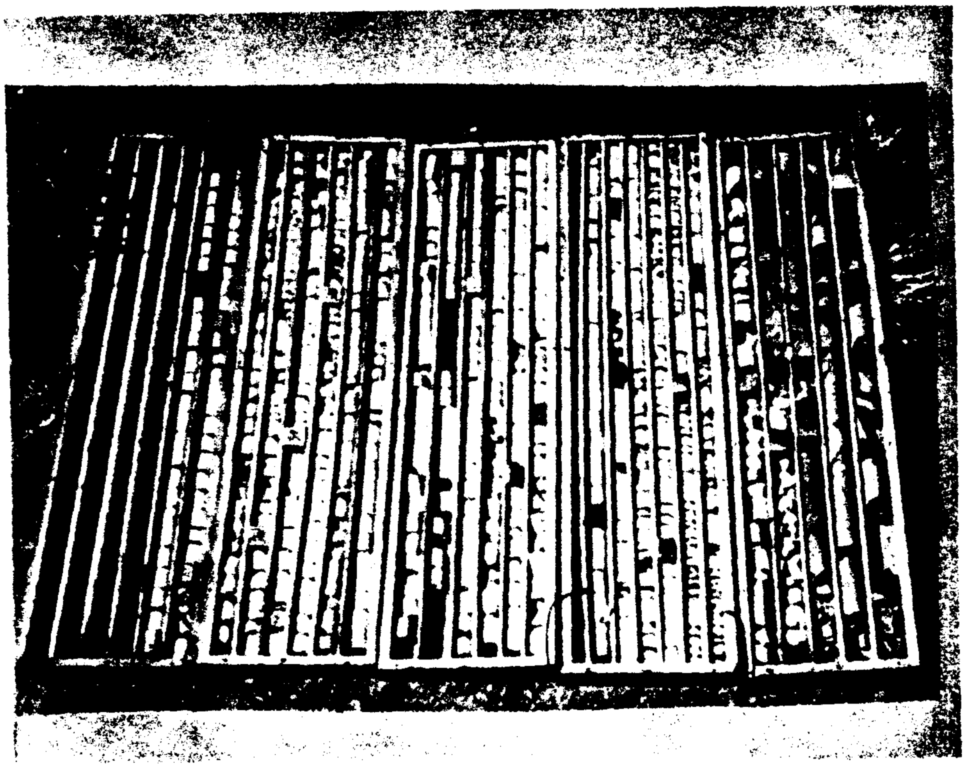
FIGURE 4



Kortes Dam

This photograph was taken of the core recovered from diamond-drill hole No. 10. The upper right-hand corner of the right hand core box is the top of the hole; the depth increases progressively towards the left. The dark gray core at the top of the hole represents the diabase dike shown on the geologic map. The ligher-colored core is granite.

The majority of the breaks in the core represent preexisting joints and are, as the photograph illusteates, more or less evenly spaced throughout the core. This is the basis for the conclusion that the entire rock mass of the abutments and foundation of the Kortes Dam site will be almost as severely fractured at moderate depth as it is on the exposed rock surface. The "button like" pieces shown at "A" (as compared to the pieces 2 to 4 inches in length "B") represent a zone of abnormally close-spaced parallel joint planes. Such zone is referred to in the text as a "sheeted" structure or "sheeted zone".

From Figure 1 Geology Report No. G-63

Figure 5

150 psi pressure at packer settings as shallow as 10 feet. Some exceptions to the use of 150 psi pressure appeared to be where the packer did not hold or where apparent pump capacity of ±25 gpm was reached. Overall the rock was very tight, particularly below 30 to 40 feet. Exceptions to this may have occurred where the hole was located near an exposed rock face allowing water to escape laterally.

(c) Foundation rock. The foundation rock at Kortes Dam consists chiefly of a light-colored, medium grained granite, with variations ranging from coarse-grained pegmatite to well developed gneissic zones. Diabase and dolerite dikes have intruded the granite at several locations in the canyon. A thick dark-colored dike crossed the dam axis near crest elevation in the left abutment area. The dike dipped about 54° NW with a strike approximately parallel to the course of the river, or N 45° E. The dike was a tough, dense, basaltic rock referred to as dolerite or diabase.

The river alluvium, consisting mainly of sand and gravel with some very large boulders, had a maximum thickness of about 25 feet above bedrock.

(d) Rock structure. As stated previously, the principal rock defects at Kortes were complex systems of joints and fractures, particularly the sheeted-zones where the joints were spaced 1/4" to 1" apart and contained altered clay minerals. During the preconstruction stage of the project the fracturing was recognized as the most serious geologic defect at the site, but its actual effects were considerably underestimated. This was due to the particular combination of fracture patterns, one paralleling the basalt dike and the other two roughly perpendicular to the dike and to each other. During the initial excavation this left many poorly supported V-shaped blocks in the left abutment that required extensive high scaling and the construction of selectively placed deflector and stabilizer walls. Photographic mosiacs of both abutments before construction are shown on figures 6 and 7, and a photograph of the powerhouse area is shown on figure 8.

The areas of most severely fractured bedrock coincided with those areas in which the close-spaced joints of both major joint systems were strongly developed. Although both abutments were effected by the complex fracturing, the pattern favored the right abutment by forming rock blocks that were held into the abutment by gravity; whereas, fracturing on the left abutment created unsupported blocks that tended to slide into the canyon. This condition was substantiated by the exploratory borings which showed higher core recovery and lower drill water losses and hence tighter joints on the right abutment than did the borings on the left abutment where the rock mass had been loosened by the adversely oriented joints and fractures. Similarly, the bedrock exposed in the exploratory drifts was of better quality in the right abutment than on the left.

As stated, faulting was not a significant geologic condition at the site. While geologic studies indicated some evidence of slippage along joint planes and the presence of minor shear zones, the explorations and excavations during construction

Figure 6

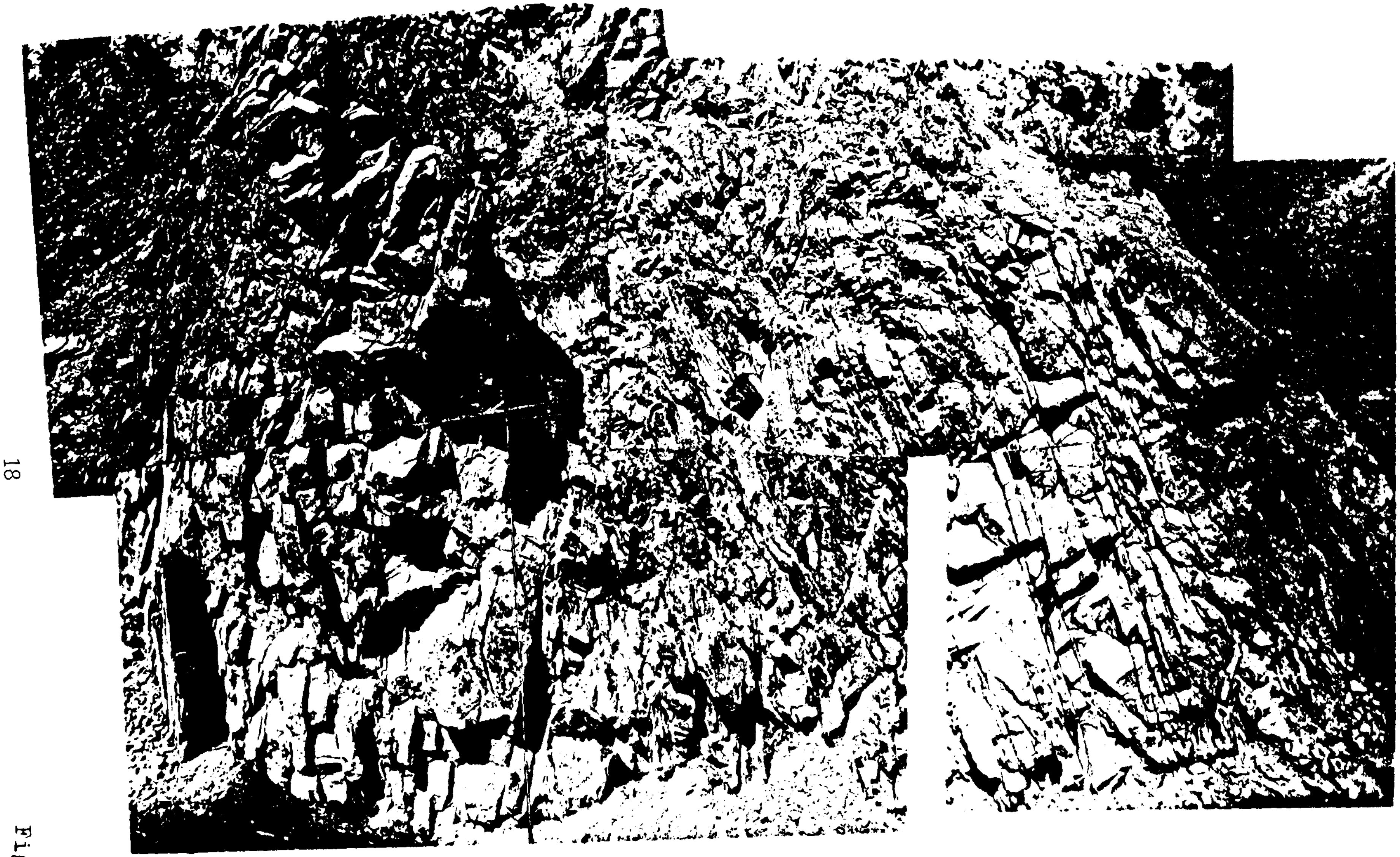


Photo mosaic of left abutment area of Kortes Dam before the start of excavation or scaling.

This photograph of the canyon wall on the east side of the river at the powerplant site illustrates the rough topography on that side of the river and the steep, precipitous slopes.

Figure 7

Figure 8 This photograph was taken to illustrate the general relations between bedrock exposures and talus or slide rock in the area of the powerhouse foundation. The approximate position of the prominent sheeted zone which crosses the river through the foundation area of the powerhouse is shown by a "dashed" line.

did not reveal any large zones of gouge or areas of extensively sheared or crushed rock.

III. FOUNDATION EXCAVATION AND TREATMENT

4. Excavation procedures. Before excavation of the dam and powerhouse foundations began, the canyon walls above the normal excavation grade lines were stripped of loose rock. This material was removed by barring, wedging and blasting. Approximately 20,469 cubic yards of rock were removed from the canyon walls in this manner. The excavation limits for the dam were estimated from a study made of drill holes, cross-sections, and a close inspection of the surface rock. The drilling operations were started on cut lines thus established. Drilling was performed by jack-hammers operated by men suspended from the abutments on ropes. Detachable drill bits were used to minimize freighting on the cliffs. The cableway was used to hoist men and materials on the abutments. The cuts were carried from the top down in a series of shots that put the material in the bottom of the canyon. The abutments were scaled after each shot; and if shots removed support from detached masses, the operation was moved back and loose unstable rock was removed. Repeated cross-sections were taken to be sure the final section would meet design requirements. Excavated material was picked up in the bottom of the canyon with a 54-B shovel.

Rock on the right abutment proved competent but considerable quantity was removed to provide a satisfactory foundation. On the left abutment, the more highly fractured and weathered rock required removal of approximately 50,000 cubic yards. Excavation in the bottom of the channel consisted primarily of cutting to prescribed grade and was not dependent upon rock conditions. A total of 166,566 cubic yards of rock was excavated for the dam, powerhouse, spillway crest and inlet.

Figure 9 shows the nearly completed excavation of the left abutment. Figure 10 shows the condition of the rock on the right abutment above the powerhouse located about 150 feet downstream from the dam. Figure 11 shows two pictures of the cleanup of the foundation at the base of the dam. Comparing figures 9 through 11 with figures 6 through 8, which show rock conditions prior to scaling or excavation, it appears that the joints in the completed excavation are relatively tight and that excavation and cleanup operations produced a sound rock surface for the gravity dam.

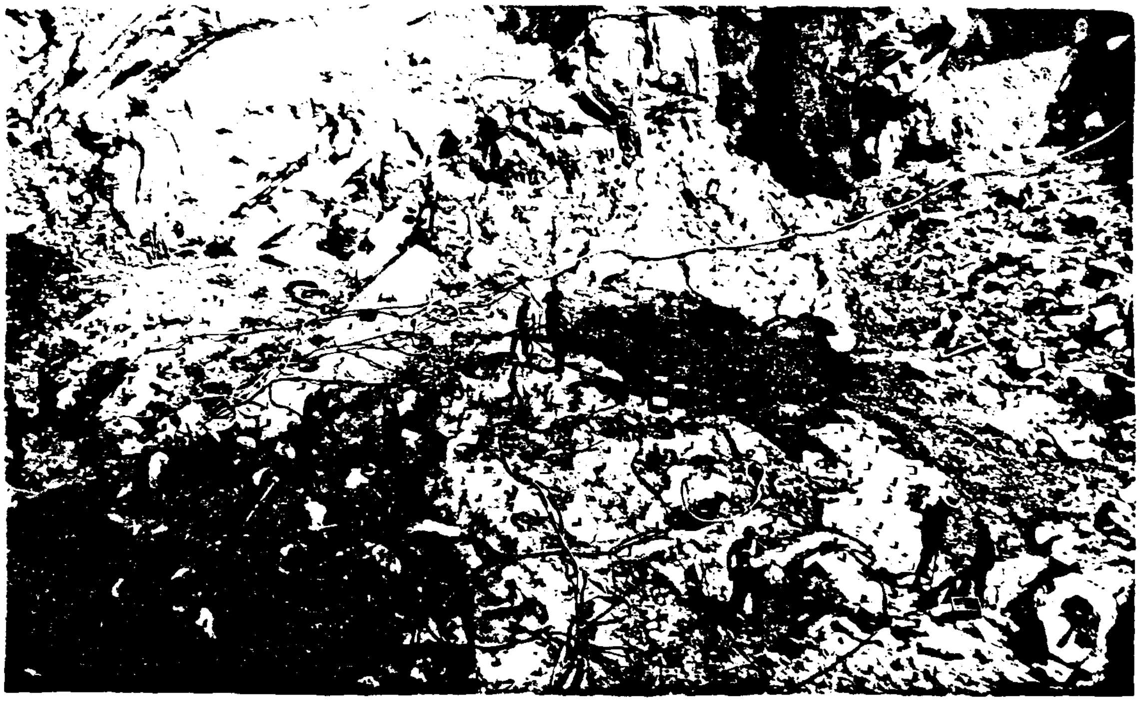
5. Foundation treatment. Excavation for the abutments was complicated by the extensive jointing encountered in the abutments. The excavation encountered intersecting joints that resulted in rock falls. One large rock fall occurred during construction and destroyed part of the contractor's aggregate plant. Many poorly supported V-shaped blocks of rock were left after the initial excavation. Extensive rock bolting was used within and outside the foundation area to hold the rock blocks. Also extensive additional scaling was accomplished to prevent damage to the dam and powerplant from falling rock after construction. Large masses of talus that had accumulated in draws adjacent to and downstream from the powerplant were removed by hydraulic monitors. These potentially unstable masses could have slid into the tailrace from actions of the fluctuating tailwater.

405-286 Nearly completed excavation of left abutment. Axis just to the left of foundation tunnels. Top of dam at the black seam at the top of the photo.

Figure 9

405-268 Excavation. Right side of powerhouse. Vertical cuts approximately 40'. Form work is for concrete protective wall to deflect falling rock and talus material away from power plant.

Figure 10



405-306 Final excavation and clean up operations of the base of dam.

405-354 Foundation rock at the base of dam near the axis. Foundation grout pipe, drain pipe and uplift pipe installed. Cooling coils on rock at $2\frac{1}{2}$ foot centers.

Figure 11

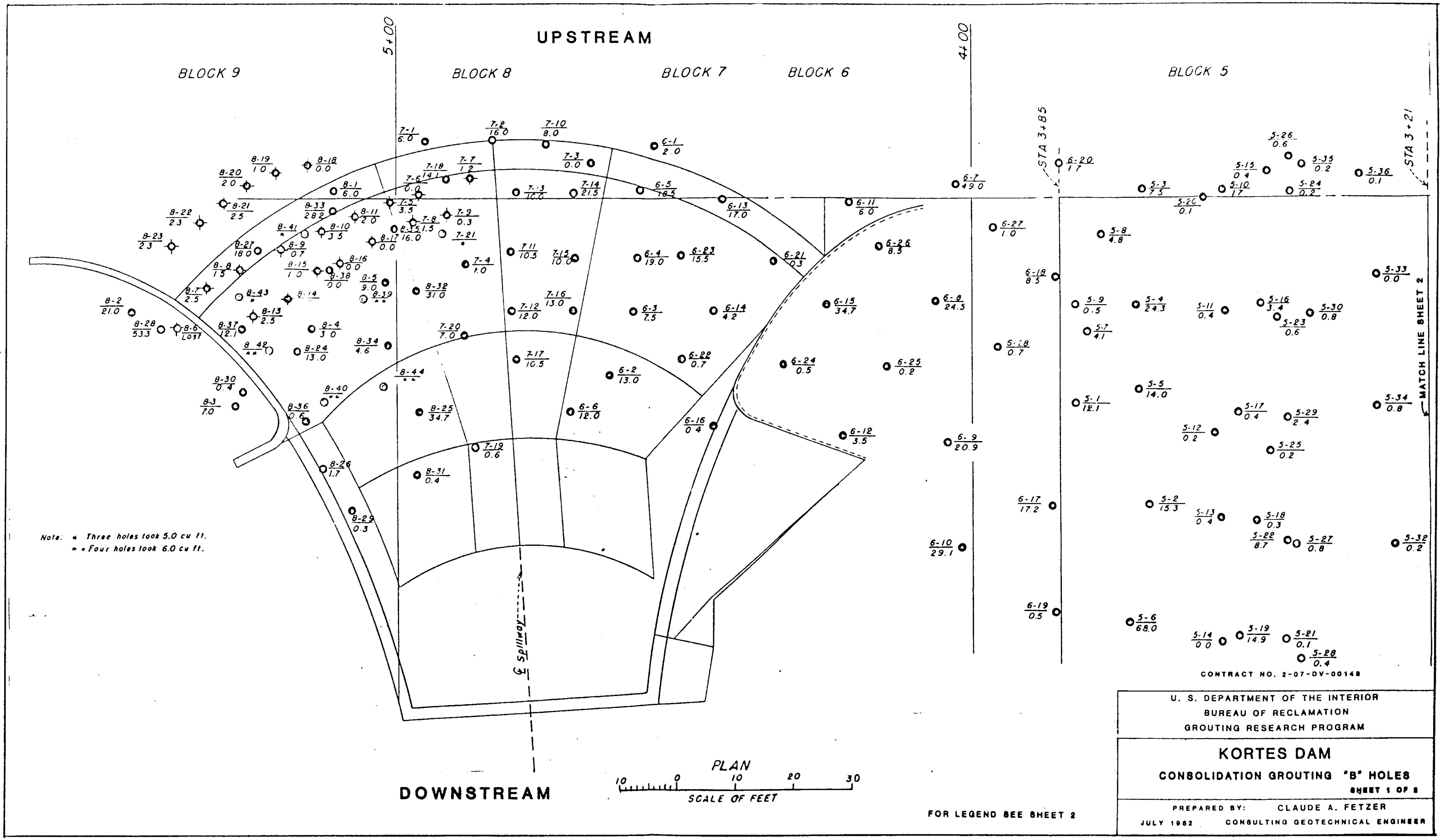
IV. GROUTING AND DRAINAGE

6. General. Chapter III, "Grouting and Drainage" of the Technical Record of Design and Construction, has been reproduced as appendix A of this report and only points of clarification will be presented herein.

