GR-86-5

ANALYSIS OF UTILIZATION OF GROUT AND GROUT CURTAINS — FLAMING GORGE DAM

February 1986
Engineering and Research Center

U.S. Department of the Interior
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16. ABSTRACT

The foundation grouting program at Flaming Gorge 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– FLAMING GORGE DAM

by

Claude A. Fetzer

Prepared Under Contract No. 2-07-DV-00148

Concrete and Structural Branch
Division of Research and Laboratory Services
Engineering and Research Center
Denver, Colorado

February 1986

UNITED STATES DEPARTMENT OF THE INTERIOR ★ BUREAU OF RECLAMATION

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This report was published in the GR series from a copy of the report provided by the Contractor, thus the quality of photographs and figures may be less than usually acceptable.

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ANALYSIS OF UTILIZATION OF GROUT AND GROUT CURTAINS

FLAMING GORGE DAM

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ANALYSIS OF UTILIZATION OF GROUT AND GROUT CURTAINS

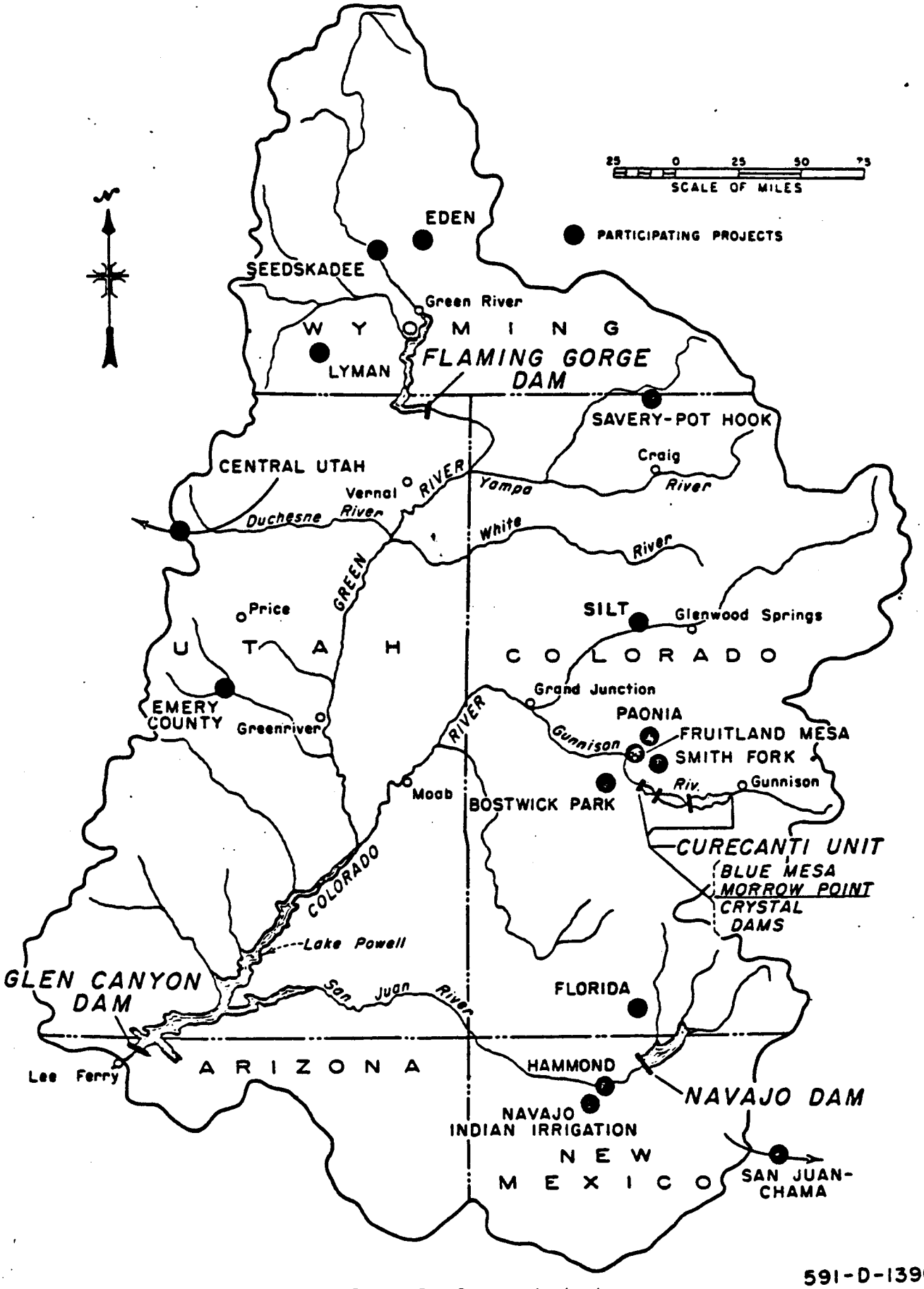
FLAMING GORGE DAM

I. INTRODUCTION

1. General description. Flaming Gorge Dam and Powerplant are located on the Green River in northeastern Utah, about 6 miles south of the Utah-Wyoming state line as shown on figure 1. The dam is a concrete thin arch and has a maximum structural height of 502 feet above the lowest point in the foundation (El. 5,545 feet). A photograph of the completed dam is shown on figure 2, and the plan, elevation and sections are presented on figure 3. The structure has a maximum base width of 131 feet and a crest width of 27 feet. The crest length is 1,285 feet and the crest elevation is 6,047 feet. The total capacity of the reservoir at the top of active conservation level, El. 6,040 feet, is 3,788,900 ac. ft.

Normal releases are made through the powerplant and through the outlet works. The outlet works consist of two 72-inch steel pipes through the dam, reducing to 66 inches at the toe of the dam and continuing downstream to a valve structure.

The spillway has an inlet structure at the left end of the dam which discharges into a concrete-lined tunnel through the left abutment. The 675-foot-long tunnel varies in diameter from 26 feet at the upstream portal to 18 feet at its downstream portal. Spillway flows are controlled by two 16.75 by 34.00 ft. fixed-



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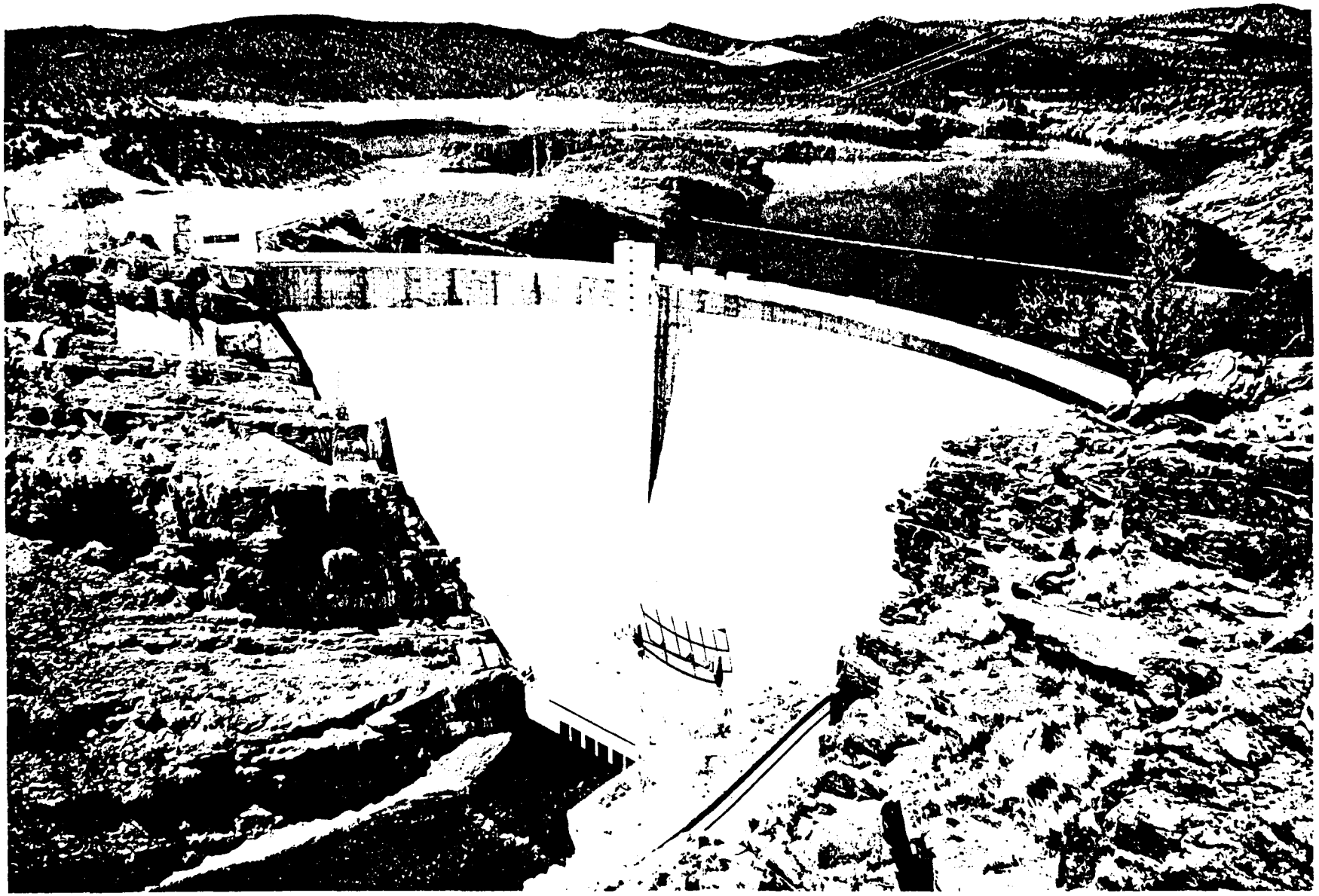
Colorado River Storage project location map.