7. Design. The general plan for grouting the foundation rock under the dam provided for the following:

(1) Consolidation grouting of shallow, low pressure grout holes along four lines spaced approximately 20 feet apart and on 20-foot centers under the upstream portion of the dam. The holes were drilled from the foundation surface to depths of approximately 30 feet. The number, location, angle and direction of the "B" holes were determined in the field by the inspectors. The layout and takes of the B-holes are shown on figures 12 and 13. The holes were drilled in a vertical direction in the valley bottom and normal to the rock surface on the abutments except that several holes were angled in various directions to intersect surface joints.

(2) The "A" line high pressure cutoff was constructed from the foundation gallery and exploratory tunnels. Ascending-stage grouting was used with a packer set at the top of each stage. The holes were split spaced and grouted by zones. By design, closure was to 5-foot centers. The holes were inclined upstream 8 degrees from the vertical and extended to a depth of 100 feet below the concrete-rock contact. All holes were drilled vertically except for the upstream inclination and except holes inclined 30 degrees from the horizontal in the upper part of the


UPSTREAM
DOWNSTREAM
BLOCK 9
BLOCK 8
BLOCK 7
BLOCK 6
BLOCK 5
5+00
4+00
STA 3+85
STA 3+21
MATCH LINE SHEET 2
℄ Spillway
Note: * Three holes took 5.0 cu ft.
* * Four holes took 6.0 cu ft.
PLAN
SCALE OF FEET
FOR LEGEND SEE SHEET 2
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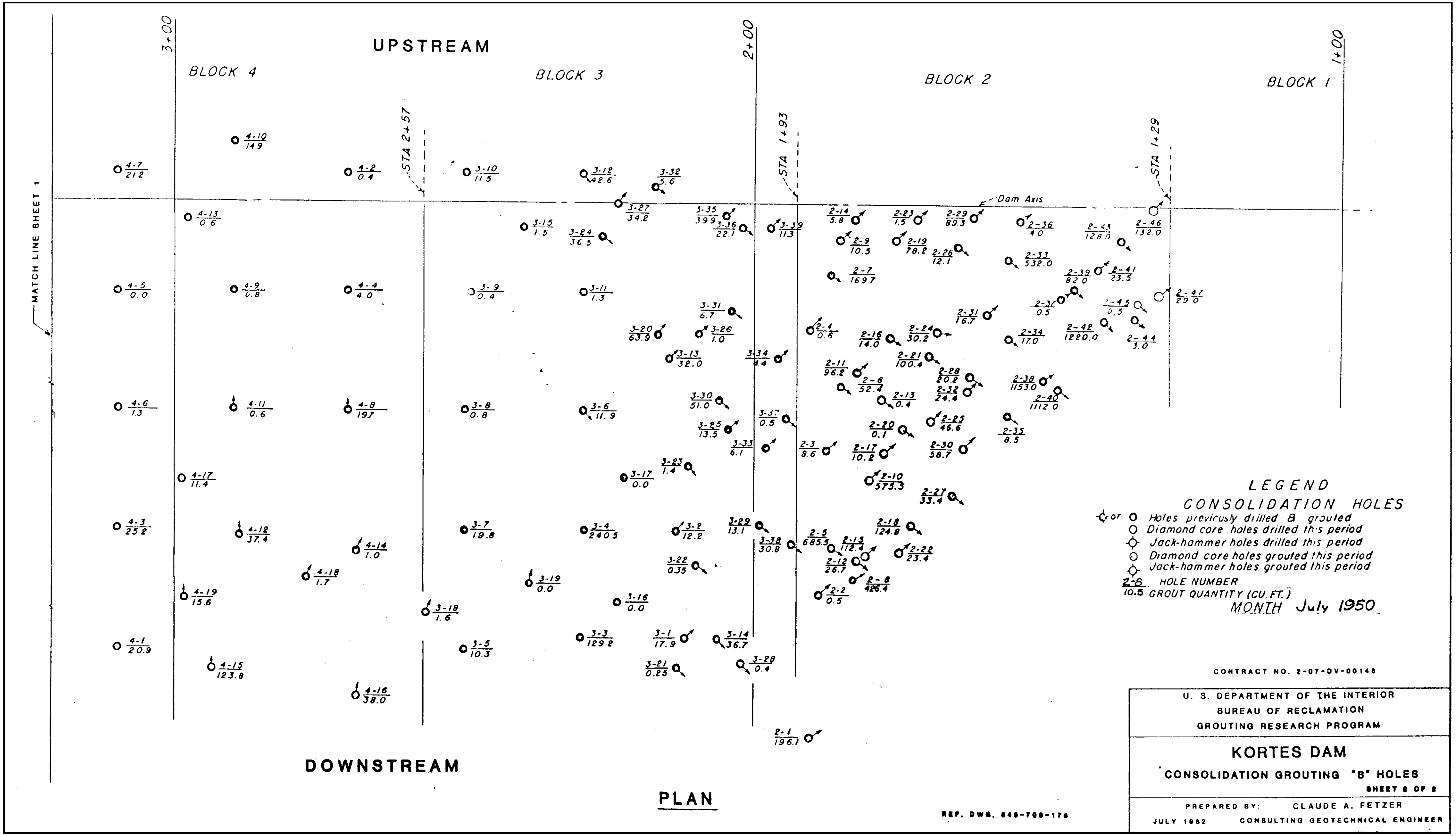
GROUTING RESEARCH PROGRAM
KORTES DAM
CONSOLIDATION GROUTING "B" HOLES
SHEET 1 OF 2
PREPARED BY: CLAUDE A. FETZER
JULY 1982 CONSULTING GEOTECHNICAL ENGINEER

FIGURE 12



UPSTREAM
DOWNSTREAM
BLOCK 4
BLOCK 3
BLOCK 2
BLOCK 1
3+00
2+00
1+00
STA 2+57
STA 1+93
STA 1+29
Dam Axis
MATCH LINE SHEET 1
LEGEND
CONSOLIDATION HOLES
Holes previously drilled & grouted
Diamond core holes drilled this period
Jack-hammer holes drilled this period
Diamond core holes grouted this period
Jack-hammer holes grouted this period
2-8 HOLE NUMBER
10.5 GROUT QUANTITY (CU. FT.)
MONTH July 1950
CONTRACT NO. 2-07-DV-00148
U. S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GROUTING RESEARCH PROGRAM
KORTES DAM
CONSOLIDATION GROUTING "B" HOLES
SHEET 2 OF 2
PREPARED BY: CLAUDE A. FETZER
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PLAN
REF. DWG. 546-700-176

FIGURE 13

left abutment. Holes were also drilled at various angles into the left abutment to intersect leaks that developed when the pool was raised to El. 6,080 by releases from Seminoe Dam in June 1950. The profile and takes of the "A"-holes are shown on figure 14, which was reproduced from the September 1950 foundation treatment report. The profile also shows that grout holes were drilled upwards and downwards from the abutment foundation tunnels. See figure 8 on page numbered 17 in appendix A.

8. Specifications. The specifications were complete so far as general performance of the work was concerned and so far as can be determined from the records did not cause any construction problems. The specifications did not establish specific requirements for distance between drilling and grouting operations, a time limit for water pressure testing, minimum capacity for mixing and grouting equipment, minimum size of grout pipe through the packer and methods for final backfill of holes. The specifications did provide for drilling 5-1/2-inch-diameter core holes to determine the condition of the foundation rock and/or the effectiveness of the grouting operations.

9. Grouting methods and procedures. (a) Water pressure tests. Water pressure tests were made when drilling of a hole was completed to determine the location of porous or fractured zones and to help formulate the grouting procedure. The test was made by setting the packer as close to the surface of the hole as possible and then pumping water into the hole at 50 to 100 psi pressure. The water take of the hole was measured and the packer then moved downward and measurements of the water take was recorded at 3-foot intervals. The results of the water test were

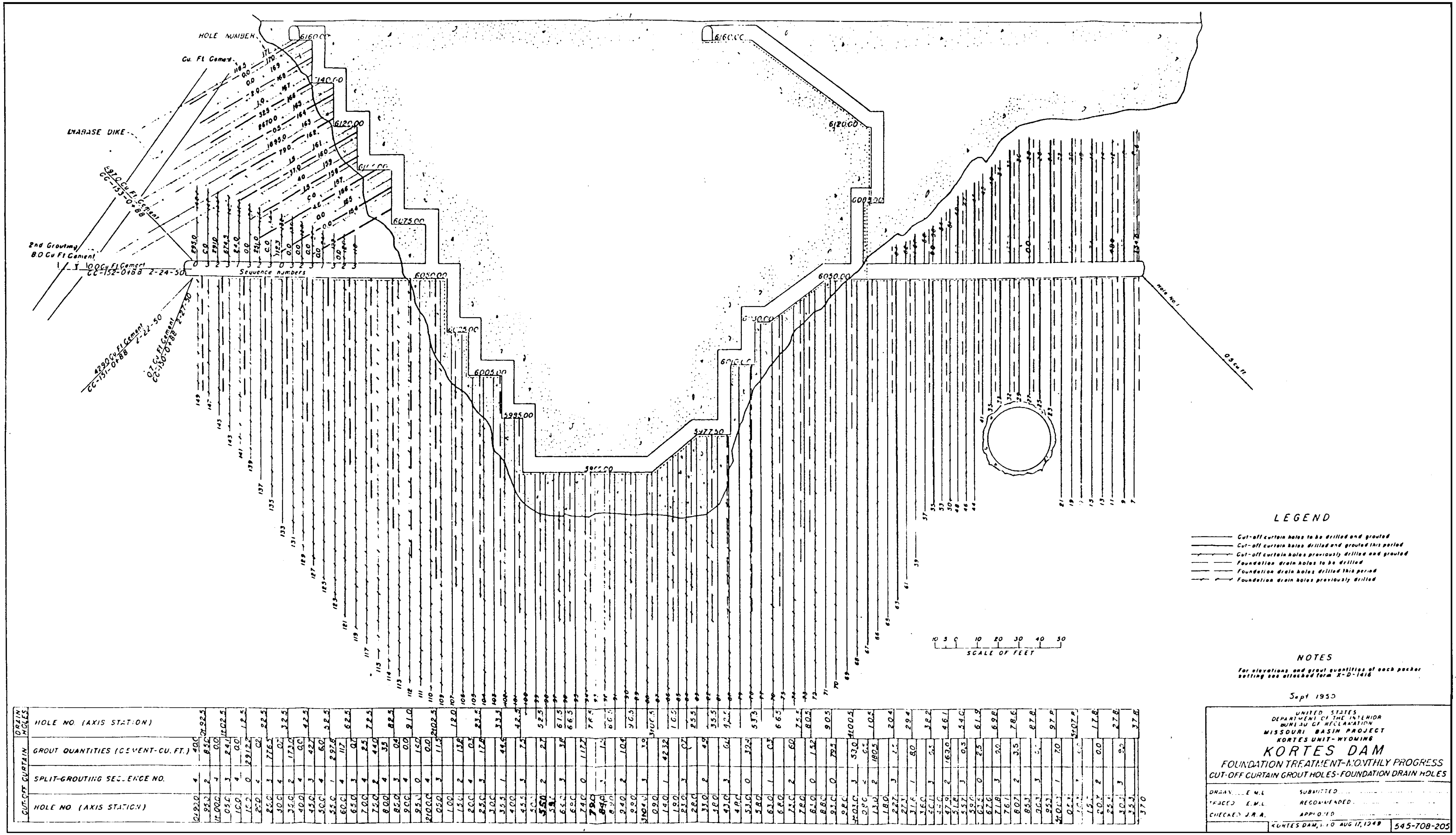

HOLE NUMBER
Cu. Ft Cement
DIABASE DIKE
2nd Grouting 8.0 Cu Ft Cement
Sequence numbers
LEGEND
Cut-off curtain holes to be drilled and grouted
Cut-off curtain holes drilled and grouted this period
Cut-off curtain holes previously drilled and grouted
Foundation drain holes to be drilled
Foundation drain holes drilled this period
Foundation drain holes previously drilled
SCALE OF FEET
NOTES
For elevations and grout quantities of each packer setting see attached form X-D-1416
Sept 1950
HOLE NO. (AXIS STATION)
GROUT QUANTITIES (CEMENT-CU. FT.)
SPLIT-GROUTING SEQUENCE NO.
HOLE NO. (AXIS STATION)
DRAIN HOLES
CUT-OFF CURTAIN
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MISSOURI BASIN PROJECT
KORTES UNIT-WYOMING
KORTES DAM
FOUNDATION TREATMENT-MONTHLY PROGRESS
CUT-OFF CURTAIN GROUT HOLES-FOUNDATION DRAIN HOLES
545-708-205

FIGURE 14

used to determine the packer settings and the starting grout mixture.

(b) Grout mixes. The water-cement ratios used in grouting were as follows:

(1) Consolidation grouting - The mixes ranged from 4:1 to 1:1 water-cement ratio by volume. The holes were grouted in three zones, with thinner grout injected under higher pressure into the lower zone.

(2) Curtain grouting - A very thin grout was used to start the holes and the mixture thickened as the grouting of the hole progressed. A water-cement ratio of 10:1 or 8:1 was generally used to start the holes; however, grouting of some holes was started and completed with mixes as thick as 1:1.

(c) Grout injection pressures. The rules used for determining the maximum pressures at the collar are presented in section 15(b) on page numbered 21 of appendix A.

(d) Spacing of holes and closure. Drilling and grouting of the deep curtain were done by the split-spacing method. The first series of holes were drilled and grouted on 80-foot centers. Thereafter, the secondary and succeeding series of intermediate holes were drilled and grouted by split-spacing until the maximum spacing between holes along the grout line was 5 feet.

The design of the curtain as shown in the plans and specifications required splitting to a maximum allowable spacing of 5 feet, but construction personnel often discontinued split-spacing after closure to 10 feet. An unsigned memorandum to the

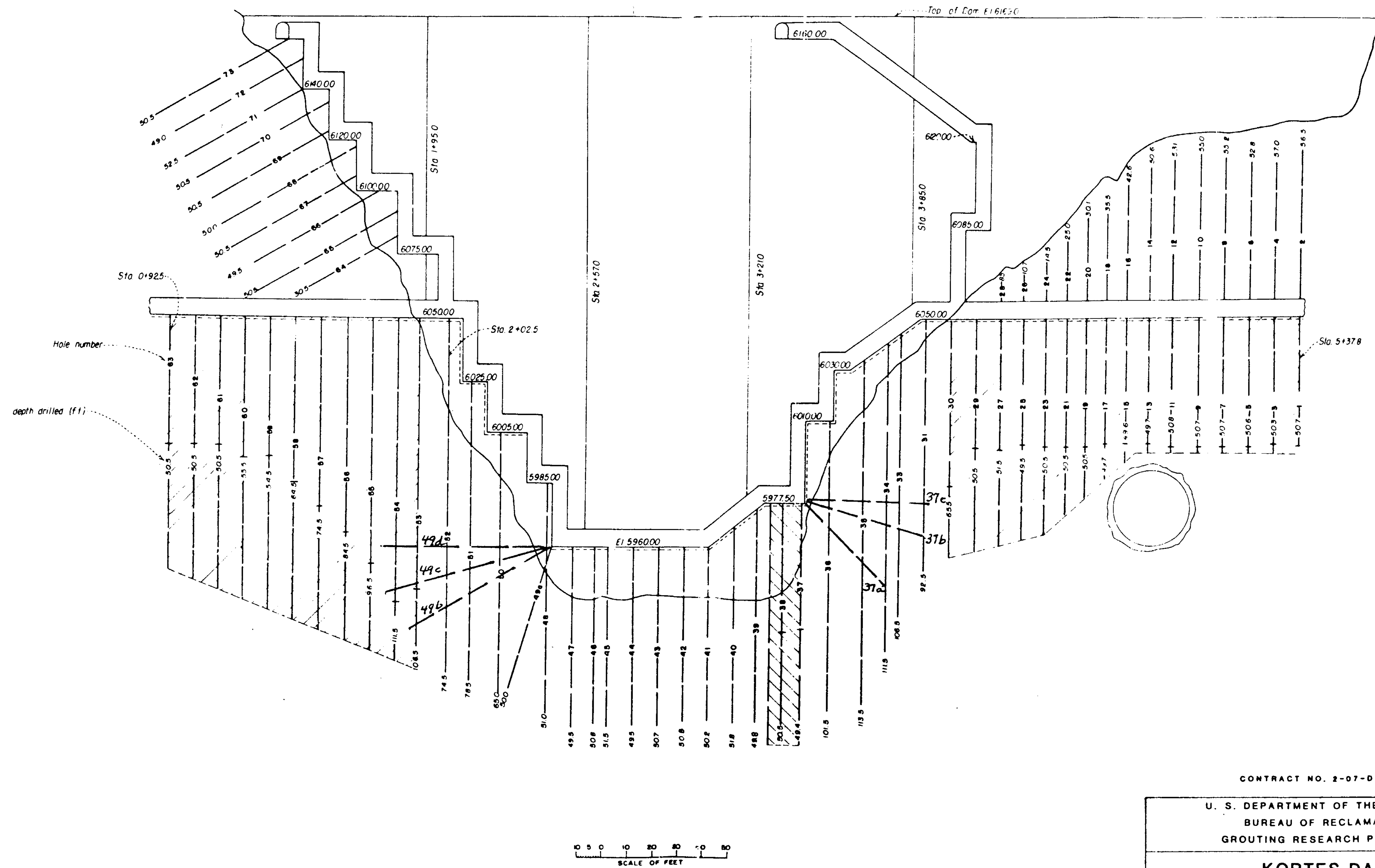
Construction Engineer (C. R. Satterfield) dated February 8, 1950, contained the following statement:

> "Cut-off curtain grouting was continued through the third intermediate series of holes in the first group which includes holes from axis station 1+95 through 3+53 ----. A modified form of the split grouting sequence series-stop grouting method, was used due to the limited number of holes and the belief that every other hole would not need to be treated."