FIGURE 1



P591-421-5225A FLAMING GORGE UNIT - COLORADO RIVER STORAGE PROJECT - UTAH-WYOMING
Flaming Gorge Dam and Powerplant - Looking upstream. Visitors Center and parking area are in upper left.
Water depth is approximately 395 feet.
5-14-64 Bureau of Reclamation photo by F. B. Slote

Figure 2

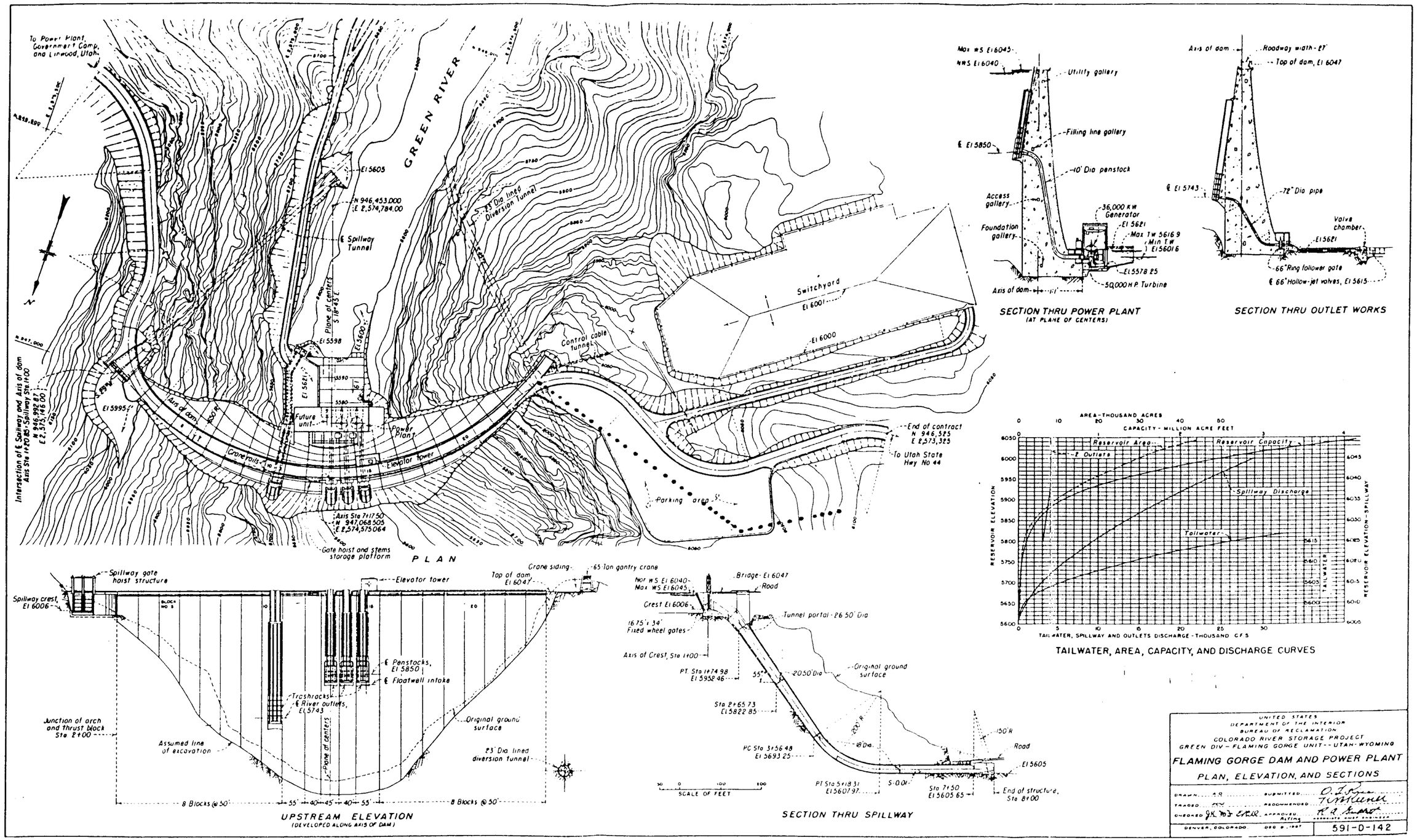
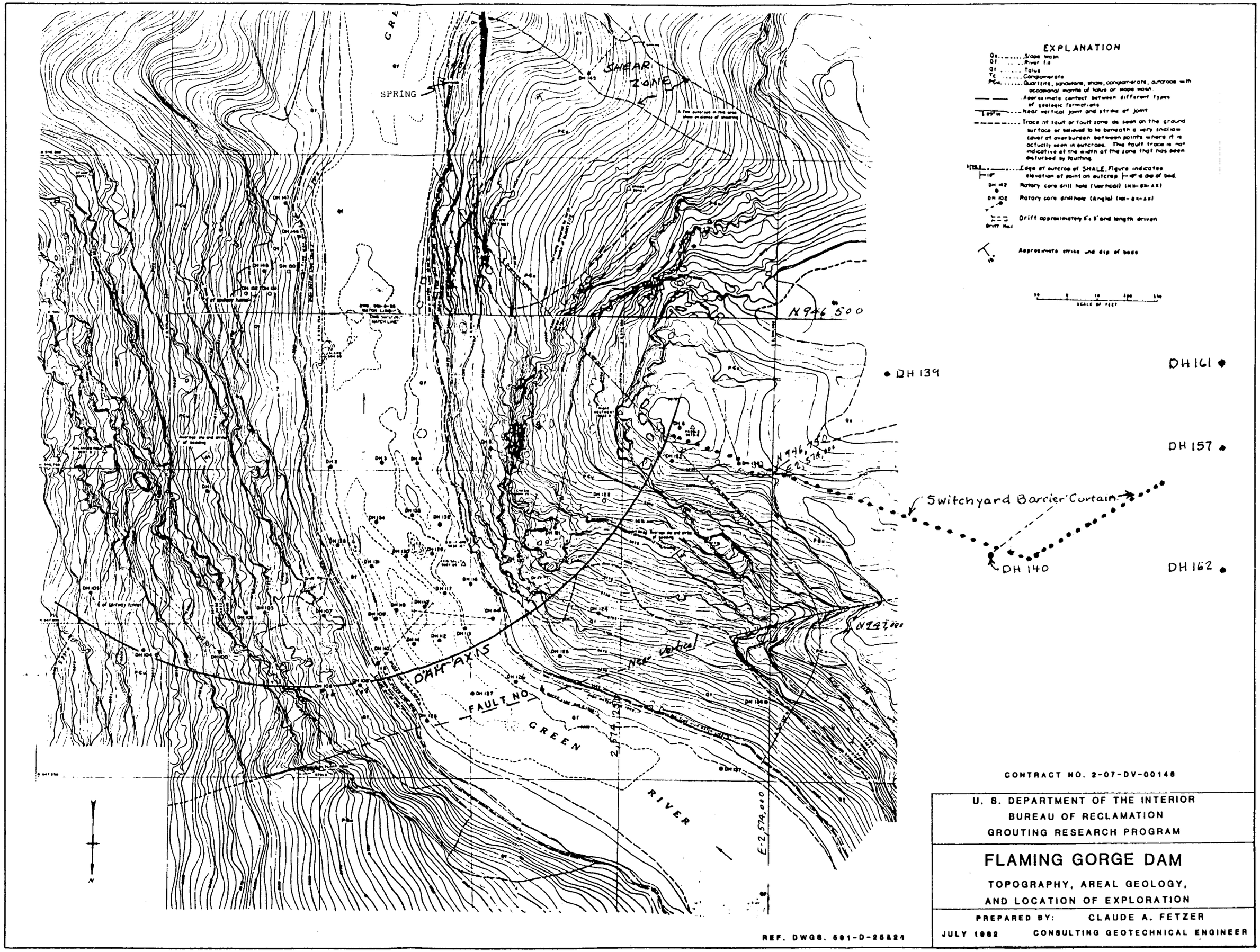

To Power Plant, Government Camp, and Linwood, Utah
GREEN RIVER
El 5605
N 946,453.000
E 2,574,784.00
℄ Spillway Tunnel
23' Dia lined Diversion Tunnel
Plane of centers S 18°45' E
El 5598
El 5600
El 5621
El 5995
Axis of dam
450' R
Future unit
Power Plant
Crane rails
Elevator tower
Axis Sta 7+17.50
N 947,068.505
E 2,574,575.064
Gate hoist and stems storage platform
Control cable tunnel
Switchyard
El 6001
El 6000
End of contract
N 946,525
E 2,573,325
To Utah State Hwy No 44
Parking area
PLAN
Max WS El 6045
NWS El 6040
Utility gallery
Filling line gallery
℄ El 5850
10' Dia penstock
Access gallery
Foundation gallery
36,000 KW Generator
El 5621
Max Tw 5616.9
Min TW El 5601.6
El 5578.25
50,000 H.P. Turbine
Axis of dam
SECTION THRU POWER PLANT
(AT PLANE OF CENTERS)
Axis of dam
Roadway width - 27'
Top of dam, El 6047
℄ El 5743
72" Dia pipe
Valve chamber
El 5621
66" Ring follower gate
℄ 66" Hollow-jet valves, El 5615
SECTION THRU OUTLET WORKS
AREA - THOUSAND ACRES
CAPACITY - MILLION ACRE FEET
Reservoir Area
Reservoir Capacity
2 Outlets
Spillway Discharge
Tailwater
RESERVOIR ELEVATION
TAILWATER
RESERVOIR ELEVATION - SPILLWAY
TAILWATER, SPILLWAY AND OUTLETS DISCHARGE - THOUSAND CFS
TAILWATER, AREA, CAPACITY, AND DISCHARGE CURVES
Spillway gate hoist structure
Spillway crest El 6006
Elevator tower
Top of dam El 6047
Crane siding
65 Ton gantry crane
℄ Penstocks, El 5850
℄ Floatwell intake
Trashracks
℄ River outlets, El 5743
Junction of arch and thrust block Sta 2+00
Assumed line of excavation
Original ground surface
23' Dia lined diversion tunnel
Plane of centers
8 Blocks @ 50'
UPSTREAM ELEVATION
(DEVELOPED ALONG AXIS OF DAM)
Nor WS El 6040
Max WS El 6045
Bridge - El 6047
Road
Crest El 6006
Tunnel portal - 26.50' Dia
16.75' x 34' Fixed wheel gates
Axis of Crest, Sta 1+00
PT Sta 1+74.98
El 5952.46
20.50' Dia
Original ground surface
Sta 2+65.73
El 5822.85
200' R
18' Dia
PC Sta 3+56.48
El 5693.25
150' R
Road
El 5605
PT Sta 5+18.31
El 5607.97
S 0.01
Sta 7+50
El 5605.65
End of structure, Sta 8+00
SCALE OF FEET
SECTION THRU SPILLWAY
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
GREEN DIV - FLAMING GORGE UNIT - UTAH - WYOMING
FLAMING GORGE DAM AND POWER PLANT
PLAN, ELEVATION, AND SECTIONS
DRAWN
SUBMITTED
TRACED
RECOMMENDED
CHECKED
APPROVED
ACTING
ASSOCIATE CHIEF ENGINEER
DENVER, COLORADO
591-D-142

FIGURE 3



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

FLAMING GORGE DAM

TOPOGRAPHY, AREAL GEOLOGY, AND LOCATION OF EXPLORATION

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

FIGURE 4

wheel gates. The crest elevation of the concrete inlet is 6,006 feet, and the elevation at the top of the gates is 6,040 feet.

The contract for the dam was awarded to Arch Dam Constructors on June 18, 1958, and all work under the contract was completed on January 10, 1964. Drilling and grouting were performed under a subcontract by Selby Drilling Corp.

During construction, the work area was protected by upstream and downstream earthen cofferdams. The river flow was diverted through a 23-foot-diameter concrete lined tunnel through the right abutment ridge. After all requirements for diversion were completed, the tunnel was plugged and abandoned, except the downstream reach was later used for abutment drainage.

The reservoir is normally operated near the top of the active conservation pool; hence, the dam is subjected to near maximum head at all times.

2. Consultants. A board of consultants was engaged on the project during the construction stage. (The available records do not make any reference to the board meeting prior to construction.) The board prepared reports based on site inspections of October 6-7, 1958, June 6-8, 1960 and October 12-16, 1961. The board had five prominent members and consisted of the following:

Julian Hinds, Chairman
Raymond E. Davis
John J. Hammond
John W. Vanderwilt
Edward B. Burwell, Jr.

Sections of the available consultants' reports concerning grouting and foundation treatment have been reproduced in appendix A.

II. GEOLOGY

3. References. Chapter II, "Site Investigations and Geology", from the Technical Record of Design and Construction, has been reproduced in appendix B.

4. Site investigations. The topography, aerial geology and location of exploration at the site are shown on figure 4. As investigations were made for different types of dams with different axis than the final arch-dam axis, the drill holes are scattered throughout the site area and only a limited number are located along the final axis. Drift No. 1 in the right abutment with a length of 27.8 feet is about 25 feet downstream from the axis. Drift No. 2 in the left abutment with a length of 15.5 feet is 150 feet downstream of the axis. The photograph on figure 5 was made on July 16, 1959 before excavation of the right abutment had started. It shows the steepness of the slope, the blocky nature of the rock, and the narrowness of the point forming the right abutment. Drift No. 1 cannot be identified in the photograph.

In the preconstruction exploratory drill holes, high water takes were recorded in many of the percolation tests (water-pressure tests). The water takes often exceeded the limited pump capacity at the maximum pressure. The tests were generally conducted in three pressure steps of 25, 75 and 150 psi for each 5-foot to 10-foot interval of hole with pressure maintained for 5 minutes. Several tests were not performed due to problems relating to seating and holding the packer. Partial and/or total

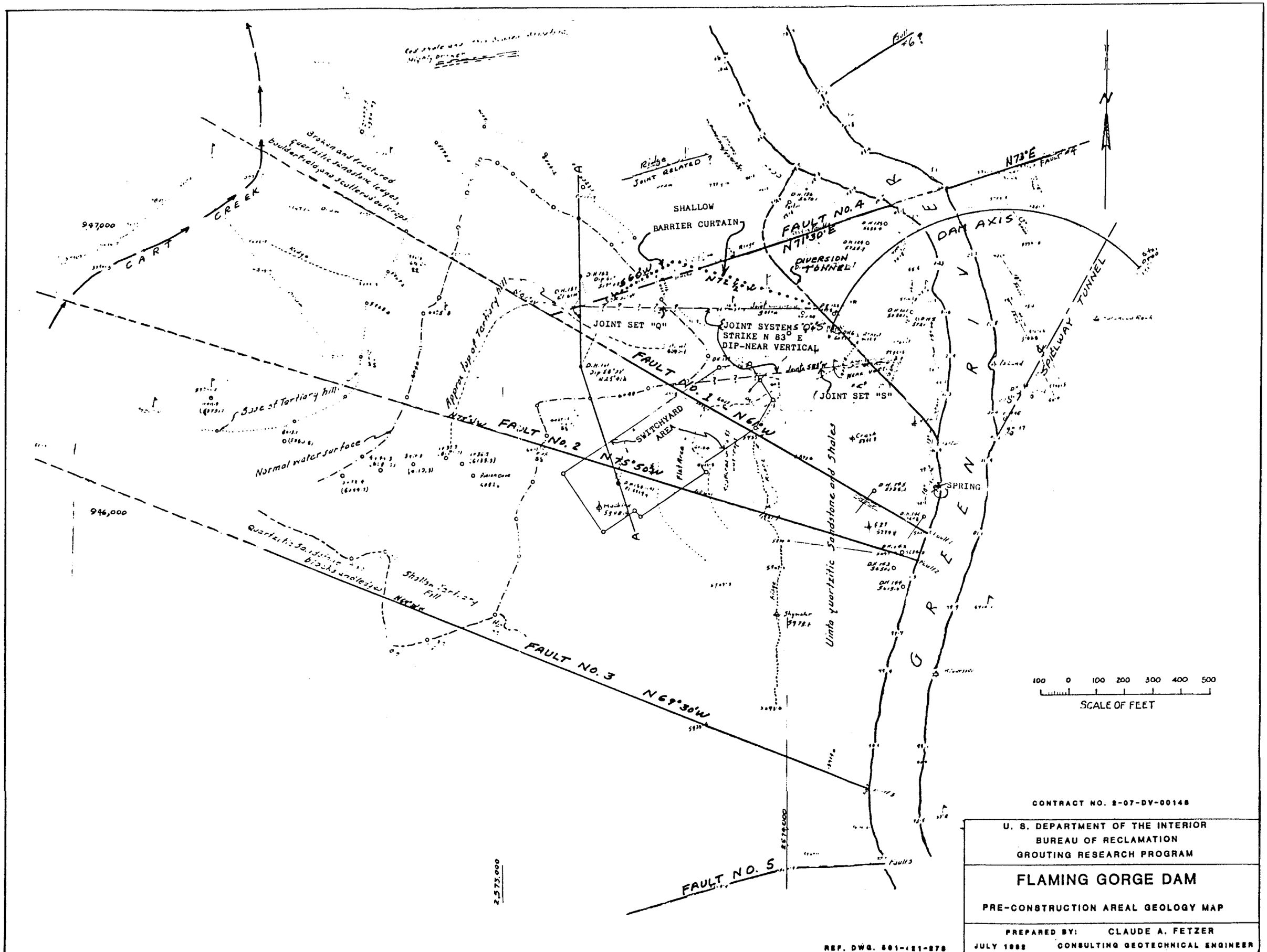
Flaming Gorge Dam - Right Abutment before construction 7/16/59

Figure 5

drill water losses were experienced in many holes. The water losses in the abutments were primarily related to relief joints; and the losses in the valley bottom were related to shattered rock zones. Highly fractured sandstone with vertical joints was found in many of the valley bottom holes including DH 107, DH 108, DH 115, DH 116, DH 126, DH 127 and DH 128. Most holes in the abutments were dry holes as the water table was evidently near streambed level.

5. Geology. The preconstruction aerial geology map for an extended area around the site is presented on figure 6. This map indicates that while the dam foundation and abutments are free of faults, there are three well developed faults or shear zones located between Cart Creek and Green River downstream from the dam axis. Another fault, No. 4, is located about 82 feet upstream from the face of the dam and continues across Green River. Fault No. 4 was encountered in the diversion tunnel and appears to connect with fault No. 1. Figure 6 also shows the location, direction and dip of the joint sets.

The foundation of the dam was mapped during construction, and the resulting foundation geology map is presented on figure 7. The shale beds encountered at the final foundation levels are highlighted.