(e) Refusal criteria. According to the specifications, the grouting of a hole was not considered complete until the hole or connection accepted the grout mixture at a rate of not less than 1 cubic foot in 15 minutes at the required grouting pressure.

(f) Final backfill of holes. The specifications required permanent grout pipes to be filled, but there was no specific requirement as to the method to be used or the means of payment for the operation. The methods used for final backfill of holes at Kortes are not shown in the records.

10. Foundation drainage holes. The drainage holes were drilled in a direction perpendicular to the axis but inclined downstream 10 degrees from vertical. The drainage holes in the upper part of the abutment were drilled 30 degrees downward from horizontal as shown on figure 15. Drainage holes were also drilled from the foundation tunnels on both abutments. On the right abutment, drainage holes were drilled upward and downward from the drainage tunnel. See figure 9 on page numbered 17 in appendix A.



CONTRACT NO. 2-07-DV-00148

U. S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GROUTING RESEARCH PROGRAM

KORTES DAM

FOUNDATION DRAINAGE

PREPARED BY: CLAUDE A. FETZER
JULY 1982 CONSULTING GEOTECHNICAL ENGINEER

REF. DWG. 545-D-773

FIGURE 15

V. ANALYSIS

11. Preconstruction geologic investigations. The preconstruction geologic investigations were recorded in Geologic Reports Nos. G-39, Febr. 5, 1945; G-50, June 3, 1945; and G-63, Oct. 2, 1945. The preconstruction geologic investigations adequately identified the main geologic features at the site including the rock types, the diabase dike, sheeted zones, and the severely jointed and fractured nature of the exposed granite in the canyon. See figures 3 and 4. Geology report No. G-63 indicated that the exposed rock in the right canyon wall was more severely broken than in the left canyon wall. The report identified two severely fractured zones in the left abutment, where the rock was broken in many directions. More excavation was to be expected in these two zones than in the other parts of the left abutment. The report did not mention a potential problem of rock falls during construction, although the photographs with the reports showed many large blocks of rocks in the talus accumulated in the canyon bottom.

Geology report No. G-86 dated Oct. 1, 1946 and prepared by C. L. Walker indicated that there was a very serious problem of rockfalls at Kortes Dam and that it was more serious than at Seminoe Dam. This report was based on a visit to the site on July 1, 2, and 3, 1946. This visit was made after the contract for the dam construction had been let in May 1946, and, therefore, cannot be considered as part of preconstruction geologic investigations.

The preconstruction geology reports mention that the dike and highly fractured zones in the left abutment must be taken into consideration when formulating the grouting program; however, the reports do not discuss any aspects of a specific design such as depth of curtain, angle and spacing of holes, anticipated grout takes, etc. The reports are considered deficient in that the geologic conditions were not related to a specific grout curtain design.

The use of 150 psi with a packer set as shallow as 10 feet in pressure testing of the exploratory borings could have caused lifting or splitting of the rock. The use of such high pressures near the surface is considered unnecessary.

12. <u>Design of grout curtain</u>. Most of the deep grout curtain was constructed using holes vertical in the upstream-downstream plane. It is considered that the curtain should have been constructed using holes angled into the abutments and criss-crossed in the valley bottom. Holes angled into the abutments as were used with the consolidation "B" holes would have had a better chance to intersect the near vertical joints in the abutments. Angle holes CC-163 and CC-165 on the upper part of the left abutment (see figure 14) were used effectively in shutting off a leak as described on pages 21 and 22 of appendix A. Other angle holes were used to intersect a dike on the left abutment.

The depth and extent of the grout curtain, which was extended in the abutments by the foundation tunnels, is considered adequate. On the left abutment, the grout curtain encompassed drill holes 10 and 11 where water losses were experienced during drilling and where moderate to high takes were found during water-pressure tests. See figure 4. In the valley bottom and on the right abutment the grout curtain encompassed all areas of water losses in drill holes. It was not economically feasible and not considered necessary to extend the grout curtain to the full depth of the "sheeted" zones as these evidently extend to a great depth.

13. Grouting methods and procedures. (a) Water-pressure tests. The procedure described for water-pressure tests would be very time consuming. The following procedure would be much faster and equally effective:

(1) If no loss of circulating water was recorded in drilling, pressure test the entire hole from the top of the rock to determine if the hole is tight. If it is found tight, no additional pressure testing or grouting is required.

(2) If the hole accepted water, lower the packer to the bottom zone and pressure test at the zone grouting pressure. If the bottom zone does not take water, raise the packer to successively higher zones until the pressure test indicates a need for grouting. This procedure is followed until all zones are tested and grouted, as required.

Note: The initial step of pressure testing would not be required if water loss is known to have occurred in drilling.

(b) Grout mixes. The basis for the starting grout mix is not presented. It is considered that the initial mix should have been based on the takes found during the water pressure tests, i.e. thin mixes should have been used for low rate of water take, and thicker mixes should have been used for higher rates of water take.

The monthly summaries of daily drilling and grouting disclose that consolidation grout holes were usually started with a water-cement ratio of 4:1 and unless leakage occurred the mix was not thickened regardless of the rate of grout take. For example in block 2, "B" hole 2-38 accepted a total of 1,153 sacks of cement at a water-cement ratio of 4:1 which was never changed throughout the grouting operation. Reportedly, there were no leaks and water was pumped in the hole "about once every 8 hours during grouting." The maximum sacks per hour was 48. A note by the inspector on the grouting records indicates there was no water loss; i.e. during drilling of hole.

A similar example is found in block 2, where consolidation grout hole 2-33 accepted 532 sacks of cement at 4:1 water-cement ratio, without a change in mix throughout the operations. The maximum rate of take was 44 sacks per hour. The hole connected to hole CC-134 at Station 1+25 in the left foundation tunnel. Both of the holes commented on above are located in a so-called "sheeted zone", and were grouted in April 1950.

Some of the cutoff curtain "A" holes were started and completed with thin mixes even though they had substantial takes. For example, Hole CC-153 at Station 0+88 accepted 894 sacks. Grouting was started and completed on a 5:1 mix although the maximum rate of take was 96 sacks per hour. Another hole, CC-142 at Station 1+05 which accepted 674.5 sacks was started with a 5:1 mix but never went below a 4:1 mix. The maximum rate of take was

142 sacks per hour, which is a very high rate of take. In both cases it is believed that thicker mixes should have been used. These holes were grouted in early March 1950.

The use of a thin mix even though the hole was taking a large quantity of grout was probably the result of observations made by O. E. Boggess on his trip to Kortes Dam, March 8-17, 1950. Paragraph 4 of his field trip report dated March 30, 1950, to the Chief Engineer is quoted below:

> "4. Several of the holes drilled from the left abutment foundation tunnel that took a considerable quantity of grout have stopped taking quite suddenly. One of these holes at the far end of the tunnel took 295 sacks of cement using a grout mixture with a water cement ratio of 2:1. When the water surface in the reservoir is above the collar of this hole, it flows almost a full hole of water. A possible explanation of this condition is that the grout being used was too thick and that the hole or seams became temporarily plugged. It is characteristic of some holes when a fairly thick grout is being used to become temporarily plugged and later open up of their own accord; especially is this true if the grout is traveling away from the hole in a downward direction. It is believed that if a somewhat thinner grout mixture is used on these free holes, this difficulty will be eliminated, and a more satisfactory job will be obtained."

A letter dated March 24, 1950 from Grant Bloodgood to Construction Engineer, Kortes Dam, Wyoming gave the following instructions:

> "Grouting from the elevation 6050 left abutment tunnel indicates the need of additional grouting in this area as shown by the flow of water from an "up" hole near the end of the tunnel. Therefore all of the holes in the tunnel should be completed to the installed 5-foot spacing. Grouting of the intermediate holes should be done with relatively thin mixes even though they will accept thicker grout for a limited period of time. It is apparent that the use of thick mixes is not adequately

> grouting the area. When a hole suddenly refuses to take grout it should be immediately washed out and tested to see if it has become prematurely plugged. Also, any previously grouted hole that may still be open should be regrouted."

The grout hole referred to in the trip report and letter is hole CC-148A, an up hole at Station 0+89. See figure 14. It was grouted on March 1, 1950 using a maximum pressure of 260 psi and a minimum pressure of 100 psi with the packer set near the surface of the gallery, El. 6,058. The maximum take per hour was 174 sacks, which is a high take. Although it took grout freely, it did have a back pressure of at least 100 psi. A review of the records does not indicate that hole CC-148A was regrouted nor that it was grouted from any other hole. The records revealed that the grout mix was thickened in several holes with large takes after the letter of March 24, 1950. In July 1950, CC-163 was grouted with a starting mix of 8:1 and ending mix of 0.8:1, and CC-165 was grouted with a starting mix of 10:1 and ended with a mix of 1:1. Boggess assisted in grouting these latter two holes.

The exact reason for the sudden stoppage of grout take in hole CC-148A is not known. There is a possibility that the joint in the rock became filled. The leakage that developed later may have resulted from shrinkage of the 2:1 grout as it set. It is very common for holes with a final mix thinner than 0.8:1 to develop leakage afterwards because of grout shrinkage. It is not considered that the use of 2:1 grout in CC-148A was the reason for the sudden refusal in the hole. It is considered that proper procedures were followed on holes grouted later when the mix was thickened on high-take holes.

(c) Grout-injection pressures. The rules used for determining the injection pressures at the collar are considered to permit pressures considerably on the unsafe side as explained below:

(1) The use of 1 psi for each vertical foot of concrete directly above the hole being grouted is unsafe. The structure will produce a static loading of about 1 psi at the foundation contact under the highest part of the structure; but due to the dam configuration, the unit load may be appreciably less under other parts of the structure where the applied uplift from the holes may also be acting. Furthermore, if grouting is accomplished with a partial or full pool behind the dam, the effective weight of the structure may be reduced by uplift and the distribution of the load may also be drastically changed to a high unit load at the toe and a low unit load at the heel where the grouting gallery is located. If the packer is set in the rock at depth, the effective unit load of the structure will be reduced by the spread of the load.

(2) The use of 2 psi for each linear foot of rock as measured between packer setting and nearest point of rock-concrete contact is unsafe. Most rock will weigh about 1 psi per foot of depth but the effective weight may be affected by the position of the water table and by the configuration of the rock surface. The angle of the hole must also be considered in computing the effective weight at the packer. The extra 1 psi in

the rock factor is dependent upon the rock acting as a beam. In highly fractured or faulted rock it may be on the unsafe side to include any additional permissible pressure due to the rock strength.

(3) The use of an additional pressure of 50 psi for completed blanket grouting is also unsafe. This rule assumes that the flexural strength of rock has been appreciably increased by the blanket grouting as the blanket grouting would not add any appreciable static weight to the rock. Blanket holes are usually on 20-foot centers and could not be expected to seal and heal all fractures in the rock surface and especially in the "sheeted zones".

(4) Progressively increasing the pressure for each successive series by 12-1/2 percent over the preceding series is also unsafe. The purpose of split-spacing holes is to determine that the curtain is getting tighter. By progressively increasing the pressure, the rock may be split by the third or fourth intermediate holes thereby damaging previously placed grout. Furthermore, there is no need to apply additional pressure with the closer spacing as it is not necessary to move grout as far in the closure holes as it was in the primary holes, where most grout penetration occurs.

(5) The fact that the rules are presented on the basis of the collar pressure is also unsafe as the effective uplift at the packer includes the gage pressure and the pressure due to the column of grout (dependent on grout mixture) in the line between the gage and the packer.

It is considered that the safe grout pressure should be computed at the packer based upon the effective weight and/or strength of the rock available to resist the uplift. The gage pressure should be determined by deducting an appropriate amount from the safe pressure at the packer for the fluid pressure in the line between the packer and the gage.

(d) Spacing of holes and closure. As shown on figure 14, there was only split spacing to 10-foot centers between Stations 2+25 and 3+35.3. According to Boggess's field trip report of March 30, 1950:

> "2. The drilling and grouting of the "A" line grout curtain between Station 1+95 and Station 3+85 (river channel area) has been completed. This area was generally quite tight and with the exception of one of the first holes grouted which accepted 423 sacks of cement, took very little grout. It was not considered necessary to drill and grout on a spacing closer than 10 feet in this area. It is expected that the grout take in the right abutment will, also, be quite low."

A review of the grout takes on figure 14 indicates many areas where 5-foot closure holes could have been justified or even closer if necessary to obtain a tight curtain. These include on either side of the hole at Station 3+14 with a take of 432.2 sacks, on either side of the hole at Station 2+74 with a take of 117.7 sacks, between the holes at Station 2+25 and Station 2+35.5 with takes of 17.8 and 44.4 sacks, between the holes at Stations 3+78, 3+93 and 4+03 with takes of 15.2, 79.5 and 53 sacks, etc. In addition all holes located in "sheeted zones" or holes grouted by communication, holes lost due to premature plugging or as a result of running out of cement, plant breakdown, etc. should have been split to 5 feet.

In the consolidation grouting, up to four holes were connected to one header (drilling and grouting note for June 6, 1948). In this case it would be difficult to determine which holes took the grout and where the split spacing should be made.

(e) Refusal criteria. The field records indicate that holes taking less than 0.5 cubic foot of water in 5 minutes were not connected for grouting. It would, therefore, seem practical and save considerable time if refusal had been based on a take of less than 1 cubic foot of grout mixture in 5 or 10 minutes rather than 15 minutes.

(f) Final backfill of holes. All completed grout holes should be sounded with a heavy rod and backfilled by the tremie method with a thick grout having a water-cement ratio in the range of 0.7:1 to 0.8:1. The contractor is usually paid for one connection even though the operation may have to be repeated several times to top out the hole with solid grout.

14. Unexpected geologic conditions discovered during grouting. The leakage path found on the left abutment when the reservoir water surface was above El. 6,060 was unexpected as indicated by this statement from Geology Report No. G-63:

> "The abutment and foundation rock of the proposed Kortes Dam will be almost as severely fractured at moderate depths as it is on exposed rock surfaces. The distribution of the fractures in the drill core supports this conclusion (see figure 1). However, the majority of the fractures are tight, simple

> joints, except in the surface weathered zone. Both water loss during pressure tests and the percent of core loss were exceedingly low (see drill logs bound with this report)."

15. <u>Grout takes as related to geologic conditions</u>. Most of the grout take in the consolidation grouting, "B" holes, occurred in blocks 2 and 3 between Stations 1+29 and 2+30. See figures 11 and 12. The large takes in the cutoff curtain, "A" holes, occurred also in this same area of the left abutment. Many of the high grout takes were found above and below the diabase dike near the top of the left abutment. See figures 4 and 14. DH-10 which passed through the diabase dike had higher water takes in several zones tested. In gravity testing at a drill-water loss at El. 6,138, the hole could not be filled at an inflow rate of 32 gpm. There were significant core losses between El. 6,163 and El. 6,157 and between El. 6,090 and 6,075. There was also a 16% loss in a 2.2-foot section between depths of 129 and 131.2 feet (El. 6,062.5 to El. 6,060.6) and water stood in the hole at a depth of 132 feet after drilling.

DH-11 extends from El. 6,083 to approximate El. 5,953. The rate of take in the water tests were low to moderately high including takes of 20 gpm at a collar pressure of 26 psi between depth intervals 50 to 60 feet and 80 to 89.7 feet. There was also a take of 26 gpm at a collar pressure of 40 psi between depth intervals of 60 to 70 feet. The water level in the hole was at El. 5,990 after the hole was completed.

Alteration of the rock creating pervious zones should be expected in the area of intrusive diabase dikes. The water loss zones and high water takes found in the fractured rock encountered in DH-10 and DH-11 indicated that high grout takes could have been expected on the left abutment.

Water losses on the right abutment in DH-12 and DH-13 below the near-surface material, which was mostly removed in excavation, were much lower than found on the left abutment. Water losses in the valley bottom in DH-5, DH-6 and DH-7 were extremely low. These data indicated that the grout takes in the right abutment and in the valley bottom should have been expected to be appreciably lower than the grout takes in the left abutment.

16. Effectiveness of grouting and drainage. The main objective of the grouting and drainage is to control the seepage and uplift to within tolerable and safe limits. A secondary purpose of the "B" hole grouting is to fill the voids directly beneath the structure to decrease the foundation movements from the dead and live loads.

(a) Seepage. After the leak through the left abutment was sealed by grouting in July 1950, the quantity of seepage through the abutments, along the abutment contacts and from the foundation drains has been very low. However, rapid accumulation of calcium carbonate in the foundation drains has been a continuing problem.