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FLAMING GORGE DAM

PRE-CONSTRUCTION AREAL GEOLOGY MAP

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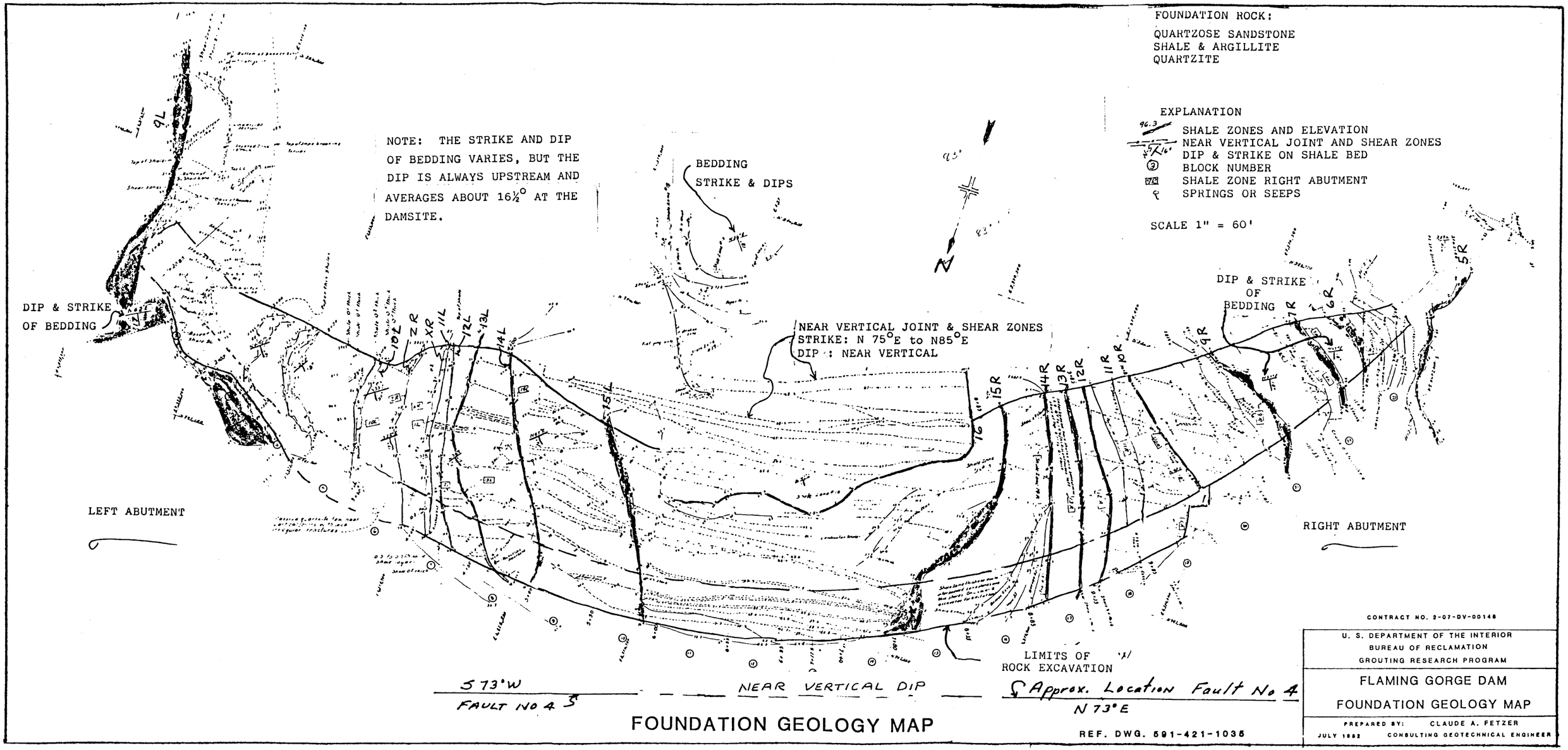
REF. DWG. 801-421-278

FIGURE 6



FOUNDATION GEOLOGY MAP

REF. DWG. 591-421-1035

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FLAMING GORGE DAM
FOUNDATION GEOLOGY MAP

PREPARED BY: CLAUDE A. FETZER
JULY 1962 CONSULTING GEOTECHNICAL ENGINEER

FIGURE 7

III. FOUNDATION TREATMENT

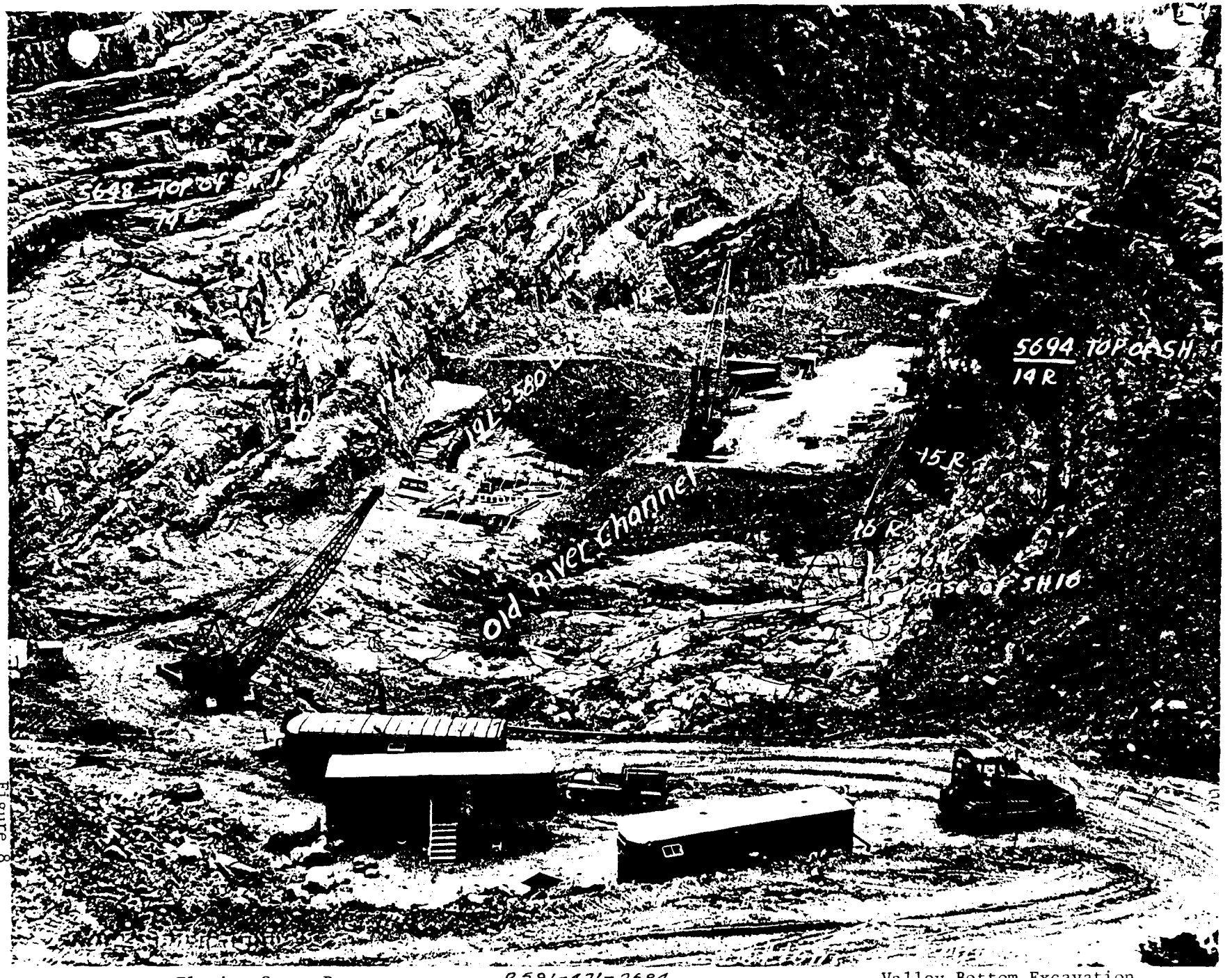
6. References. Chapter III, "Foundation Treatment", from the Technical Record of Design and Construction has been reproduced as appendix C. Chapter III also presents a brief description of the grouting and drainage.

7. Keyway excavation. Photographs of the keyway excavations are shown in figures 4 and 5 of appendix C and large-scale photographs are shown on figures 8 through 11 of this report. The keyway excavation on both abutments extended over a long period of time. Numerous setbacks from the design excavation line were made before obtaining a foundation satisfactory to the Bureau and its consultants. Excavation of the left abutment keyway was started in April 1959, and excavation of the right abutment was started in September 1959. The board of consultants examined the excavations in detail on its site visits, and in their letter report of June 8, 1960 stated in part:

> "The excavations for the keyways brought out the presence of more and thicker red shale beds than had been anticipated. Also the open vertical joints extended to greater depth than had been expected."

Treatment of the upper part of the abutments was still in progress when the board examined the abutments in October 1961 and made further recommendations regarding treatment and design analysis. See appendix A.

The overrun in rock excavation for the dam was 110,418 cu.yds., and the overrun in the concrete for the dam was 66,644 cu.yds. The number and length of cutoff tunnels in the shale



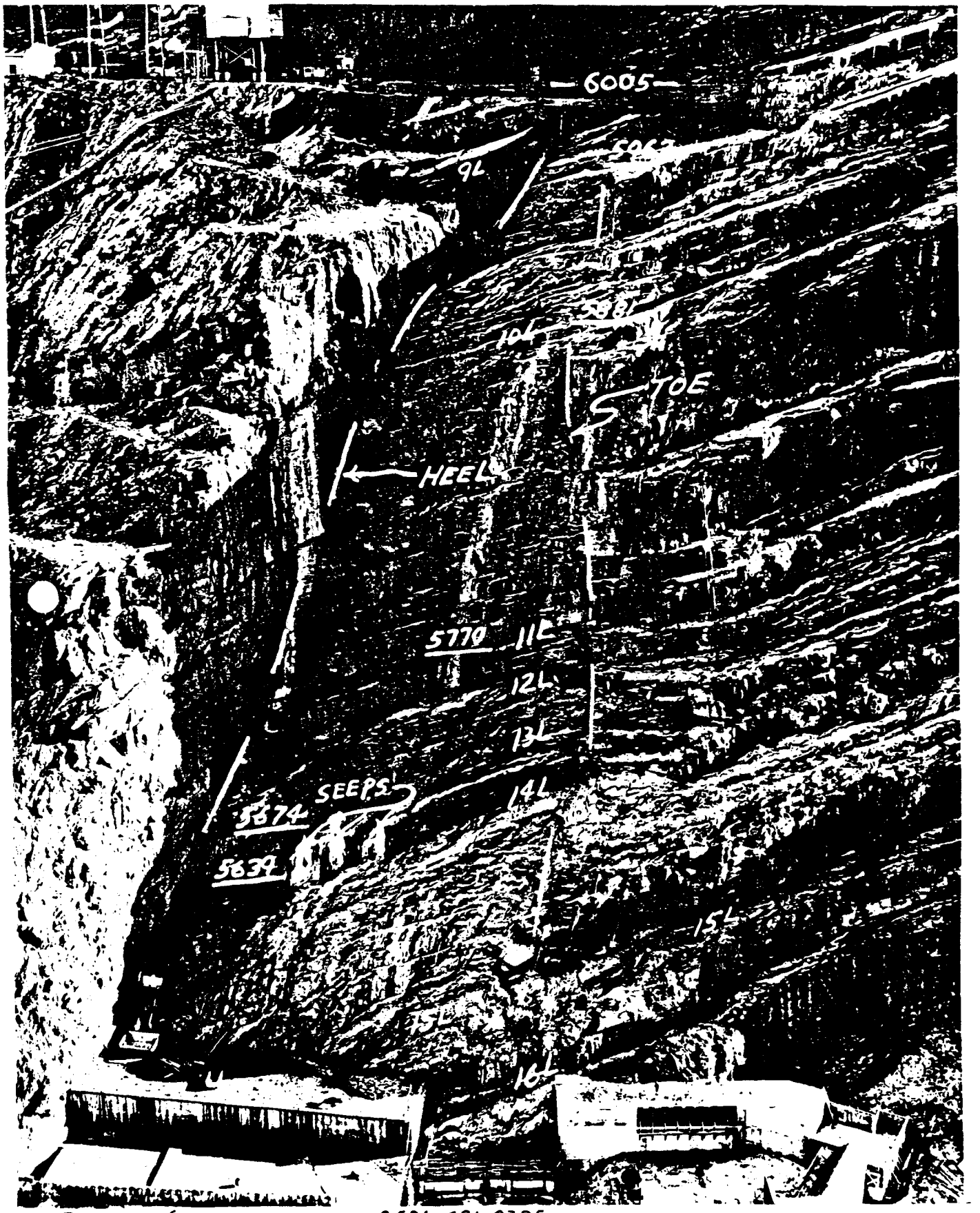
Flaming Gorge Dam P591-421-2584 Valley Bottom Excavation

Figure 8

Flaming Gorge - Right Abutment Keyway

P591-421-2785

Figure 9



FLAMING GORGE P591-421-2795

Lower Left Abutment Keyway 12/15/60

Figure 10

Flaming Gorge Dam - Upper Left Abutment Keyway 9/30/60

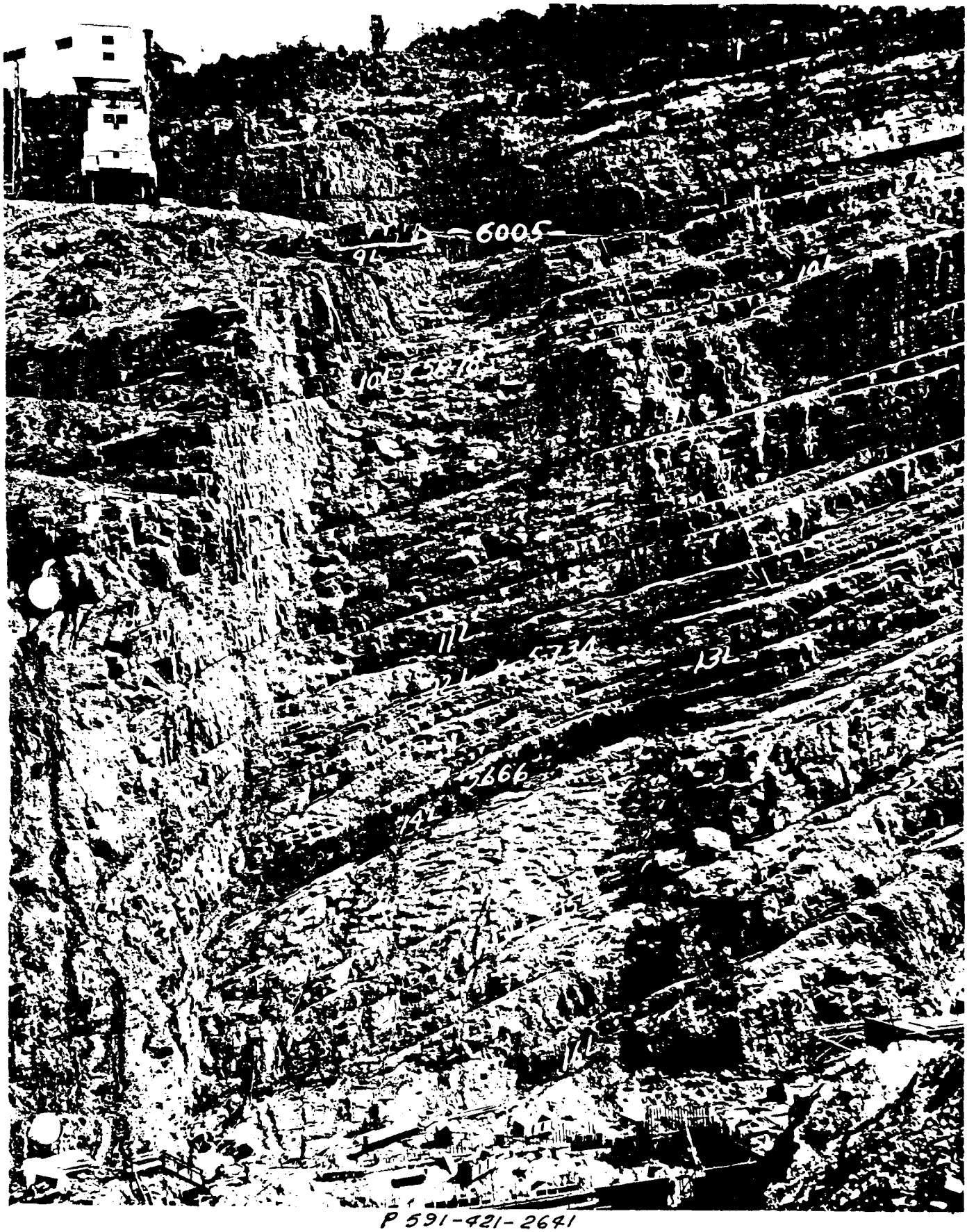


Figure 11

zones were also materially changed resulting in an increase from 750 to 2,859 cu.yds. for shale tunnel excavation.

Protection and anchorage of the shale and sandstone downstream were items that were covered under orders for change.

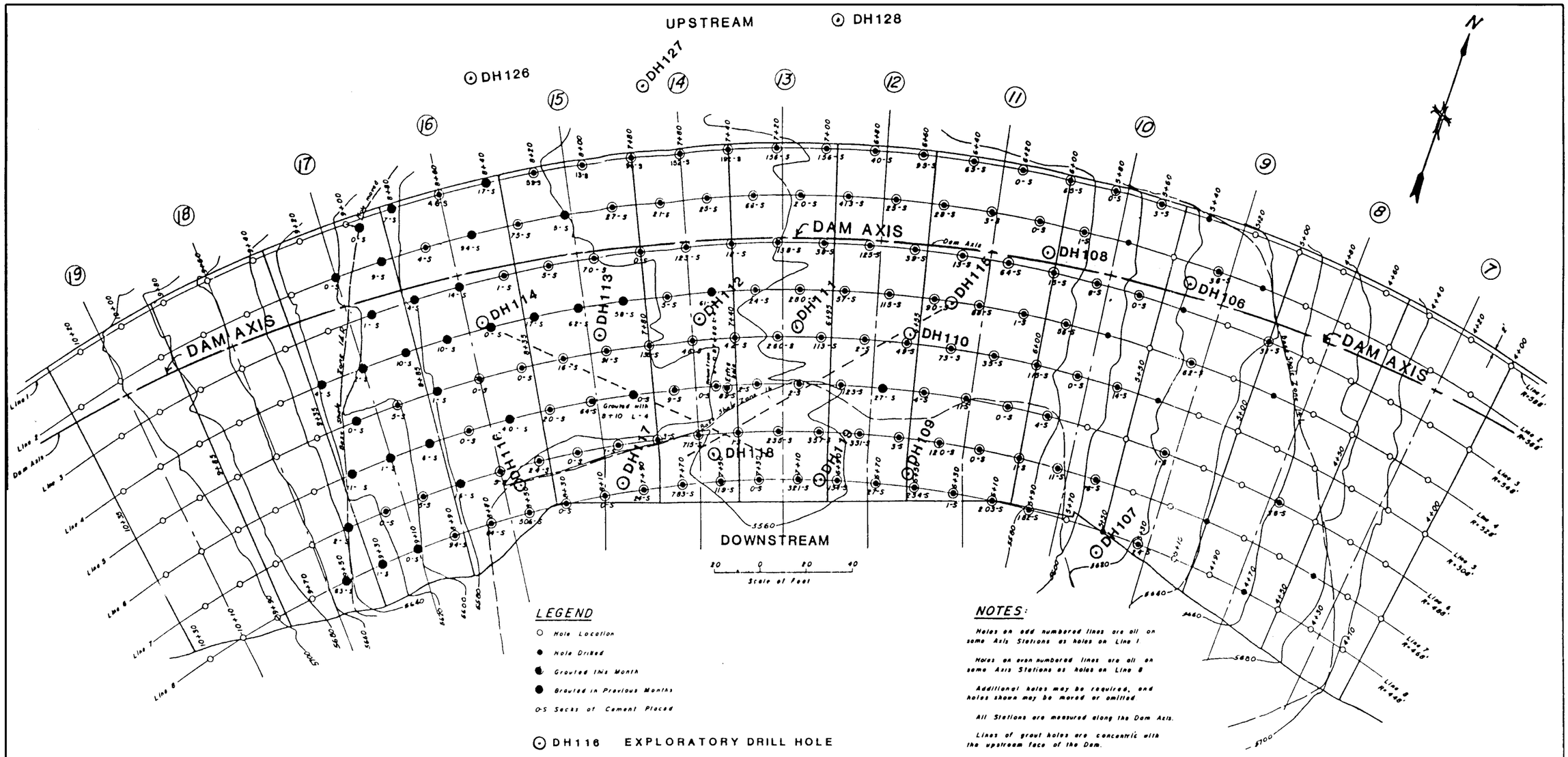
The total contract earnings of the Arch Dam Constructors was $37,028,481 which was an increase of $7,425,984 over the original contract amount of $29,602,497. Most of the cost increase was related to the changes in the foundation treatment, i.e., the overrun in the keyway excavation; the extra cement and concrete; protection of shale layers; and additional drilling, grouting and drainage.

IV. GROUTING AND DRAINAGE

8. Reference. Part E, "Foundation Grouting, Drilling and Drainage" of Chapter XII, Construction, Dam, Powerplant and Appurtenant Structures from the Technical Record of Design and Construction has been reproduced in appendix D.