The recorded drain flows for 1980 and 1981 are shown on figure 16. It is assumed that the El. 5,977.5 weir represents the flow from the right abutment and that the flows from the left

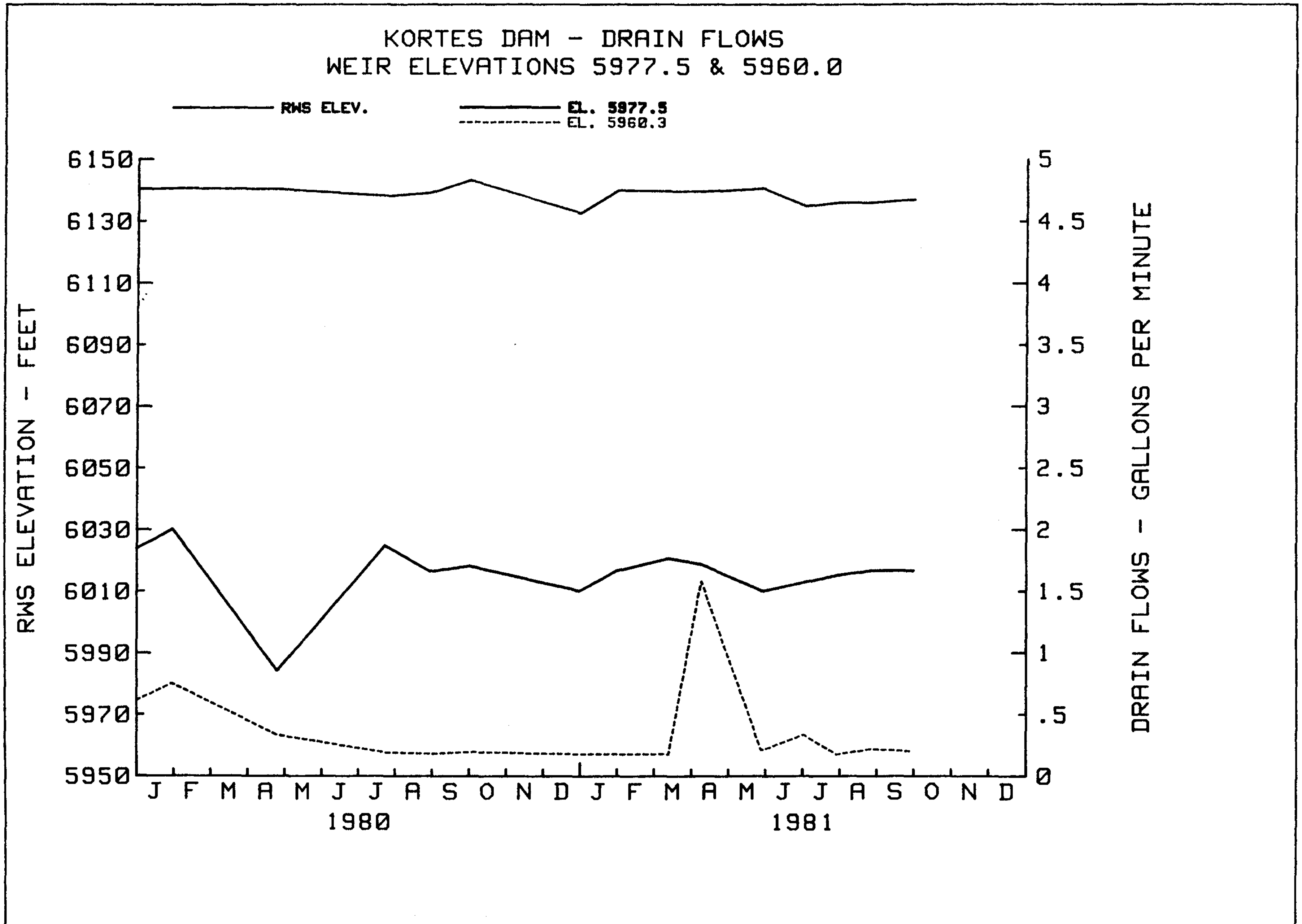


FIGURE 16

Header file 7KOSMA

abutment and bottom are collected by the El. 5,960 weir. These data indicate that the seepage flows from the drains are very low as the combined flow from the two weirs is approximately 2 gpm. The SEED inspection of November 3, 1980, found a small amount of seepage emitting from the downstream left abutment adjacent to the contact; this seepage had been found on previous inspections and was not increasing. A minor seep was also found on the right abutment near the spillway lip. These data indicate that the grouting has adequately controlled the seepage, and that the amount of seepage is relatively small for a dam with a hydraulic head of approximately 200 feet.

(b) Uplift. Uplift was recognized as a problem during the initial filling in 1951. The uplift, particularly at the B pipes, was found to be higher than the values used in design and the uplift was increasing with time under a constant head. The layout of the uplift measurement system is shown on figure 17, and the uplift pressures for the period of 1951-1981 are plotted on figures 18 through 21.

Various tests were conducted on the drains in early June 1951, by O. E. Boggess to determine the effect of shutting off and pressurizing various drains on the uplift gages. It was decided that more drains were needed. Six additional drains were installed in January and February 1952, by a government crews. These drains were numbered 37a, b and c and 49 b, c and d and are shown on figure 15. A summary of the driller's logs is presented in table 1, which was taken from a report from the District

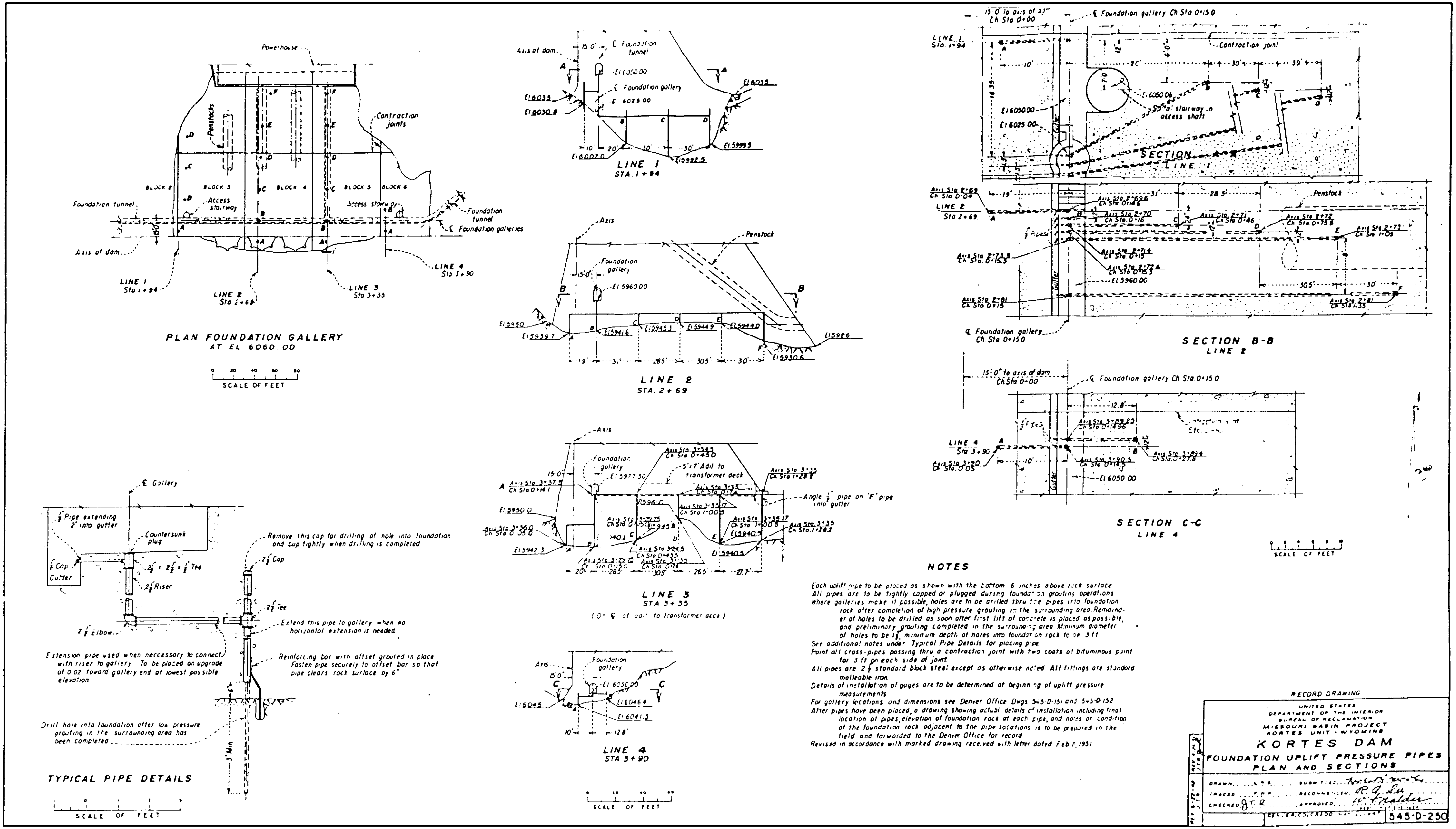

PLAN FOUNDATION GALLERY
AT EL 6060.00
SCALE OF FEET
LINE 1
STA. 1+94
LINE 2
STA. 2+69
LINE 3
STA 3+35
LINE 4
STA 3+90
SECTION B-B
LINE 2
SECTION C-C
LINE 4
TYPICAL PIPE DETAILS
NOTES
Each uplift pipe to be placed as shown with the bottom 6 inches above rock surface
All pipes are to be tightly capped or plugged during foundation grouting operations
Where galleries make it possible, holes are to be drilled thru the pipes into foundation rock after completion of high pressure grouting in the surrounding area. Remainder of holes to be drilled as soon after first lift of concrete is placed as possible, and preliminary grouting completed in the surrounding area. Minimum diameter of holes to be 1½, minimum depth of holes into foundation rock to be 3 ft.
See additional notes under Typical Pipe Details for placing pipe
Paint all cross-pipes passing thru a contraction joint with two coats of bituminous paint for 3 ft on each side of joint
All pipes are 2½ standard black steel except as otherwise noted. All fittings are standard malleable iron
Details of installation of gages are to be determined at beginning of uplift pressure measurements
For gallery locations and dimensions see Denver Office Dwgs 545-D-151 and 545-D-152
After pipes have been placed, a drawing showing actual details of installation including final location of pipes, elevation of foundation rock at each pipe, and notes on condition of the foundation rock adjacent to the pipe locations is to be prepared in the field and forwarded to the Denver Office for record
Revised in accordance with marked drawing received with letter dated Feb 8, 1951
RECORD DRAWING
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MISSOURI BASIN PROJECT
KORTES UNIT - WYOMING
KORTES DAM
FOUNDATION UPLIFT PRESSURE PIPES
PLAN AND SECTIONS
545-D-250

FIGURE 17

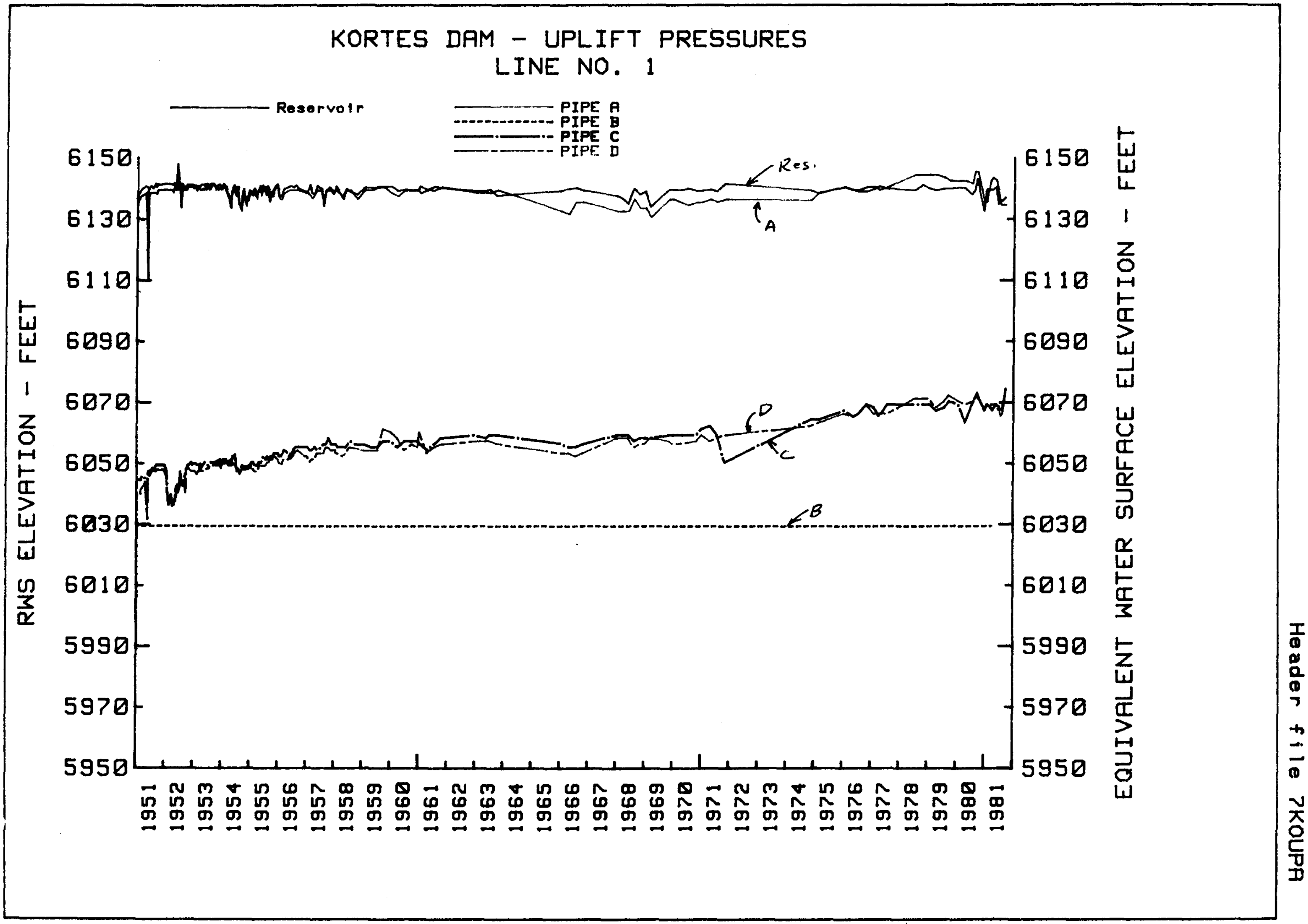
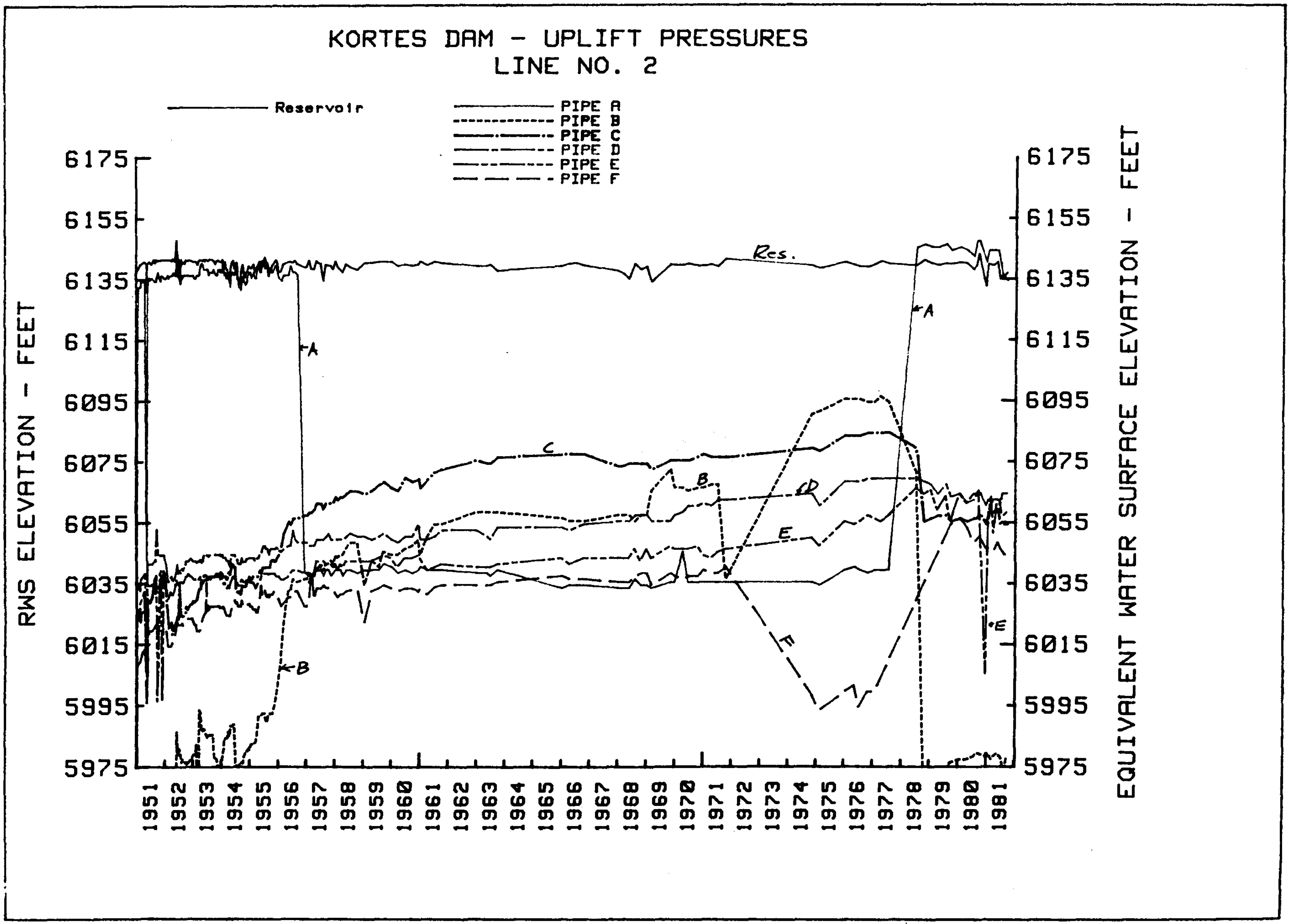


FIGURE 18

ENGINEERING & RESEARCH CENTER - STRUCTURAL BEHAVIOR SECTION



ENGINEERING & RESEARCH CENTER - STRUCTURAL BEHAVIOR SECTION

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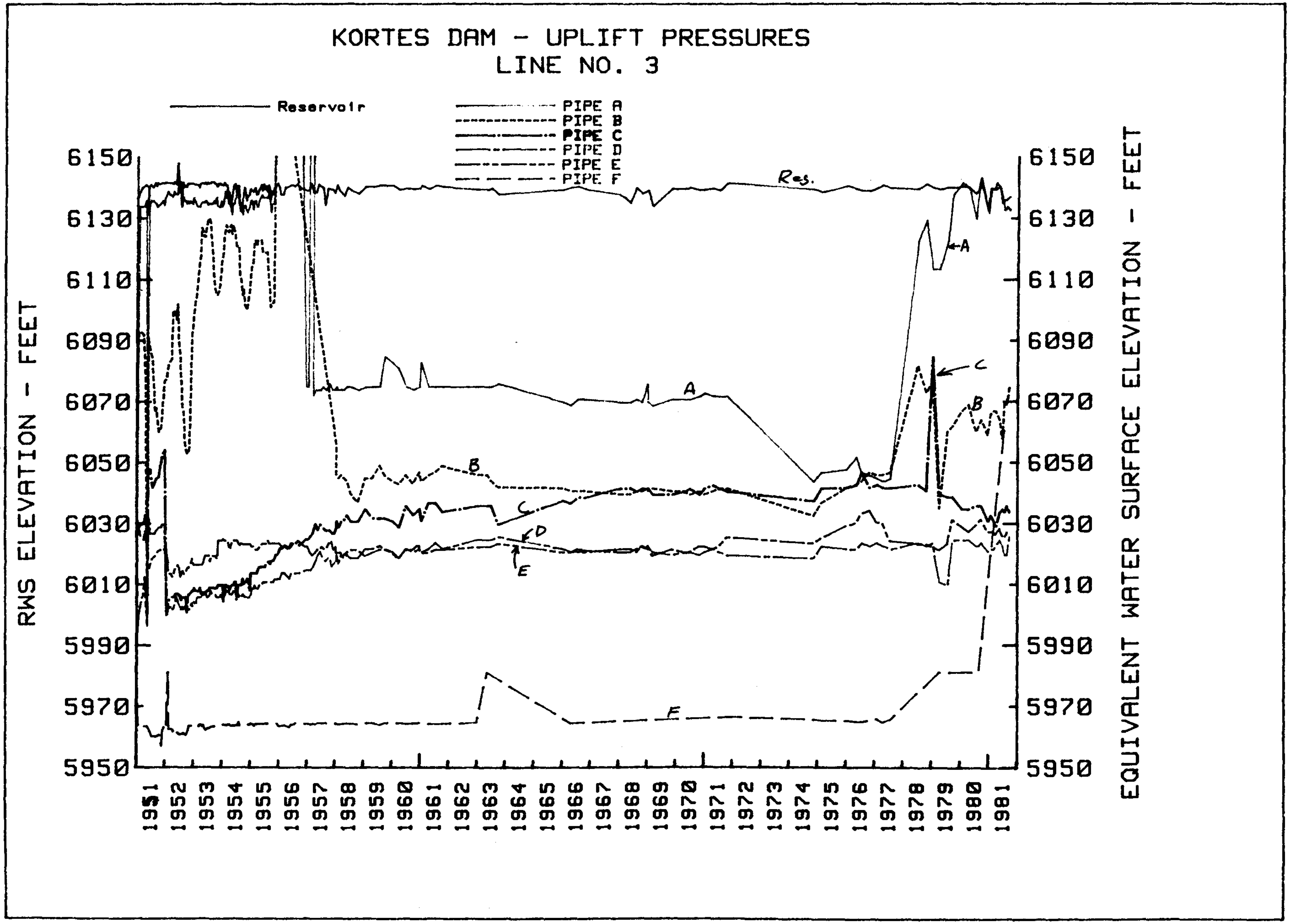
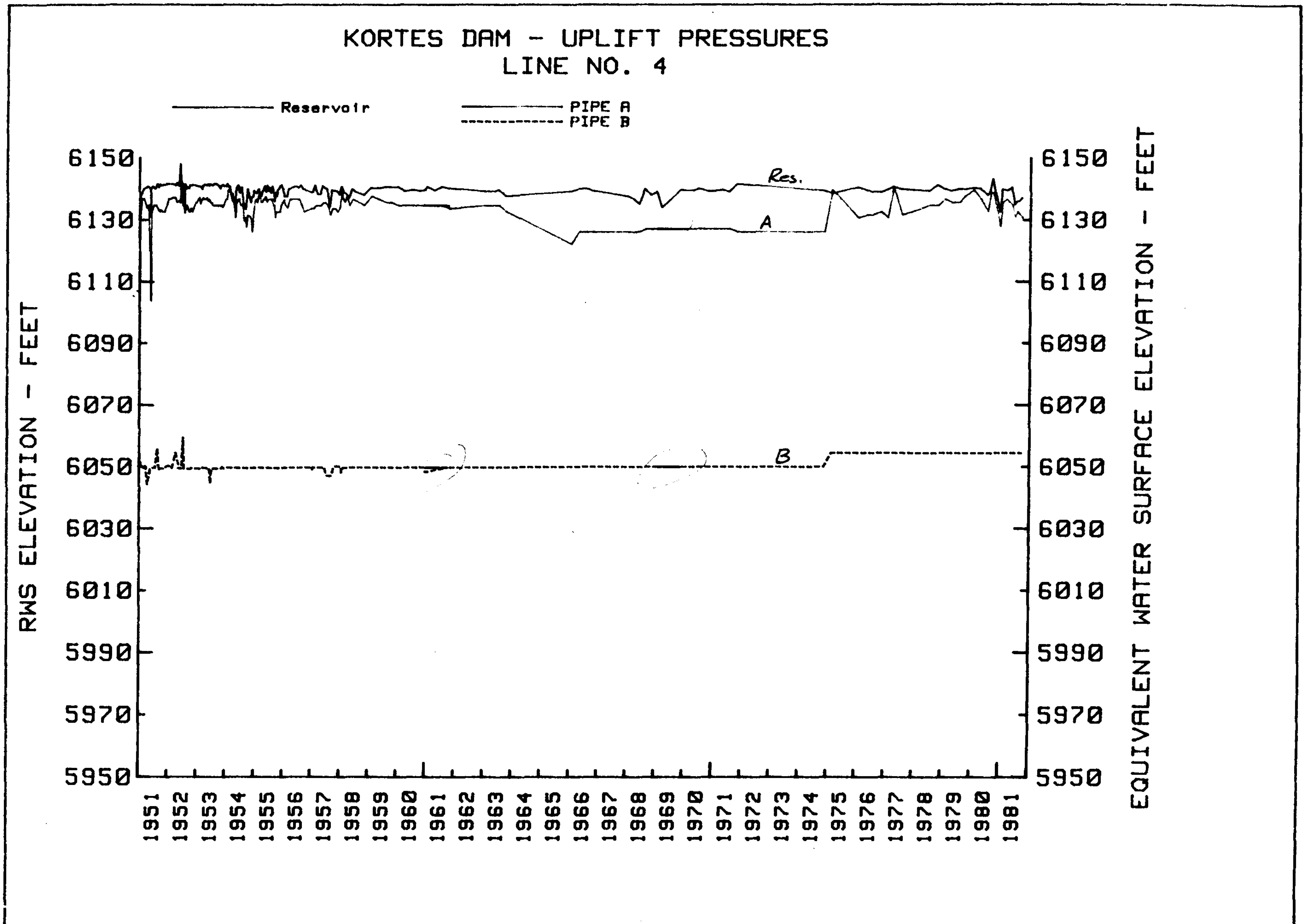
FIGURE 19



Header file 7KOUPC

FIGURE 20



Header file 7KOUPD

FIGURE 21

Table 1

Summary of Driller's Logs

Hole 37a (vert. angle from horz.-45°; horz. angle from axis-25°; drilled - 1/21 to 1/23/52)

0 - 4'	Concrete, Six-inch core
4' - 17'	Concrete, Nx core
17' - 50'	Granite, some fractures, almost 100% core recovery, while drilling from 44 to 47.8 water come out hole 37b, stopped when drill water in this hole shut off.

Hole 37b (vert. angle from horz.-15°; horz. angle from axis-18°; drilled - 1/14 to 1/18/52)

0 - 8'	Concrete, Six-inch core
8' - 9.8'	Concrete, Nx core
9.8' - 50'	Granite, fractured, almost 100% core recovery, water pressure pushed rods out of hole at 35.8', 3" pipe ran full for awhile, then went down, hole 33, 34, 35 and 36 dried up.

Hole 37c (vert. angle from horz.-0°; horz. angle from axis-21°; drilled - 1/8 to 1/14/52)

0 - 9.5'	Concrete, Six-inch core
9.5' - 50.3'	Granite, fractured, some quartz stringers, almost 100% core recovery.

Hole 49b (vert. angle from horz.-30°; horz. angle from axis-24°; drilled - 2/4 to 2/7/52)

0 - 4'	Concrete, Six-inch core
4' - 17'	Concrete, Nx core, drilled through 4" pipe at 8'
17' - 18.4'	Concrete and iron, reduced to Bx as Nx barrel was hanging up on pipe drilled through at 6'.
18.4' - 70'	Granite, fractured except between 40'-50' and 55'-85', close to 100% core recovery.

Hole 49c (vert. angle from horz.-15°; horz. angle from axis-16°; drilled 1/29 to 2/1/52)

0 - 4'	Concrete, Six-inch core
4' - 17.4'	Concrete, Nx core
17.4' - 70'	Granite, fractured except for last 20'; almost 100% core recovery.

Hole 49d (vert. angle from horz.-0°; horz. angle from axis-15°; drilled 1/24 to 1/29/52)

0 - 4'	Concrete, Six-inch core
4' - 21'	Concrete, Nx core
21' - 70'	Granite, fractured, almost 100% core recovery, contacted hole 54 at 57.5', drill hole ran full 3" stream of water for awhile, decreased to about 5 gpm now, dried up drainage holes 51, 52, 53, 54, 55, 56.

Geologist to the District Engineer dated Feb. 28, 1952, subject, "Report of Drain Drilling, Kortes Dam".

The logs of the additional drainage hole indicate that much of the rock was fractured and that water under considerable pressure was encountered in 37b. The angle holes intersected the water-carrying fractures and discharged it at a much lower level than the adjacent vertical holes, thereby causing the vertical holes to dry up. The drilling of the additional drains resulted in a considerable drying up of the abutments and a reduction in the uplift pressures.

The SEED report indicates that additional foundation grouting was done after construction, but there is no record in the available documents to indicate that any additional grouting was accomplished nor does Fred H. Lippold recall any foundation grouting at Kortes after construction.

With reference to the 1951-1981 plots of uplift pressures, line 1 indicates an increasing uplift for pipes C and D. In line 2, pipes B, C, D and E were gradually increasing until 1978 when they were lowered, possibly from drain cleaning. In line 3, pipe B gave the high uplift which was of major concern in the early 1950's. In 1957 the uplift readings for pipe B were reduced by about 100 feet to about El. 6,050. Since 1978, the uplift readings for pipe B have risen about 20 feet in an erratic manner. The other pipes in line 3 have remained fairly constant except pipe F has recently risen about 100 feet. There have been only very small changes in uplift readings for the pipes in line 4.

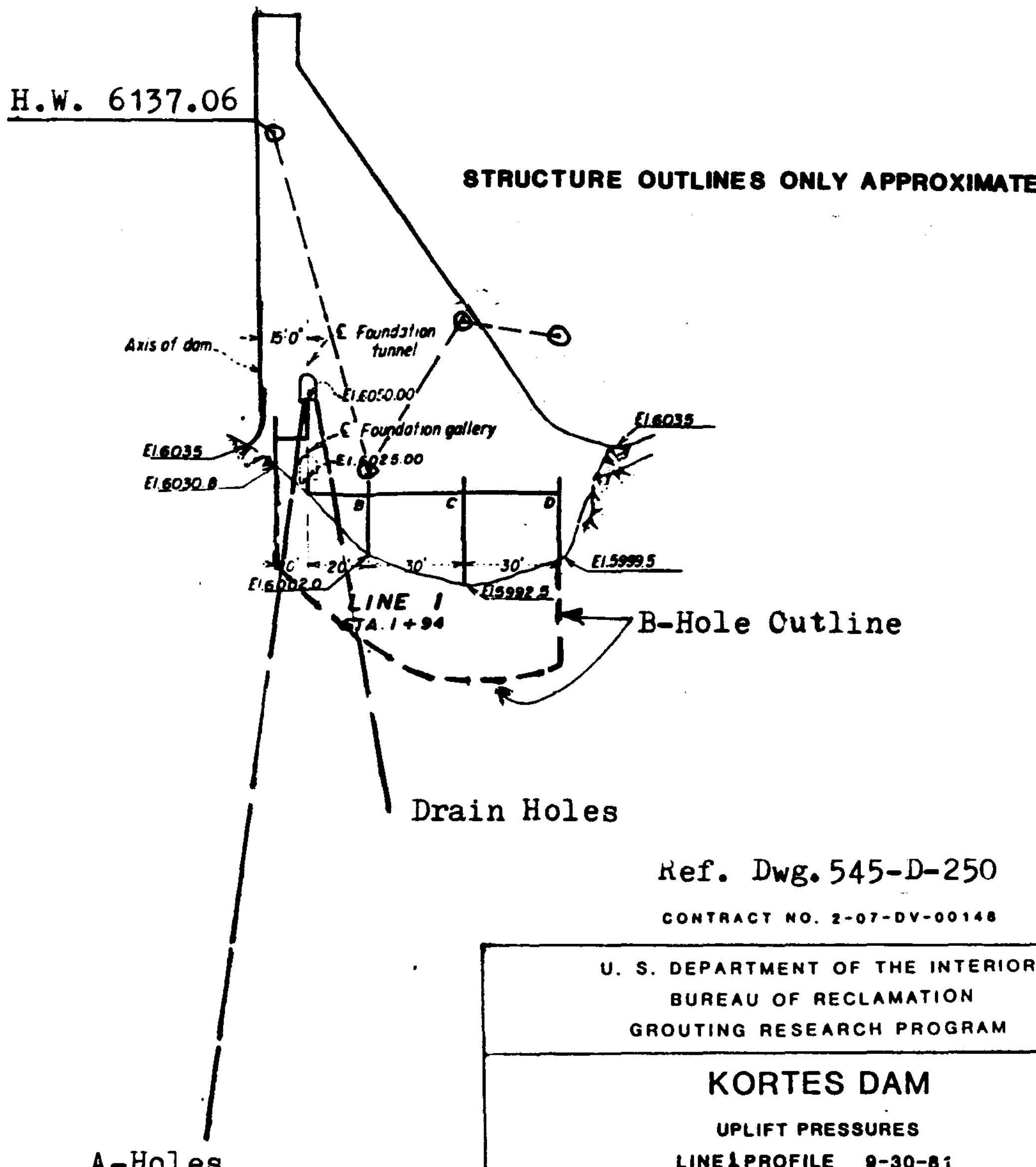
The uplift readings for September 30, 1981 are plotted on the dam cross-sections on figures 22 through 25. On line 1, there is an approximate drop in pressure of 80 feet between pipes A and C and D across the grouting and drainage lines; the pressure in pipe B has been holding steady at El. 6,030 for 30 years and may not be responsive.

On line 2, the pressure in pipe A, which is upstream of the A line, rose approximately 100 feet to its present level in 1978 and now is near the reservoir level. The pressure in pipe B fell dramatically, about 120 feet, in 1978 to its present level which is near tailwater. The pressures in pipes C, D, E and F are near El. 6,055 which presents an 80-foot drop across the grouting and drainage lines.

On line 3, the pressure in pipe A also rose dramatically in 1978 to its present near reservoir level. The pressure in pipe B which is just downstream of the A-line has an approximate drop of 65 feet from pipe A. The pressures in pipes C, D, and E are about 40 feet lower than in pipe B. The pressure in pipe F, the furtherest downstream, had a dramatic rise of about 80 feet in 1981; this pipe and its gage should be thoroughly tested.

On line 4, the pressure in the upstream pipe, A, is fairly close to reservoir level. The pressure in pipe B, which is downstream of the grouting and drainage lines, is about 90 feet lower than in pipe A.