9. Design. (a) Consolidation grouting. The B-hole grouting was modified during construction as described in appendix D. The layout and takes of the B-hole grouting in the valley bottom through September 1960 are shown on figure 12. The layout and takes in the abutment B-holes are shown on figures 13 and 14.

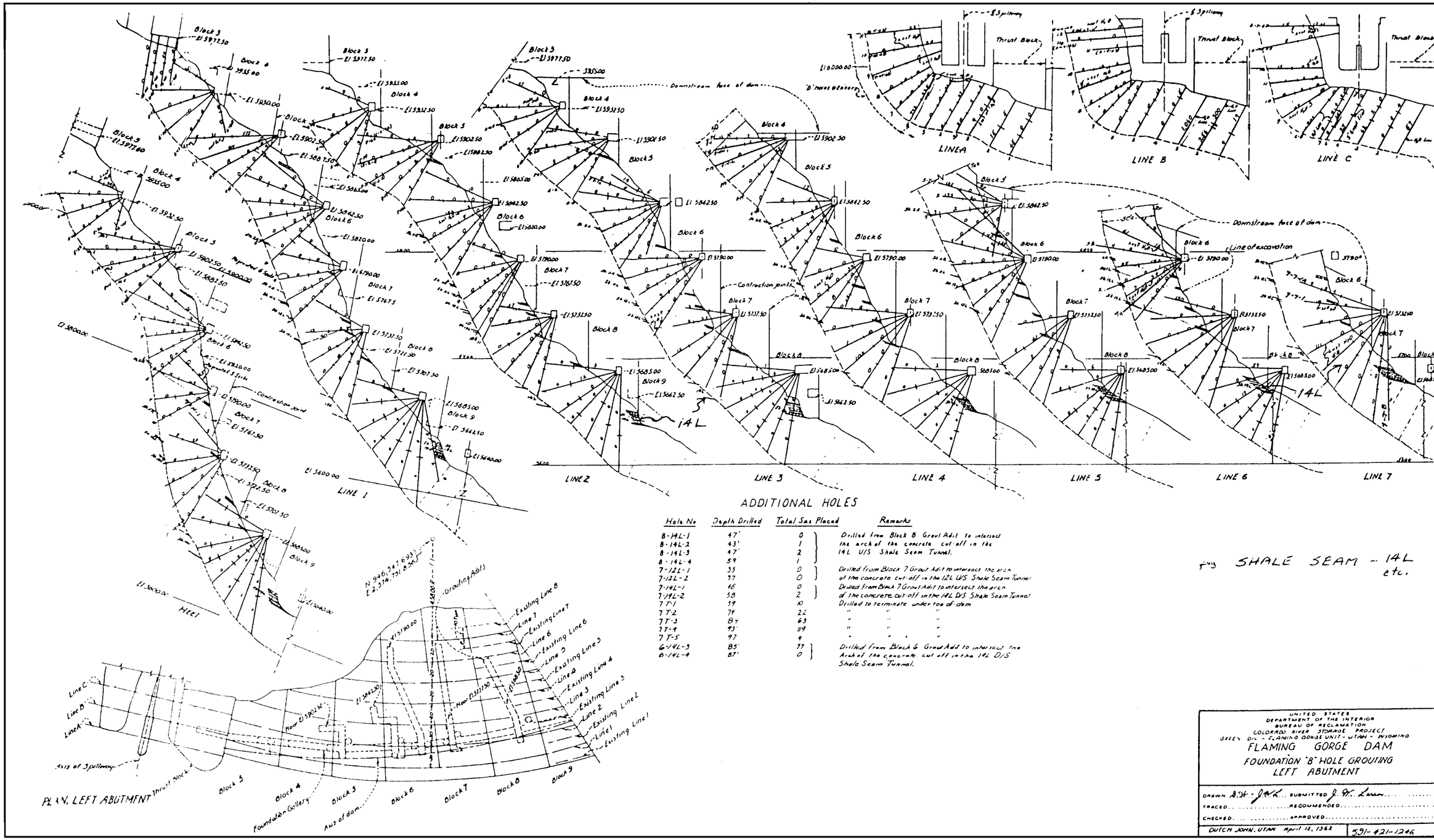
(b) Deep curtain. The design of the deep cutoff grout curtain (A-holes) is shown on figure 15 and on figures 200 and 201 of appendix D; this design was modified from the original contract design to include deeper holes above shale zone 14R and an extension at the top of the right abutment. The developed profile for the installed A-line showing the depth of holes, spacing and takes is presented on figures 16, 17 and 18 of this report. The location and profile of the right abutment auxiliary A-hole grout curtains are shown on figure 8 of appendix C, and the location and profile of the left abutment auxiliary A-hole grout curtain are shown on figure 9 of appendix C. The grout takes for the right and left abutment auxiliary curtains are shown on figures 19 and 20. The plan and profile of the foundation grouting of right abutment beyond end of dam are shown on figure 7 of appendix C; this curtain was installed to protect the switchyard from excessive seepage, and the grout takes are shown on the profile on figure 21. A geologic map based on surface mapping



REF. DWG. 591-421-902

SEPT. 1960 *MONTHLY PROGRESS REPORT*

FIGURE 12



ADDITIONAL HOLES

Hole No	Depth Drilled	Total Sax Placed	Remarks
8-14L-1	47'	0	Drilled from Block 8 Grout Adit to intersect the arch of the concrete cut-off in the 14L U/S Shale Seam Tunnel.
8-14L-2	43'	1	
8-14L-3	47'	2	
8-14L-4	59'	1	
7-12L-1	35	0	Drilled from Block 7 Grout Adit to intersect the arch of the concrete cut-off in the 12L U/S Shale Seam Tunnel
7-12L-2	37	0	
7-14L-1	46	0	Drilled from Block 7 Grout Adit to intersect the arch of the concrete cut-off in the 14L D/S Shale Seam Tunnel
7-14L-2	58	2	
7 T-1	59	10	Drilled to terminate under toe of dam
7 T-2	74	22	" " "
7 T-3	84	63	" " "
7 T-4	93'	119	" " "
7 T-5	97'	4	" " "
6-14L-3	85'	77	Drilled from Block 6 Grout Adit to intersect the Arch of the concrete cut off in the 14L D/S Shale Seam Tunnel.
6-14L-4	87'	0	

Fig SHALE SEAM - 14L etc.

UNITED STATES
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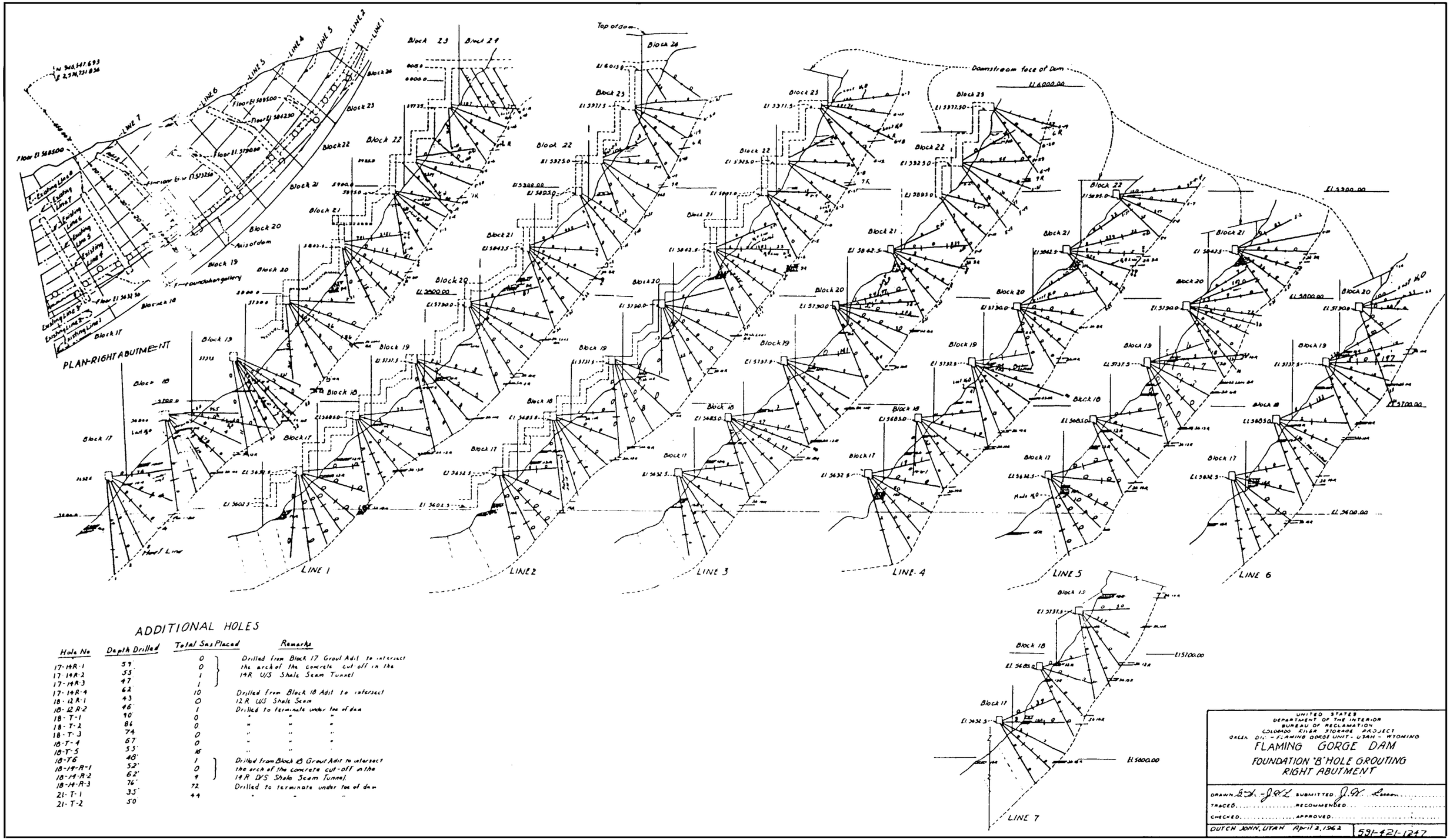
COLORADO RIVER STORAGE PROJECT
GREEN DIV - FLAMING GORGE UNIT - UTAH - WYOMING