A complete history of the operation and maintenance of the uplift system is not available, nor is a complete history of the cleaning of the drainage holes available; hence, a thorough



Ref. Dwg. 545-D-250

CONTRACT NO. 2-07-DV-00146

U. S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GROUTING RESEARCH PROGRAM

KORTES DAM

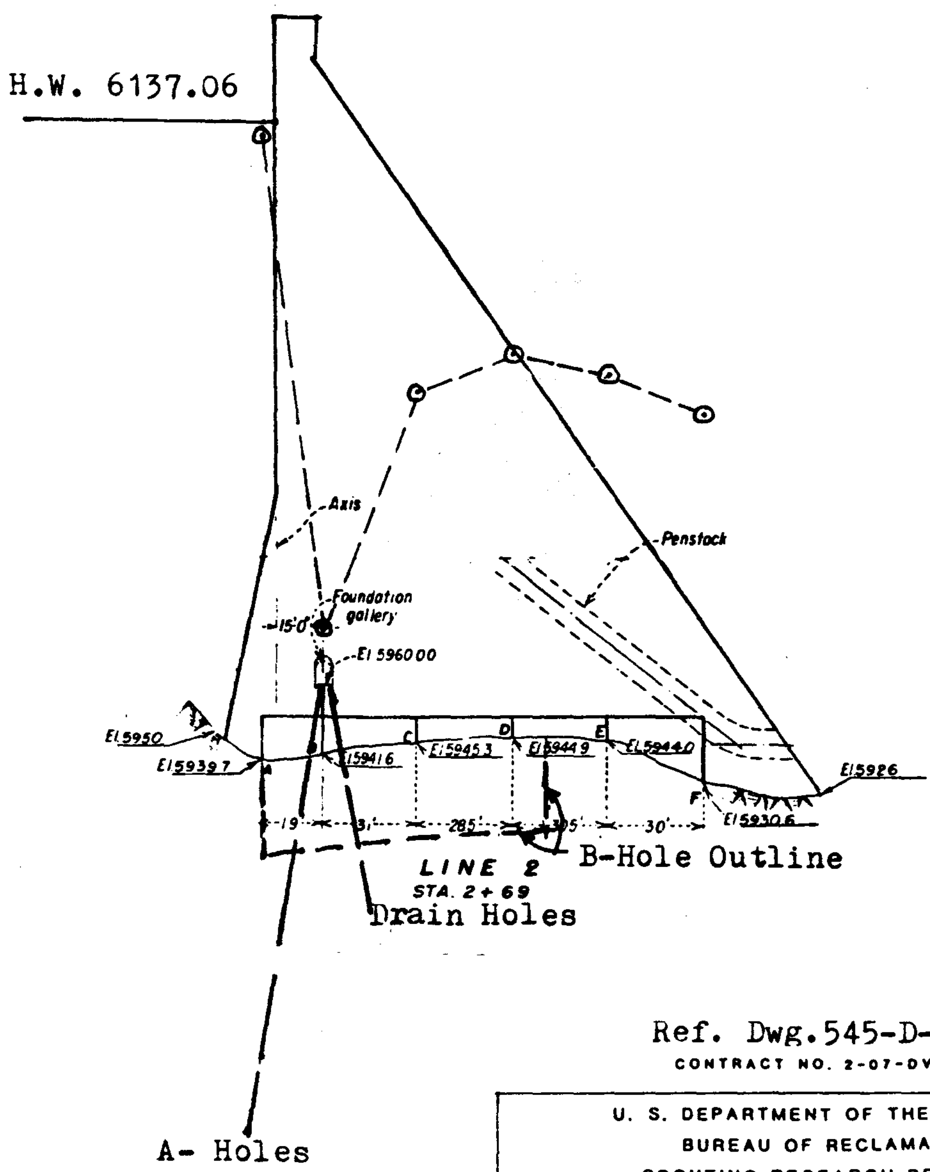
UPLIFT PRESSURES

LINE 1 PROFILE 9-30-81

PREPARED BY: CLAUDE A. FETZER

JULY 1982 CONSULTING GEOTECHNICAL ENGINEER

FIGURE 22



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CONTRACT NO. 2-07-DV-00148

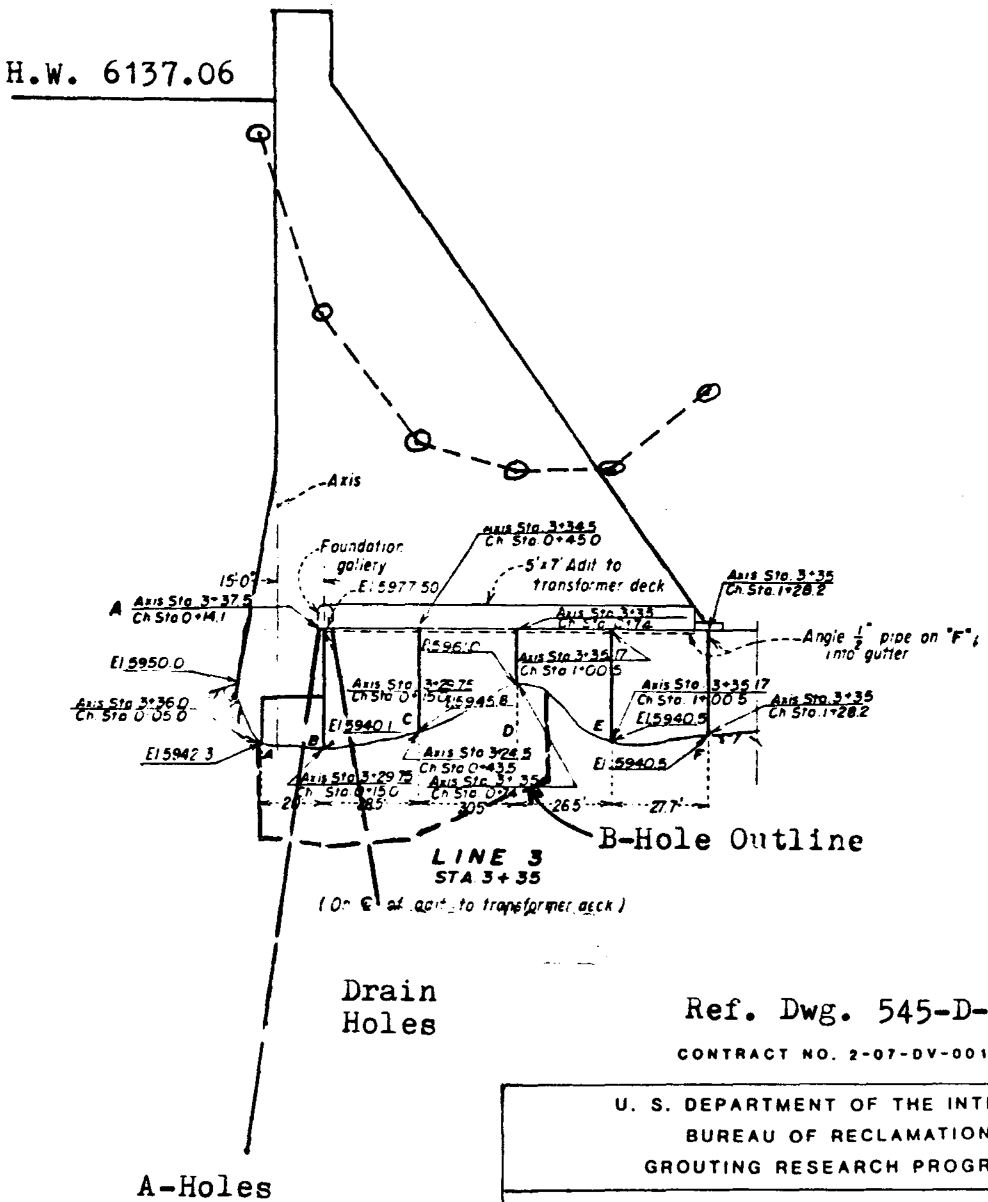
U. S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GROUTING RESEARCH PROGRAM

KORTES DAM

UPLIFT PRESSURES
LINE 2 PROFILE 9-30-81

PREPARED BY: CLAUDE A. FETZER
JULY 1982 CONSULTING GEOTECHNICAL ENGINEER

FIGURE 23



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GROUTING RESEARCH PROGRAM

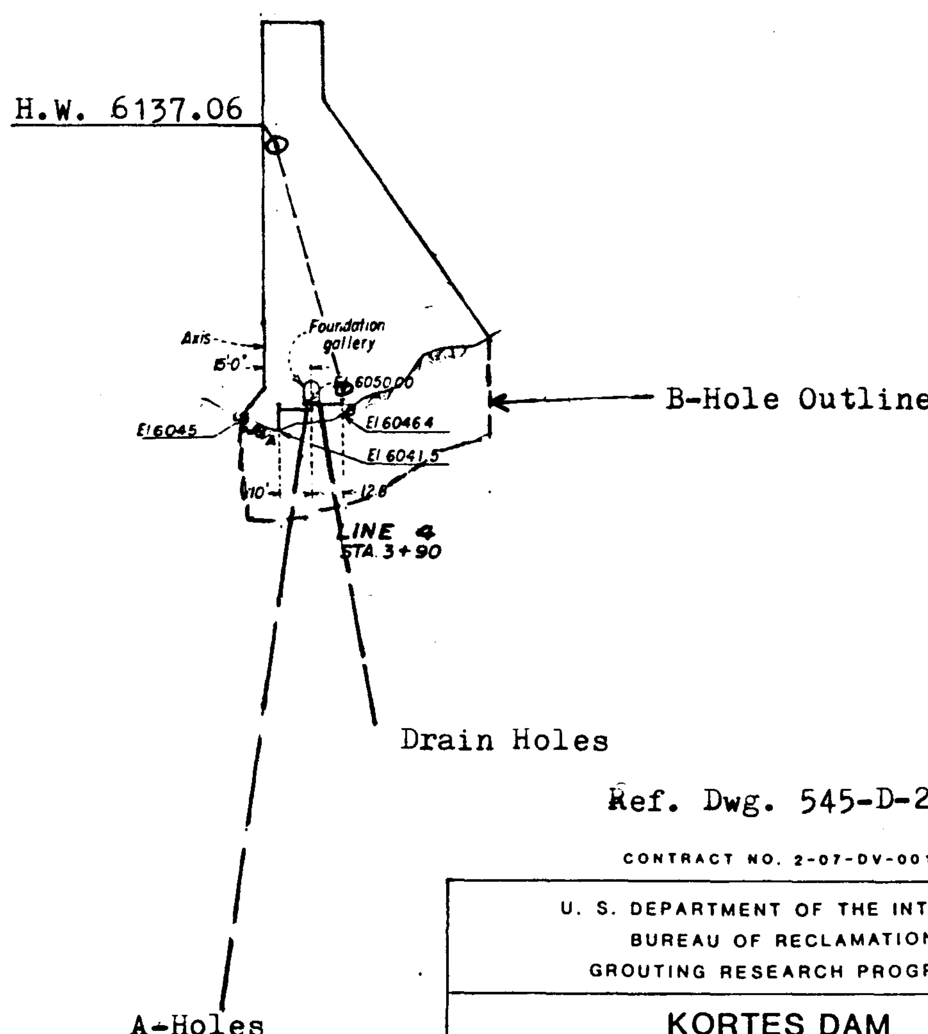
KORTES DAM

UPLIFT PRESSURES

LINE 3 PROFILE 9-30-81

PREPARED BY: CLAUDE A. FETZER
JULY 1982 CONSULTING GEOTECHNICAL ENGINEER

FIGURE 24



Ref. Dwg. 545-D-250

CONTRACT NO. 2-07-DV-00148

U. S. DEPARTMENT OF THE INTERIOR BUREAU OF RECLAMATION GROUTING RESEARCH PROGRAM
KORTES DAM UPLIFT PRESSURES LINE 4 PROFILE 9-30-81
PREPARED BY: CLAUDE A. FETZER JULY 1982 CONSULTING GEOTECHNICAL ENGINEER

FIGURE 25

evaluation of the effects of grouting and drainage on the dam uplift cannot be made. Assuming that the available data is correct, it is concluded that the grouting has reduced the seepage volume sufficiently but that the drainage system has not reduced the uplift pressures sufficiently near the downstream toe of the dam. An additional line of drainage holes is needed along the downstream toe to reduce the hydrostatic uplift line below the face of the dam. The drains would have to be insulated at the surface to prevent freezing in the winter time.

The uplift pipes are connected to holes only 3 feet deep into rock. Uplift measuring devices drilled and sealed into rock at a deeper level could have appreciably different pressures than found at the base of the structure. For a massive gravity structure such as Kortes and without evidence of weak shear planes below the structure, higher pressures at depth may not be critical. However, for a thin arch dam such as Morrow Point, the consequences of high uplift pressures at any level near the rock surface in the foundation or in the abutments could be very significant from a safety standpoint.

VI. RECOMMENDATIONS

17. Kortes Dam.

(1) The dam uplift system be thoroughly bled and all gages tested for accuracy. Many of the readings are very erratic and appear to be unreasonable.

(2) As the drainage holes rapidly accumulate calcium carbonate, coring of the grouted area beneath the dam be accomplished under Phase II to determine if the grout has been piped out of the fractures. Areas of high grout take should be selected for the coring.

(3) An additional line of drainage holes be installed along the downstream toe of the dam to reduce the high uplift pressures. Vertical holes can be drilled across the bottom, and a fan of angled holes can be drilled at the base of the abutments. The tops of the drainage holes and the collector pipe would have to be well insulated to prevent freezing in the winter time. The collection system could be discharged into a low sump in the powerplant.

18. Other concrete dams.

(1) Preconstruction geologic investigations include an evaluation of the need for grouting and should include an evaluation of the geologic conditions as related to the specifics of a grout curtain design, i.e. depth of different curtains after excavation, angle and spacing of holes, anticipated grout takes, etc. Similar evaluation should be made for the drain holes.

(2) The pressures used in water tests in explorations be limited to pressures that will not lift or split the rock.

(3) The grout holes and drain holes be angled to intersect the fractures and joints and other geologic features such as dikes and "sheeted" zones.

(4) The specifications include requirements for: distance between drilling and grouting operations; a time limit for water pressure tests; minimum capacity of grout pump; minimum capacity of mixing tubs and agitator sump; minimum size of hole and inside diameter of pipe in the packer; sounding and final backfill of completed grout holes; test drilling with NX holes.

(5) The starting grout mixes be based on the takes in the water pressure tests and that the mix be thickened in accordance with an evaluation of the takes and back pressures. Where there is concern about losing the hole, gradually reduce the water-cement ratio in 1/4 or 1/2 units rather than whole units. To reduce the amount of shrinkage in the grout, the thickest mix that can be injected without plugging the hole should be used.

(6) Grout injection pressures and water-test pressure be based on a balance of effective pressures at the packer. For abutments, the safe pressures may be limited by the shear strength of horizontal or dipping weak seams in the rock, and rock bolting may be required before grouting starts.

(7) Closure to the design spacing be made unless there is positive evidence that the curtain is tight at a wider spacing.

(8) All completed grout holes be sounded and filled with thick grout by tremie method - several times if required to fill hole with solid grout.

(9) Results of drilling and grouting be continuously evaluated by project geologist and by supervising engineer to make adjustments due to encountering unexpected geologic conditions. Liberal use of core drilling be made when questions arise on flow of grout, etc.

(10) For dams with wide bases, additional foundation drainage lines be installed downstream.

(11) Drain holes be installed in rock of the valley walls immediately downstream of the dam to reduce possibility of rock slides and rock falls due to build up of water pressure from the reservoir seepage.

(12) Uplift pipes be installed in weak seams or open joints at depth below the base of the dam.

APPENDIX

APPENDIX A

CHAPTER III--GROUTING AND DRAINAGE

12. General Plan. - The general plan for grouting the foundation rock under the dam provided for preliminary low-pressure shallow grouting to be followed by final high-pressure deep grouting. The shallow holes for low-pressure grouting are designated on the drawings as "B" holes, and the deep holes for high-pressure grouting are designated as "A" holes (see fig. 8). Prior to the placement of concrete in the dam, the near-surface rock of the foundation under the upstream portion of the dam was grouted by means of the shallow B holes, approximately 30 feet in depth. The purpose of this relatively shallow grouting was to secure general solidification (consolidation) of the upper layers of rock so that deeper penetration and more effective sealing could be secured with the higher pressures used in the deeper A holes or the main cutoff curtain. The main cutoff curtain grouting consisted of grouting under high-pressure, holes drilled at approximately 5-foot centers on a line downstream from and parallel to the axis of the dam. The holes in the cutoff line were inclined in an upstream dip of approximately 82° from the horizontal and extended to depths necessary to grout the foundation to a depth of 100 feet from the concrete contact. Basically, the same drilling and grouting equipment was used to drill and grout both the A and B holes. In general, blocks of the dam were used to identify specific areas of the dam foundation for grouting operations. For location of dam blocks, see figure 4.

Drainage for the rock forming the dam foundation was provided by a series of holes drilled approximately 50 feet into the foundation rock at approximately 10-foot centers and along a line parallel to and downstream from the grout curtain.

A system of pipes was embedded in the dam at the rock and concrete contact at certain locations, to record water pressure or uplift at these points. Installation of grout pipe, drain pipe, and uplift pipe in the dam foundation is shown in figure 9. All foundation treatment was performed by the prime contractor, under provisions of specifications No. 1151.

13. Drilling and Grouting Equipment. - Equipment used for drilling holes consisted of four diamond core drills. These drills were powered either by electric or air motors, ranging from 7-1/2 to 16 horsepower. Drilling operations in the foundation gallery of the dam are shown in figure 10.

The grout plant consisted of a grout mixer, agitating sump, grout pump, grout lines, and injection header equipped with pressure gages. The mixer consisted of a horizontal cylinder mixing chamber, a horizontal propeller shaft and mixing blades operated by an air motor, and a charging hopper with a 4- by 1/4-inch cover screen. Mixing water was measured by a meter as it entered the top of the mixer. After mixing, the grout was discharged through a 1/4-inch screen into a pump sump. The sump was equipped with a mechanical agitator. Air and electric motors were both used to power the grout pump at different times during the grouting operations. A 6- by 3-1/2- by 6-inch, double-acting grout pump, equipped with rubber valves and iron-fluid pistons with removable rubber sleeves, was used. The pump was converted for higher pressures by reducing the size of the cylinder liners and the fluid piston. Grout was pumped through a system of steel pipe and rubber hose that extended to the drill hole and returned to the pump. In general, a 1-1/2-inch-diameter supply line and a 1-inch-diameter return line were used. The grout injector header was a "tee" arrangement of pipe equipped with control valves and a pressure gage, by which grout was transmitted from the circulating line into the hole. One of the valves regulated the flow of grout into the hole and another regulated the flow in the line returning to the pump. The desired pressure was maintained on the hole by regulating these valves, and grout in excess of that accepted by the hole was returned to the sump for recirculation. A pressure gage reading in pounds per square inch was connected on the injector header just below the valve that controlled the flow of grout into the hole. By connecting at this point, the gage indicated the grout pressure applied to the hole and not the line pressure.

In order to grout separately different depths of a hole, a grout packer was used. This packer consisted of three or four leather cup washers attached to the end of a piece of 1/2-inch pipe. This end of the pipe with the washers attached was inserted in the hole. As the grout escaped the end of the pipe and attempted to return up the hole, the washers were forced against the side of the hole and effected a seal.

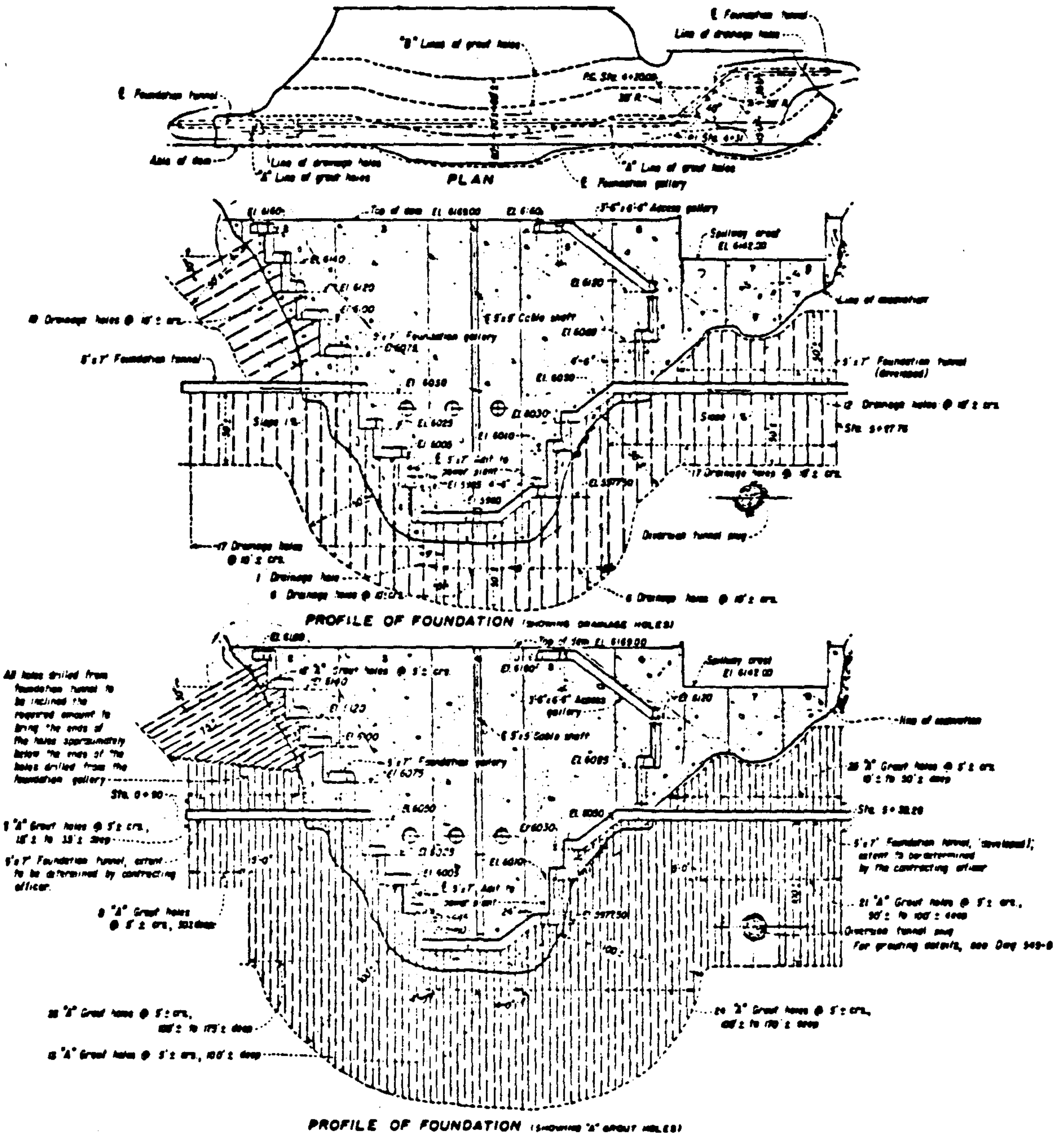


Figure 8. --Kortes Dam--Foundation grouting and drainage. (Sheet 1 of 2.) From drawing 545-D-95.

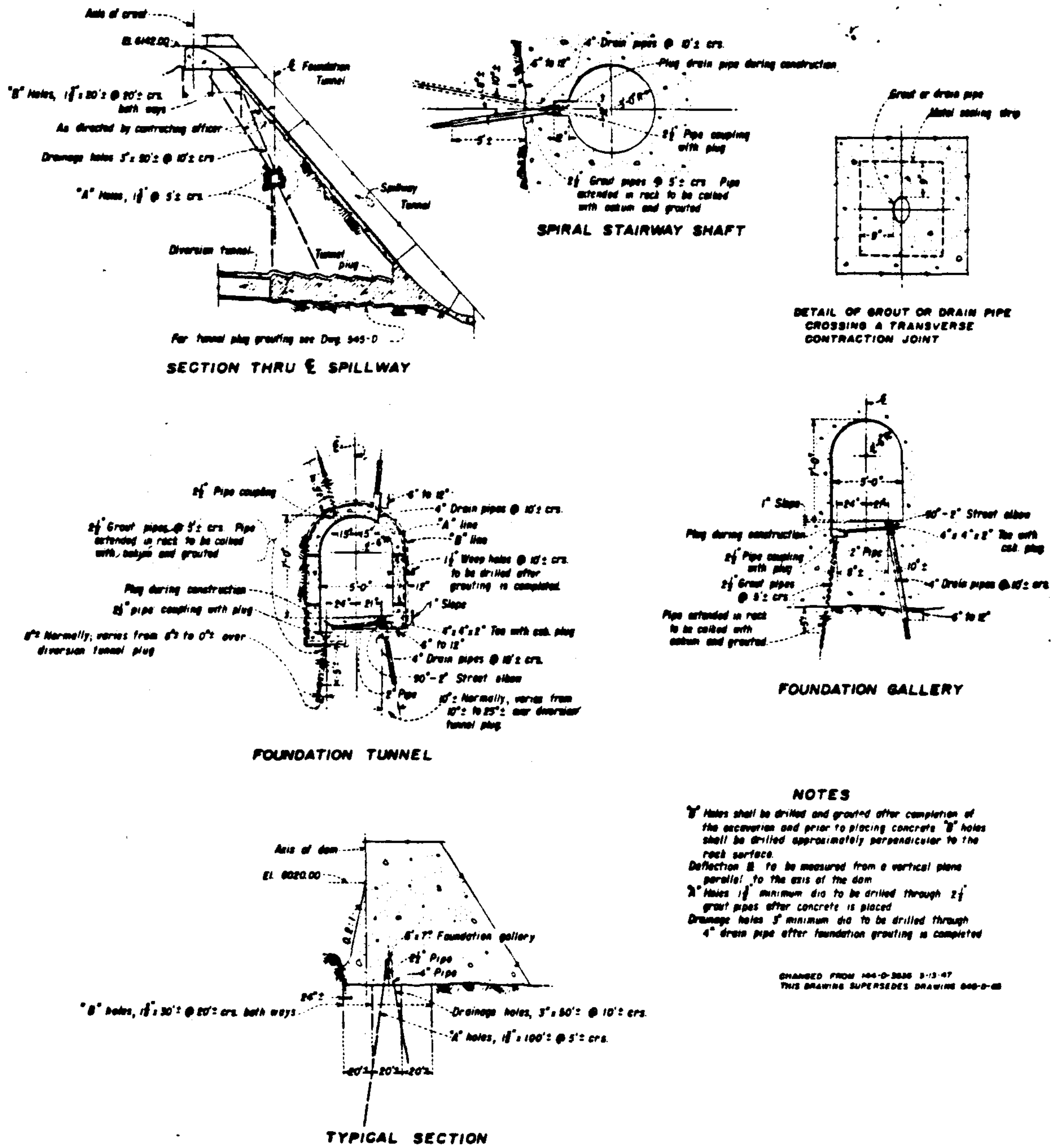


Figure 8. --Kortes Dam--Foundation grouting and drainage. (Sheet 2 of 2.) From drawing 545-D-95.

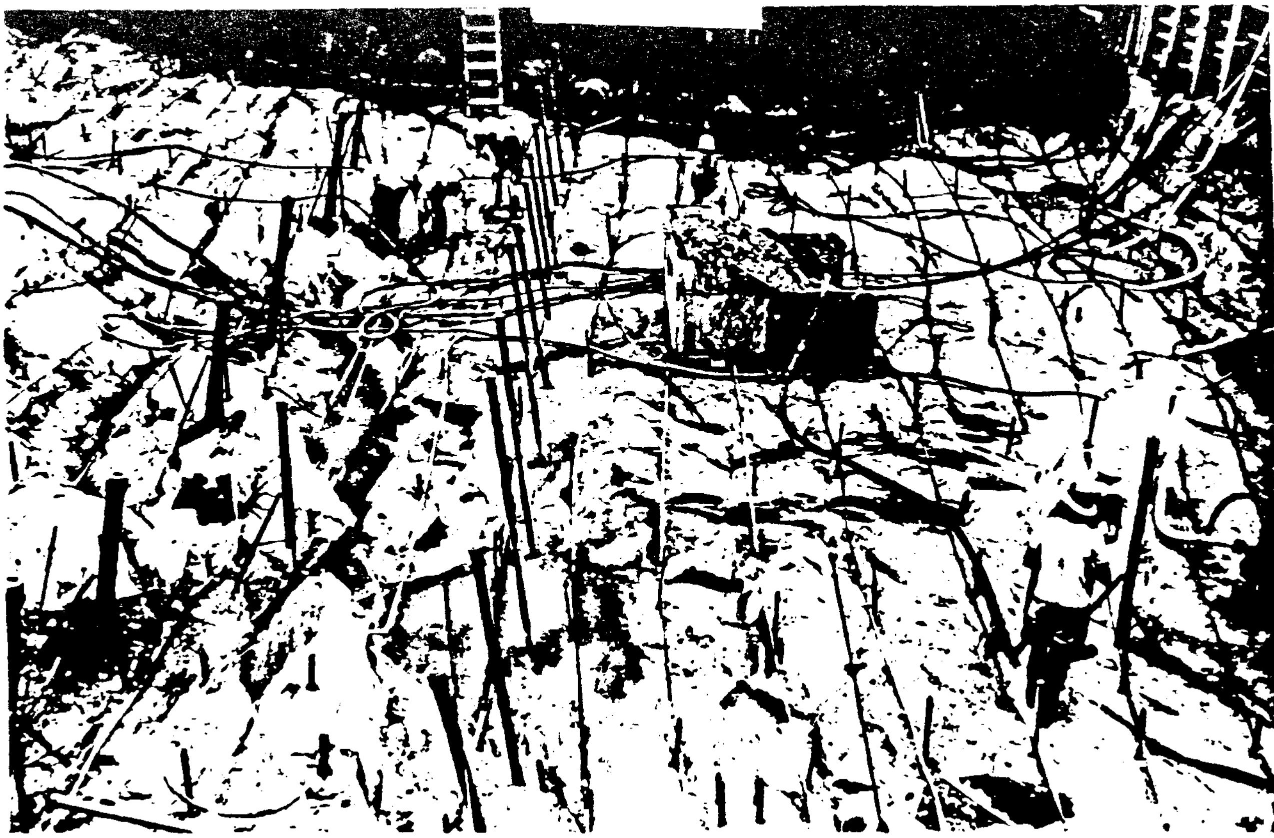


Figure 9. --Foundation rock at base of dam near the axis. View shows grout pipe, drain pipe, and uplift pressure pipe installation. The cooling pipe was placed on rock at 2-1/2-foot centers. 405-354, September 17, 1948.

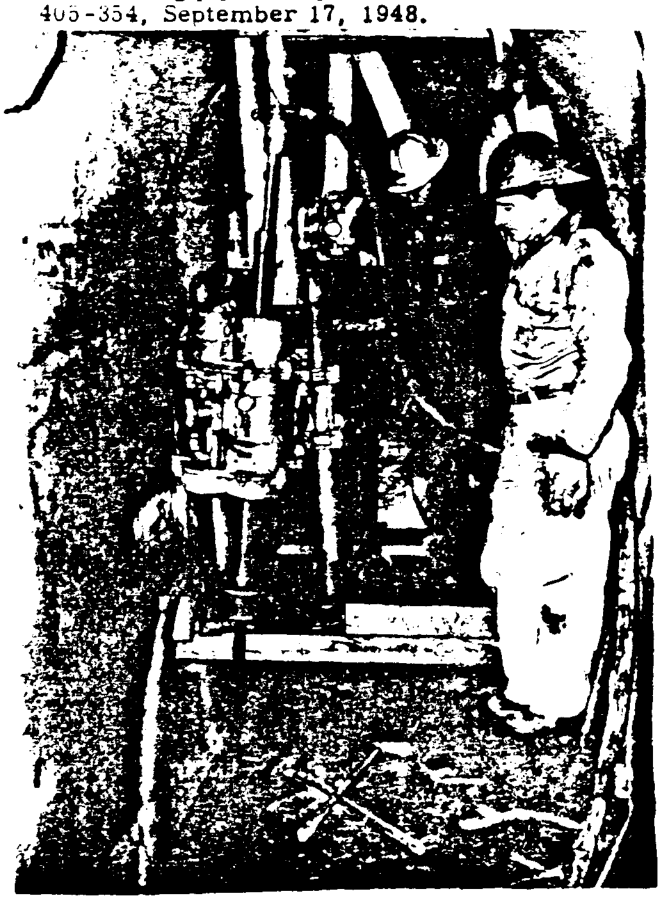


Figure 10. --Drilling curtain grout holes in gutter of the foundation gallery in the dam. 405-728.

14. Consolidation Grouting. - Because of the jointed and fractured conditions of the foundation bedrock and variation in the spacing of the joints, a predetermined plan for drill hole pattern was not possible. Inspectors on the site determined the number, location, angle, and dip of each hole. The results of the grouting operation helped to determine the extent that additional holes should be drilled and grouted. At the start of the grouting operation, the contractor grouted in pipe nipples at the collar of the hole to assist with the grout connection. After the procedure of grouting with the packer was developed, the use of nipples was abandoned and the final setting of the packer was made as close to the collar of the hole as possible. An EX-size core bit was used for drilling the holes.

When drilling of a hole was completed, a water test was made on the hole to determine the location of porous or fractured zones and to help formulate the grouting procedure. This test was made by setting the packer as close to the surface of the hole as possible and then pumping water into the hole at pressures ranging from 50 to 100 pounds per square inch. The water take of the hole was measured, and the packer was then moved downward in the hole and measurements of the water take were recorded at 3-foot intervals. The locations of water leaks were noted, and major leaks were calked before the grouting started. The results of the water test were used to determine the packer settings and the initial grout mixture.

Grout holes were usually grouted in three zones. Usually the bottom 15 feet of the hole was treated first, and the center and top zones followed in sequence. Thinner grout was injected under higher pressures in the lower zone. Thicker grout and lower pressures were used to seal the more fractured rock at the surface. Joints in the rock, where grout was escaping, were calked by driving wooden wedges into the joints. When it was impossible to seal the leaks in this manner, thick grout was injected intermittently into the hole until the leak sealed off.

(a) *Grouting Details.*--The portion of the dam foundation covered by blocks 6, 7, 8, and 9 was grouted first. Blocks 7, 8, and 9 comprised the spillway crest, and the concrete depth overlying the rock was only 18 feet. This rock was of poor quality and required extensive grouting to solidify it. The same grouting procedures were followed in the spillway area as in other parts of the dam foundation, except that the holes were more closely spaced. Some shallow holes were drilled and grouted to solidify the top rock surface. This was followed by deeper grouting. Leaks were numerous and difficult to calk satisfactorily. The rock underlying block 6 was of better quality and somewhat easier to grout. The average water-cement ratio of the grout mixture used in blocks 6 and 7, and in blocks 8 and 9 was 2.0 and 1.0, respectively.

The portion of the foundation rock covered by block 6 represented the center of the canyon under the riverbed. This rock was generally tight but was fractured or jointed by two joint patterns. In the toe area of the dam, the joints lie in almost a horizontal plane for the full width of the block. Near the downstream end of the grout blanket, the bedrock is broken by a series of joints that belong, for the most part, to the major joint system of the abutments. These joints strike across the river channel and dip 70° in a downstream direction. Grout holes were drilled at angles to intersect as many of the joints as possible. Some of the joints were filled with sand and clay and were washed out with water before grouting. Numerous surface leaks occurred during the grouting operations, and grouting had to be done intermittently to adequately seal the joints. The grout mixtures used in this area had an average water-cement ratio of 3.0.

The foundation rock for block 5 was the tightest encountered in the whole dam foundation. Deeper excavation had been done in this area to shape the foundation and to provide clearance for the elevator and access gallery. The grout mixtures used for this area had an average water-cement ratio of 3.0.

Because of the badly jointed condition of the rock in the left abutment, the grouting of the rock had to be done just ahead of the concrete placement for the dam, in order to take advantage of the concrete to provide support for the rock. Holes were drilled from the top of the concrete that was placed against the abutment. The holes dipped down into the abutment which permitted the injection of grout into the rock already covered by the concrete. The grouting operations continued until the grout raised and started to leak out of joints above the concrete. Additional concrete was then placed and more grout holes drilled on 10-foot vertical centers and grouted. The holes were also angled in an upstream and downstream direction in order to intersect as many joints as possible. The grout mixtures and pressures used varied considerably in this area. The average water-cement ratio of the grout was 2.0.

The condition of the bedrock underlying block 2 was similar to, but more open than, that underlying block 3 and the same grouting procedures were followed. On occasions, pipe nipples were embedded in the concrete and extended 5 feet to permit more concrete cover and thereby reduce the number of surface leaks. In general, the following pressures were used: 15- to 30-foot zone, 150 pounds per square inch; 7- to 15-foot zone, 100 pounds per square inch; and 0- to 7-foot zone, 75 pounds per square inch. Grouting pressures were reduced in the left abutment. During the grouting operations in this area, the reservoir was filled to elevation 6060 and a water leak through the left abutment developed between elevations 5994 and 6010. The first grout hole to connect with this leak took 1,112 sacks of cement before the hole was "lost." At the time the hole was lost, no appreciable effect on reducing the leak had been made. Another hole was purposely drilled to hit the leaking zone. To facilitate the grouting of this hole and minimize the cement losses, the reservoir was lowered so that the water no longer leaked through the abutment. There were 1,220 cubic feet of thick grout injected into this hole and considerable calking was done at the point where the grout was leaking out of the abutment. It was necessary to raise the reservoir before ample time had elapsed to allow for the grout to harden. The grouting had sealed the leak quite successfully but as small water leaks developed in this area, more and more cement was washed out and the leak again increased. No further effort was made to seal the leak with the consolidation grouting as it was planned to complete sealing the leak with the curtain grouting (see sec. 15).

A summary of the consolidation grouting data is shown below:

Block No.	Total number of holes	Total linear feet of holes	Total cubic feet of cement		Cubic feet of cement per hole	Cubic feet of cement per linear foot of hole
2	47	1,545	7,697		163.8	5.0
3	39	1,147	913		23.4	0.8
4	19	560	339		17.8	0.6
5	36	1,068	189		5.3	0.2
6	16	470	206		12.9	0.4
7	12	350	110		9.2	0.3
8	21	435	148		7.0	0.3
9	43	723	293		6.8	0.4
Totals	233	6,298	9,895	Averages	42.5	1.6

15. Curtain Grouting. - Specifications No. 1151 stipulated that the contraction joints in the dam be grouted before the curtain grouting. It was also specified that all required concrete should be placed within a radius of 300 feet to its full height before grouting the curtain. The narrow canyon and the height of the dam were such that this specifications requirement would have prevented grouting of the curtain until all concrete had been placed and joints grouted. This condition would have resulted in the contractor being substantially through with all other work on the contract except the grouting. The contractor therefore asked that he be allowed to proceed with the curtain grouting before the contraction joints were grouted. This request was granted with the understanding that grouting be stopped if leaks developed into the contraction joints. After the dam had been completed to elevation 6105, grouting of the cutoff curtain was begun during the winter of 1950 and completed during the following summer months. No trouble was encountered with grout leaking from the drill holes into the contraction joints. Water was circulated through the grout system of all contraction joints adjacent to the area being grouted.