FLAMING GORGE DAM
FOUNDATION "B" HOLE GROUTING
LEFT ABUTMENT

DRAWN D.H. - J.K.L. SUBMITTED J. K. Lewis
TRACED ..... RECOMMENDED .....
CHECKED ..... APPROVED .....
DUTCH JOHN, UTAH April 12, 1962 | 591-421-1246

FIGURE 13



ADDITIONAL HOLES

Hole No	Depth Drilled	Total Sas Placed	Remarks
17-14R-1	59'	0	Drilled from Block 17 Grout Adit to intersect the arch of the concrete cut-off in the 14R U/S Shale Seam Tunnel
17-14R-2	55'	0	
17-14R-3	47'	1	
17-14R-4	62'	1	
18-12R-1	43'	10	Drilled from Block 18 Adit to intersect 12R U/S Shale Seam
18-12R-2	46'	0	
18-T-1	90'	1	Drilled to terminate under toe of dam
18-T-2	86'	0	" " "
18-T-3	74'	0	" " "
18-T-4	67'	0	" " "
18-T-5	55'	0	" " "
18-T6	48'	16	" " "
18-14-R-1	52'	1	Drilled from Block 18 Grout Adit to intersect the arch of the concrete cut-off in the 14R D/S Shale Seam Tunnel.
18-14-R-2	62'	0	
18-14-R-3	76'	9	
21-T-1	35'	72	Drilled to terminate under toe of dam
21-T-2	50'	44	" " "

FIGURE 14

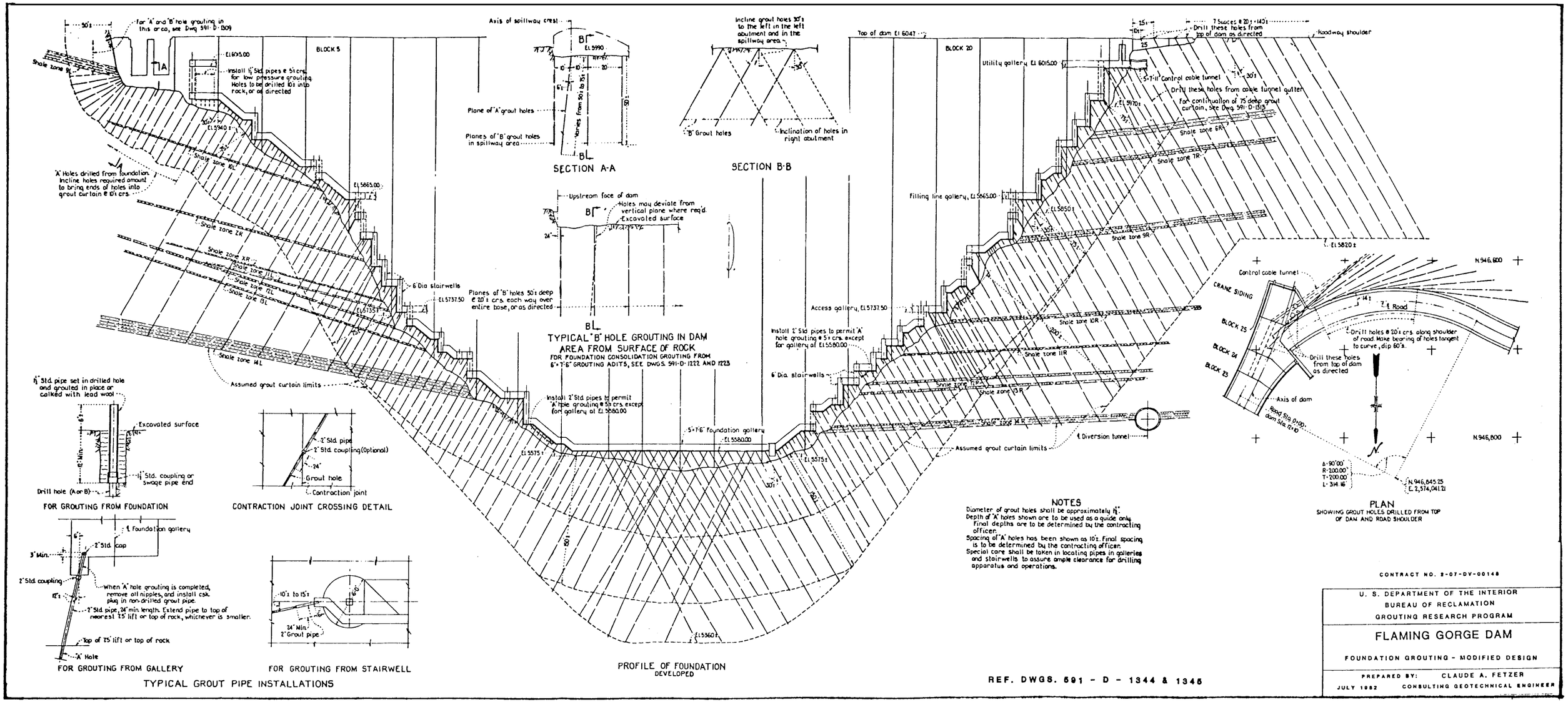
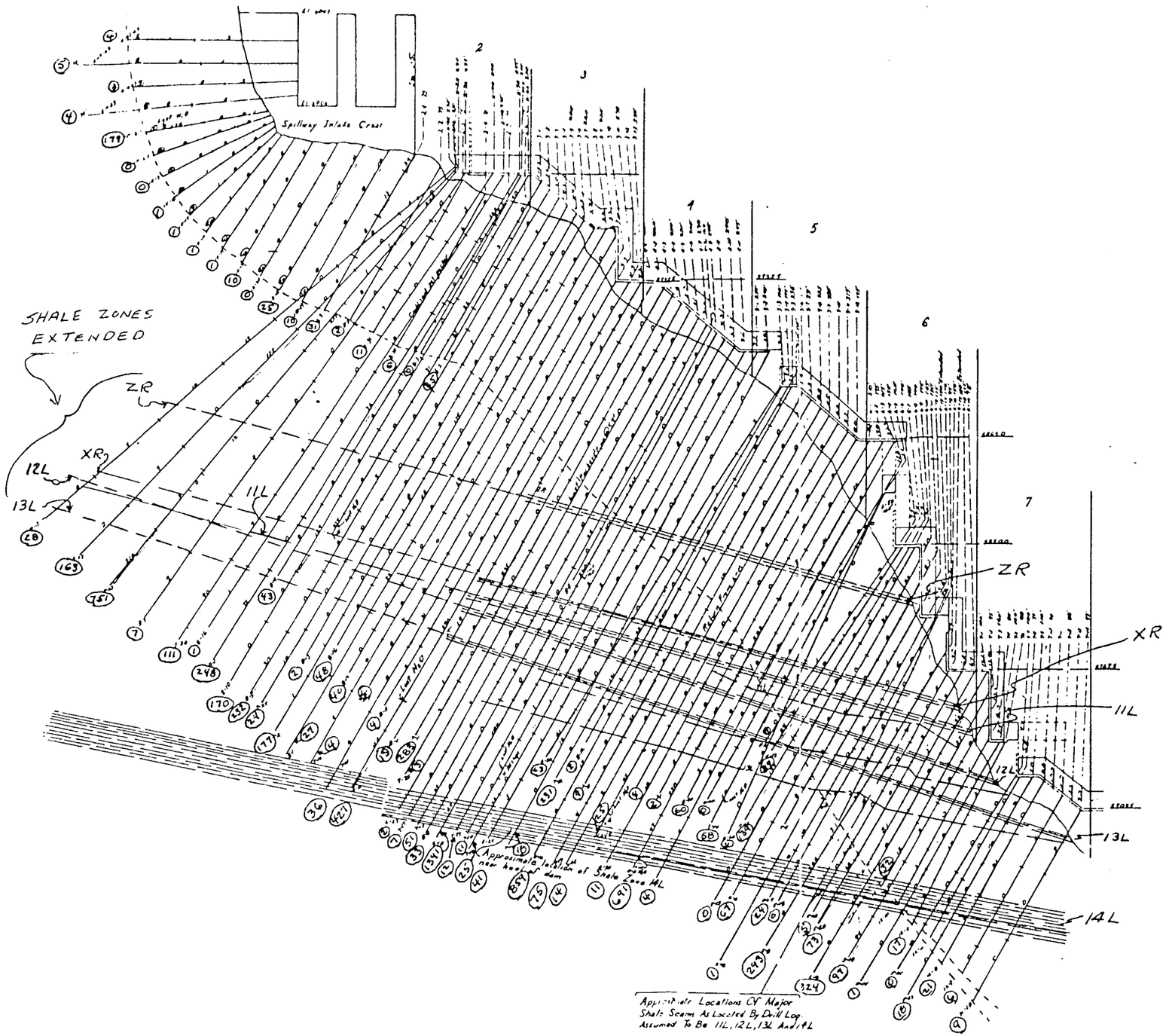
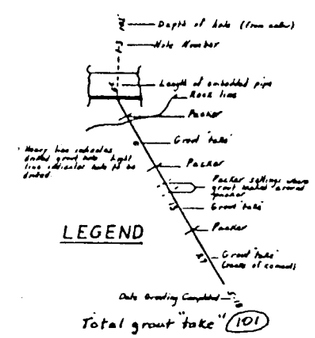