Dial gages were installed at contraction joints in the foundation galleries as a means of detecting uplift or movement of blocks.

(a) *Grouting Plan.*-- For convenience of grouting and to best fit the grouting program into the overall construction schedule, the grout curtain was divided into five sections each of which was grouted separately in the following order:

Section 1. Left abutment below the foundation tunnel and the bottom of the canyon.

Section 2. Right abutment below the right foundation tunnel.

Section 3. Area above and below the left foundation tunnel.

Section 4. Left abutment above the left foundation tunnel.

Section 5. Area above and below the right foundation tunnel.

The general procedure for drilling and grouting was to drill and grout the two holes forming the outside boundary of an area and then follow by grouting the hole in the center of the area. Intermediate holes were then drilled and grouted. Other holes were drilled and grouted as necessary in the centers of the spaces between the already grouted holes, until the entire area was grouted. Packers were used to grout every hole, and the spacing of the packer was determined from results of the water test. Unless there was some reason to set the packers at some other locations, the packers were usually set at 30-foot intervals.

(b) *Grouting Pressures.*--The maximum pressures used for the injection of grout were estimated by the following rules:

(1) One pound per square inch for each vertical foot of concrete superimposed above the hole being grouted.

(2) Two pounds per square inch for each linear foot of rock, as measured between the packer setting and nearest point of rock-concrete contact.

(3) Fifty pounds per square inch added because of the blanket grouting.

(4) Pressure was increased approximately 12 percent for each series of intermediate holes that were grouted.

The above rules were used only generally and pressures were varied as conditions of grouting dictated.

(c) *Grouting Details.*--Following the drilling of any hole or immediately before grouting, the hole was tested with water, as described under the procedures of the consolidation grouting (sec. 14). A very thin grout was used to start all of the holes and the mixture was thickened as the grouting of the hole progressed and the take of the hole dictated. Generally, the initial grout mixture had a water-cement ratio of 8.0 or 5.0.

The first hole grouted in the section 1 area took 117.7 sacks of cement, which was the next to the largest grout take of any hole in the group. Grout from this hole undoubtedly grouted a large portion of this area because a good spread was noted by a contact with another hole some 80 feet away. Drill water was lost during the drilling operation of one hole, and the drilling was stopped before the final depth was reached and the hole grouted. This hole took 423.2 sacks of cement. Subsequent holes were drilled and grouted in this area until all holes were treated on 10-foot centers. One additional hole was then drilled and grouted in the area that had received the most grout to test the effectiveness of the grouting, and to determine if drilling and grouting of holes on 5-foot centers would be justified. This hole accepted only 0.3 of a sack of cement; as a result no other holes were drilled on 5-foot centers.

Only three holes were drilled in the section 2 area. The rock was relatively tight, as evidenced by the small grout take, 21.5 sacks of cement for the three holes.

During grouting operations in the section 3 area, the reservoir was filled to an average elevation of 6060. A substantial quantity of water leaked through seams in the left abutment. None of the holes drilled made a connection with the leaking zone. This portion of the foundation was badly fractured with many open seams, and took considerable quantities of grout. The first hole grouted took a total of 2,912.2 sacks of cement. Three holes made water during and after drilling. Except for a few small grout leaks through the cracks in the concrete lining of the left foundation tunnel, there were no leaks found while grouting this group of holes. Two holes were drilled through the diabase dike from the end of the tunnel, one horizontally and the other upward at a 45° angle. The latter hole was grouted first and took 894 sacks of cement on the far side and 3 sacks on the channel side of the seam. The holes were drilled on 5-foot centers.

Since there was a known leak through the abutment area included in section 4 area of the grout curtain, the normal procedure of sequence grouting was not followed and holes were drilled in an effort to contact the leaking zone. During these operations, the level of the reservoir was held below elevation 6085 at which elevation water leaked through the abutment. After several holes were drilled, one hole was drilled that made a good connection with the leaking area. This hole accepted a total of 2,670 sacks of cement. Grout leaks from this hole occurred in all the cracks where water had previously flowed when the reservoir was above elevation 6085. Considerable calking was required in order to control the leaks. Wooden wedges and lead wool were used to effect good calking.

A second hole was drilled 10 feet below and parallel to the first hole and accepted a total of 1,895 sacks of cement. Leaks from this hole also occurred in the abutment downstream from the dam but at a higher elevation and in less volume than the previous hole. The grout mixture was progressively thickened to control the leaks. Two adjacent holes were then drilled, penetrating this same area, but these holes took very little grout. This indicated the seam or seams that carried the water through this area were probably well filled with grout. The remaining holes in this area were drilled and grouted, and these also took very little grout. The entire area was grouted before the level of the reservoir was raised above elevation 6085. When the reservoir was filled, only a few wet spots occurred in the area that had leaked badly before the grouting.

The section 5 area was grouted last. A good spread of grout was obtained during treatment of two holes, each of which connected with one or more other holes in the area. One of these holes made water after drilling, indicating a possible connection to the reservoir. The grout take for one of these holes was 180 sacks of cement and that of the other was 163 sacks. The amount of grout injected into the remaining holes was not significant, hence drilling of 5-foot spaced holes was not warranted.

The following tabulation summarizes the results of the cutoff curtain grouting:

Block No.	Total number of holes	Total linear feet of holes	Total cubic feet of cement		Cubic feet of cement per hole	Cubic feet of cement per linear foot of hole
2	57	4,101.7	12,250.5		214.9	3.0
3	9	1,148.5	112.0		12.4	0.1
4	6	611.1	560.5		93.4	0.9
5	7	1,009.1	72.5		10.4	0.1
6	5	833.1	316.0		63.2	0.4
7	9	609.4	175.5		19.5	0.3
8	8	514.3	197.5		24.7	0.4
9	9	704.4	237.0		26.3	0.3
Totals	110	9,531.6	13,921.5	Averages	126.6	1.5

16. Diversion Tunnel Grouting. - Grouting operations in the diversion tunnel consisted of grouting the foundation rock surrounding the concrete plug, grouting contraction joints between plug sections, and grouting the contact zone of concrete and rock. Because of the necessary coordination of some phases of the grouting operations with the concrete placing operations for diversion plug sections, grouting operations are discussed with the placement of the diversion plug (see sec. 151).

17. Spillway Tunnel Grouting. - The grouting program for the spillway tunnel consisted of drilling a series of low-pressure and high-pressure radial holes at regular intervals along the length of the tunnel. Three low-pressure holes were drilled in the arch portion of the tunnel at each 20-foot station. These holes extended only through the concrete lining and were for the purpose of filling any unfilled portion of the concrete lining and to grout near-surface rock fractures, so that the high pressures applied to the

other holes would not be exerted on the concrete lining. Six high-pressure holes were drilled and grouted also at each 20-foot station. These holes extended 20 feet into the surrounding rock. Pipe was installed and embedded in concrete in the sections of the tunnel that were reinforced. Holes were drilled through the concrete in the nonreinforced sections of the tunnel. All except a few holes were drilled with jackhammers. Packers were used to seal the top of the hole during the grouting operation.

Backfill grouting was accomplished under low pressures of 30 pounds per square inch. Pressures up to 200 pounds per square inch were used to grout the high-pressure holes. In general, the rock was very tight and the holes did not take any great quantity of grout.

The grout mixture was varied as the take of the hole varied. In general, the high-pressure holes were grouted with a mix having a water-cement ratio of 5.0. The water-cement ratio for the backfill grout ranged between 3.0 and 1.0. Grouting of the inclined portion of the tunnel was performed from platforms built on the concrete form framework prior to form stripping operations. The horizontal section of the tunnel was grouted from jumbo platforms that were moved along the tunnel invert.

The grouting operations for the entire tunnel required a total of 924 sacks of cement, 462 sacks for the low-pressure holes and 462 sacks for the high-pressure holes. For the low-pressure grouting, 62 sacks were used for the inclined portion of the tunnel and 400 sacks were used for the horizontal portion of the tunnel. The average take was 1.2 sacks of cement per linear foot of tunnel.

18. Grout Backfill--Foundation Grouting and Drainage Tunnels. - The concrete and rock contact zone around the foundation grouting and drainage tunnels in the abutments were grouted by drilling holes through the linings and injecting grout under low pressures. Holes were drilled just above the springline for the injection of the grout, and vent holes were drilled in the arch at points of large overbreak and at other locations. The entire tunnel was grouted in one operation and grout connected with all vent holes. To backfill the right tunnel, 141 sacks of cement were used, which represented approximately 1 sack per foot of tunnel length. This cement was mixed in a relatively thick mixture having a water-cement ratio of 1.0. A total of 1,409 sacks of cement were pumped into the top of the left foundation tunnel between the rock and concrete lining. There were several large open seams in the rock surrounding this tunnel, and it was obvious that a large portion of this grout was being used to grout the rock abutment.

19. Grout Backfill--Foundation Exploratory Tunnels. - Concrete settlement and void grouting of both the left and right foundation exploratory tunnels was accomplished through a system of grout pipe installed in the arch of the tunnel before the concrete was placed. This system consisted of a supply and return header with outlets at high points in the tunnel arch. Separate vent returns were also installed from all high points. The grouting was performed after the portal of the tunnels had been covered with the dam concrete. A thick grout with a water-cement ratio of 1.0 was used. A total of 170 and 282 sacks of cement were injected in the left and right tunnels, respectively. These tunnels were 70 and 100 feet long, respectively, and the grout take represented 2.4 and 2.8 sacks per foot length of tunnel.

20. Foundation Drainage. - Foundation drainage for the dam was provided by a series of holes drilled 50 feet into the foundation rock at approximately 10-foot centers through pipes 4 inches in diameter placed in the roof and floor of the foundation tunnel, and in the floor of the grouting and drainage gallery (fig. 8). All holes were drilled with 3-inch-diameter diamond core drills. The holes were drilled vertically into the rock on a line parallel with and close to the dam axis on the downstream side. Drainage holes were drilled after the cutoff curtain grouting had been completed. In 1952, the total flow from the 72 drain holes was 4.3 gallons per minute. The drainage from one hole was measured at 1.6 gallons per minute. The rate of flow from 19 other holes ranged between 0.01 and 0.4 gallons per minute.

21. Uplift Pressure Holes. - In order to measure and record the static uplift pressure exerted on the base of the dam, a system of drain holes was drilled through the concrete and rock contact, and the drainage piped to the foundation gallery. These pipes were then closed and a pressure gage installed on the end of the pipe. Four lines of holes were drilled at axis stations shown on figure 11. Holes were drilled to indicate the pressure upstream from the grout curtain, immediately downstream from the grout curtain,

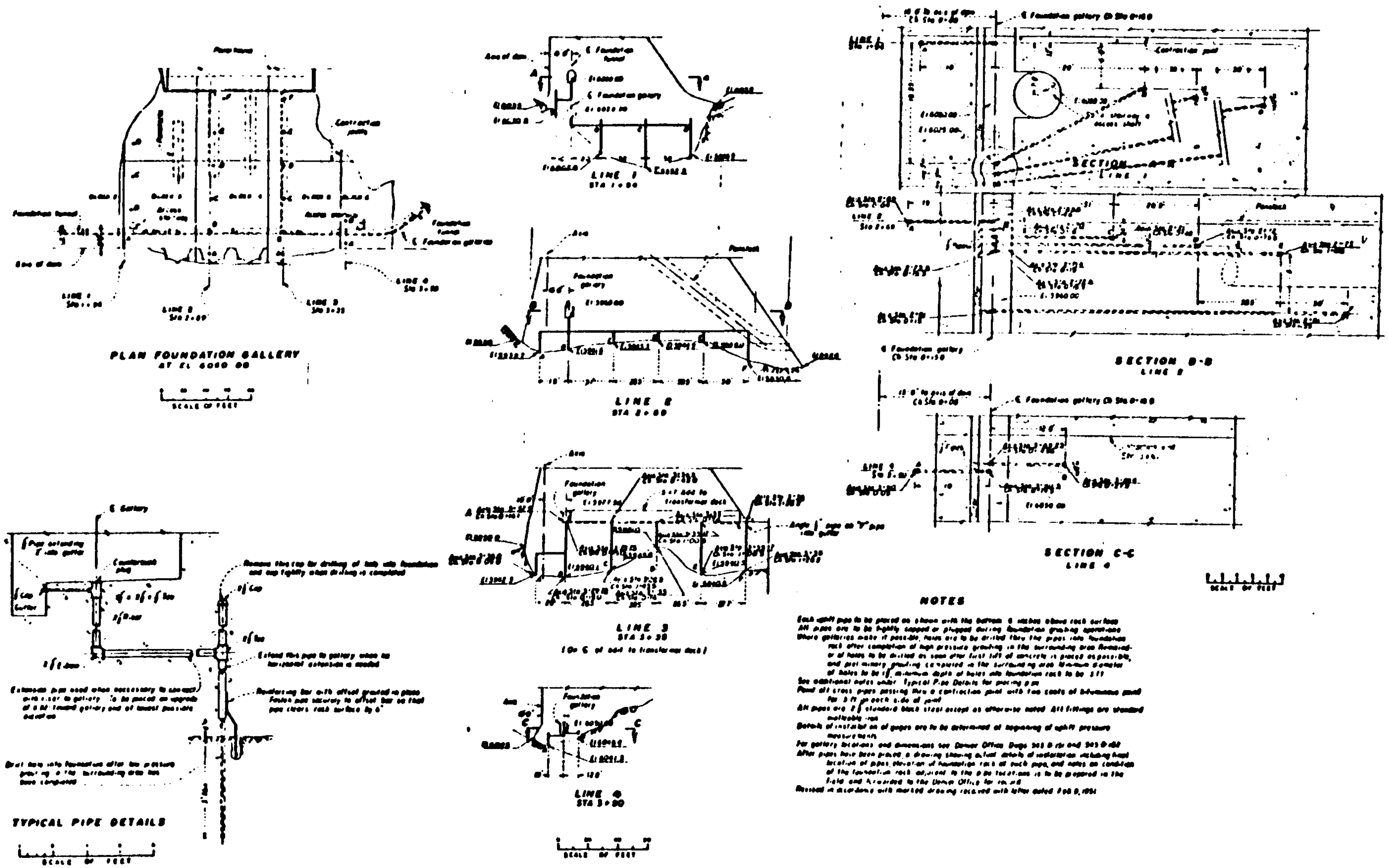


Figure 11. --Foundation uplift pressure pipes in dam. From drawing 545-D-250.

and at approximately 20-foot intervals extending downstream. Pipe casings were embedded in the concrete of the dam with the end of the pipe 6 inches above the rock. When possible, the hole was drilled 3 feet into the rock after the curtain grouting had been completed. When this was not possible the hole was drilled before grouting and the pipe extended to the gallery. During the curtain grouting there was no grout observed coming from any of the uplift holes. The reservoir was filled initially during the summer of 1950. Following this filling, some of the uplift pipes were observed to be flowing a small amount of water. Pressure gages were installed during December 1950 and the first complete readings of the pressure gages were taken during January 1951. Unexpected relatively high pressures were measured. The pressure on the base of the dam in the center of the channel was approximately equal to two-thirds the reservoir head at the heel and zero at the toe. Studies indicated the need for additional grouting work and the drilling of drainage holes. Accordingly, additional grouting was performed, and drain holes were drilled from the foundation gallery at elevation 5950 into the abutments to provide an escape for water that may be getting past the grout curtain and to relieve the resulting underpressures. Flows from each of the holes was small (not over 2-1/2 gallons per minute).

APPENDIX B

DOCUMENTS REVIEWED

1. Technical Data Workbook, Kortes Dam, prepared by Jack M. Hogan, Converse Ward Davis Dixon, Inc., October 1980.

2. Draft SEED Report, Kortes Dam, July 21, 1981.

3. Geology Report, Kortes Dam and Powerhouse Foundation, by R. D. Dirmeyer, Jr., Geologist, Casper, Wyoming, February 1952.

4. Monthly Foundation Treatment Reports, April 1949 through October 1950.

5. Field trip report dated March 30, 1950 from O. E. Boggess to Chief Engineer, subject: "Foundation and diversion tunnel grouting -- Specifications No. 1151 -- Kortes Dam".

6. "Foundation and Materials Investigation", undated.

7. Specifications No. 1151, Kortes Dam, "Drainage" and "Pressure Grouting", pp. 35-48.

8. Geology Report G-63, "Geologic Examination, Kortes Dam Site", October 2, 1945.

9. Logs of drill holes 1 through 30.

10. Geology Report No. G-50 (Supplement No. G-39), "Geologic examination, Kortes Dam Site", June 23, 1945.

11. Geology Report No. G-39, "Geology field examination of Kortes Dam site", February 5, 1945.

12. Geology Report No. G-86, "Geologic Examination, Kortes Damsite", October 1, 1946.

13. Letter dated February 27, 1952 from Chief, Dams Branch, to District Manager, Casper, Wyoming, subject: "Report of uplift pressures and embedded instruments, Kortes Dam".

14. Letter dated February 29, 1952 from District Manager to Chief Engineer, Denver, Colorado, subject: "Report of drain drilling, Kortes Dam".

15. Letter dated April 7, 1952 from Acting District Manager to Chief Engineer, subject: "Transmittal of Uplift Pressures Readings and Resistance Thermometer Readings, Kortes Dam".

16. Field trip report dated April 28, 1952 from O. E. Boggess, Engineer, to Chief Engineer, subject: "Uplift pressures at Kortes Dam".

17. Letter dated September 20, 1954, from District Manager to Assistant Commissioner and Chief Engineer, subject: "Trend of increasing pressure on dam foundations, Kortes Dam".

18. Letter dated June 3, 1954 from Acting District Manager to Assistant Commissioner and Chief Engineer, subject: "Trend of Increasing Pressure on Dam Foundation, Kortes Dam".

19. Letter dated May 4, 1954 from Chief Designing Engineer to District manager, Casper, Wyoming, subject: Trend of increasing pressures on dam foundation, Kortes Dam".

20. Drain flow records, 1980-1981.

21. Uplift pressure plots, lines 1 through 4, 1951-1981.

22. Computer print outs of uplift pressure readings.

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