TYPICAL "B" HOLE GROUTING IN DAM AREA FROM SURFACE OF ROCK
FOR FOUNDATION CONSOLIDATION GROUTING FROM 6'x7'6" GROUTING ADITS, SEE DWGS. 591-D-1222 AND 1223
SECTION A-A
SECTION B-B
PROFILE OF FOUNDATION
DEVELOPED
FOR GROUTING FROM FOUNDATION
CONTRACTION JOINT CROSSING DETAIL
FOR GROUTING FROM GALLERY
FOR GROUTING FROM STAIRWELL
TYPICAL GROUT PIPE INSTALLATIONS
NOTES
Diameter of grout holes shall be approximately 1½".
Depth of "A" holes shown are to be used as a guide only. Final depths are to be determined by the contracting officer.
Spacing of "A" holes has been shown as 10'±. Final spacing is to be determined by the contracting officer.
Special care shall be taken in locating pipes in galleries and stairwells to assure ample clearance for drilling apparatus and operations.
PLAN
SHOWING GROUT HOLES DRILLED FROM TOP OF DAM AND ROAD SHOULDER
REF. DWGS. 591 - D - 1344 & 1345
CONTRACT NO. 8-07-DV-00148
U. S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GROUTING RESEARCH PROGRAM
FLAMING GORGE DAM
FOUNDATION GROUTING - MODIFIED DESIGN
PREPARED BY: CLAUDE A. FETZER
JULY 1982 CONSULTING GEOTECHNICAL ENGINEER

FIGURE 15





LEFT ABUTMENT

A-LINE GROUTING PROFILE

REF. DWG. 801-421-1248

CONTRACT NO. 8-07-DV-00148

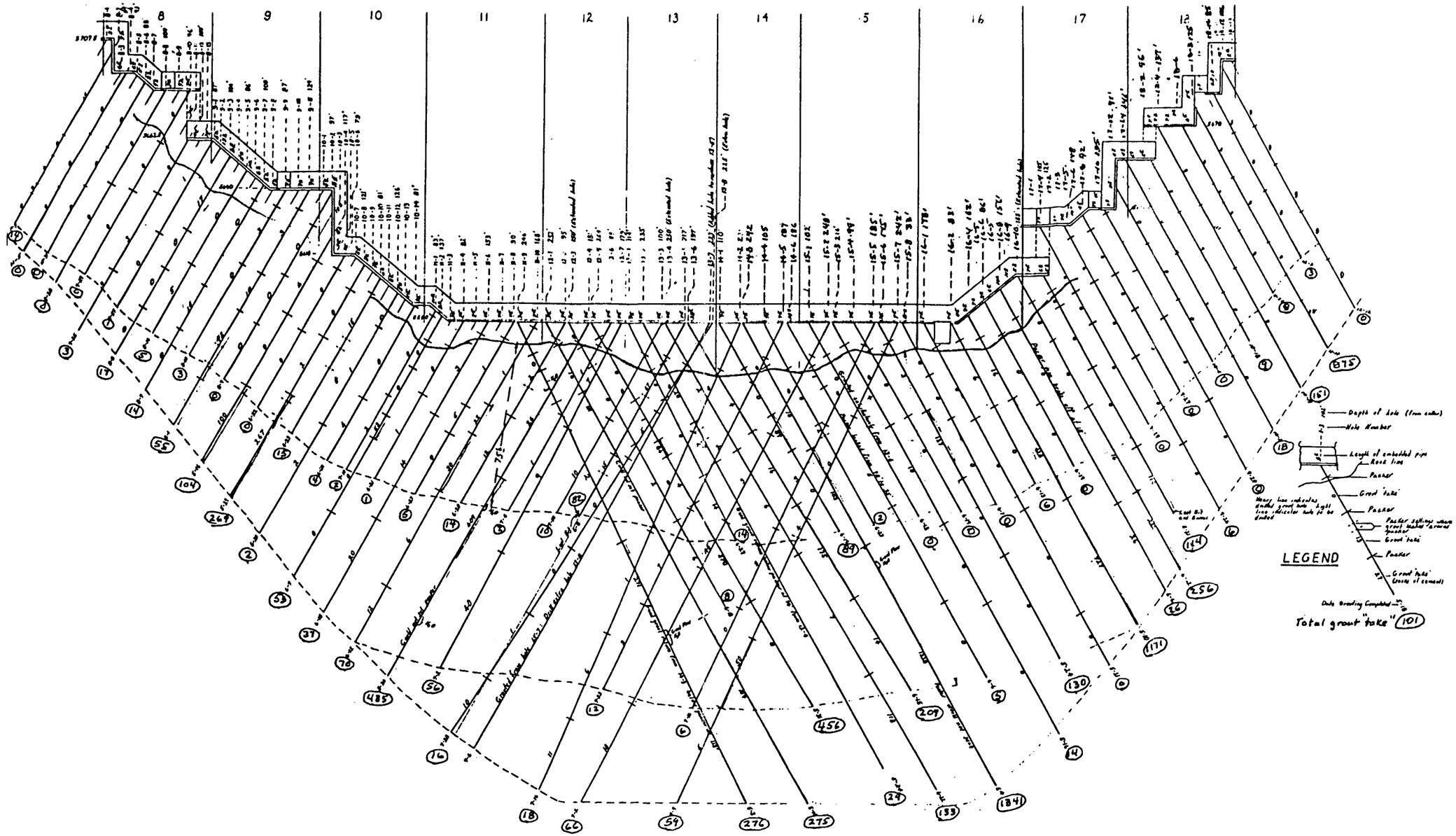
U. S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GROUTING REBEARCH PROGRAM

FLAMING GORGE DAM

FOUNDATION GROUTING A-LINE
LEFT ABUTMENT

PREPARED BY: CLAUDE A. FETZER
JULY 1969 CONSULTING GEOTECHNICAL ENGINEER

FIGURE 16



VALLEY BOTTOM

A-LINE GROUTING PROFILE

REF. DWG. 581-421-1240

CONTRACT NO. 8-07-DV-00148

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

FLAMING GORGE DAM
FOUNDATION GROUTING A-LINE
VALLEY BOTTOM

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

FIGURE 17

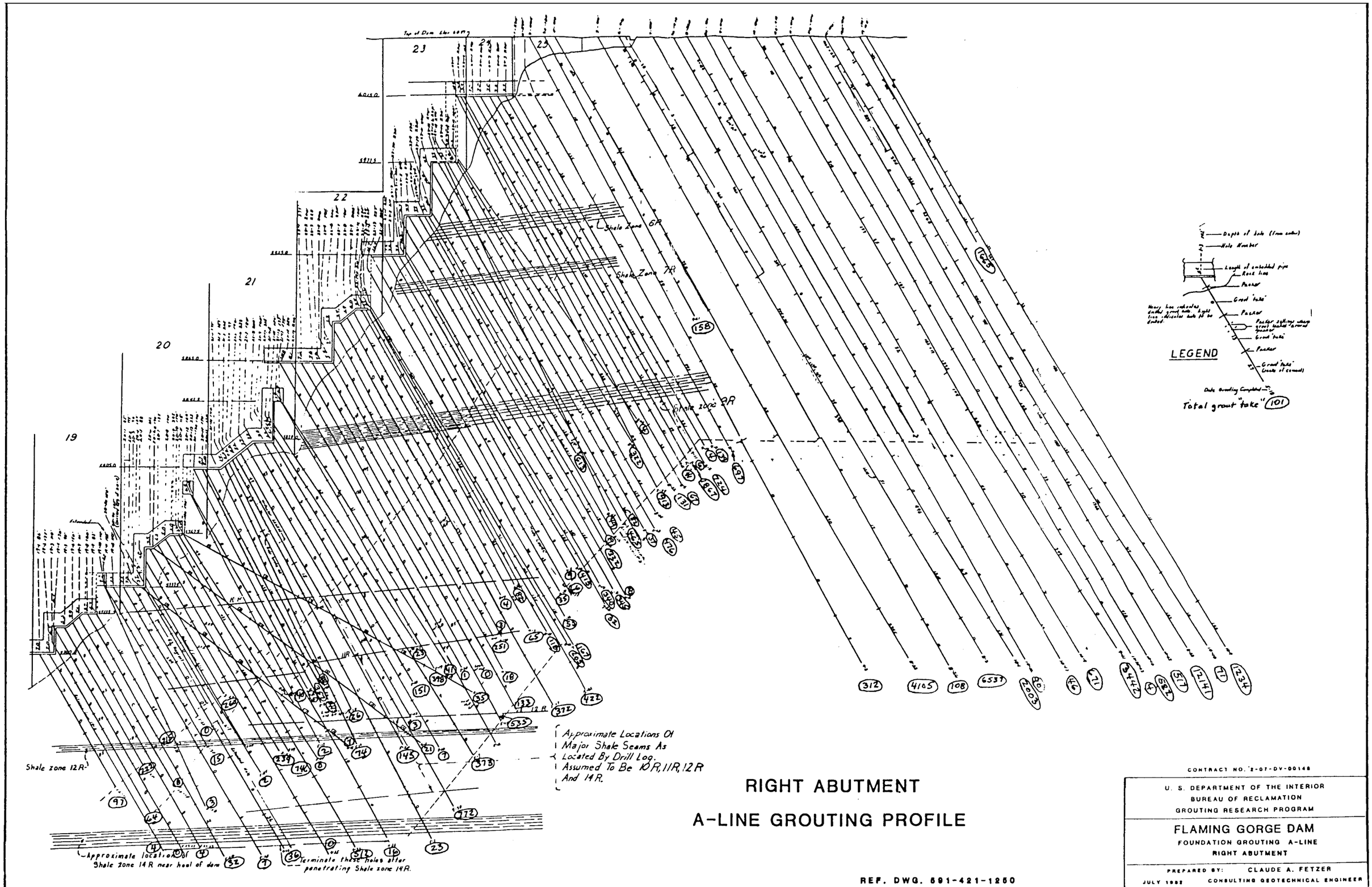

Shale Zone 6R
Shale Zone 7R
Shale zone 9R
Shale zone 12R
Approximate Locations Of Major Shale Seams As Located By Drill Log. Assumed To Be 10R, 11R, 12R And 14R.
Approximate location of Shale zone 14R near heel of dam
Terminate these holes after penetrating Shale zone 14R.
LEGEND
Total grout "take" (101)
RIGHT ABUTMENT
A-LINE GROUTING PROFILE
REF. DWG. 591-421-1250
CONTRACT NO. 2-07-DV-00148
U. S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GROUTING RESEARCH PROGRAM
FLAMING GORGE DAM
FOUNDATION GROUTING A-LINE
RIGHT ABUTMENT
PREPARED BY: CLAUDE A. FETZER
JULY 1982
CONSULTING GEOTECHNICAL ENGINEER

FIGURE 18

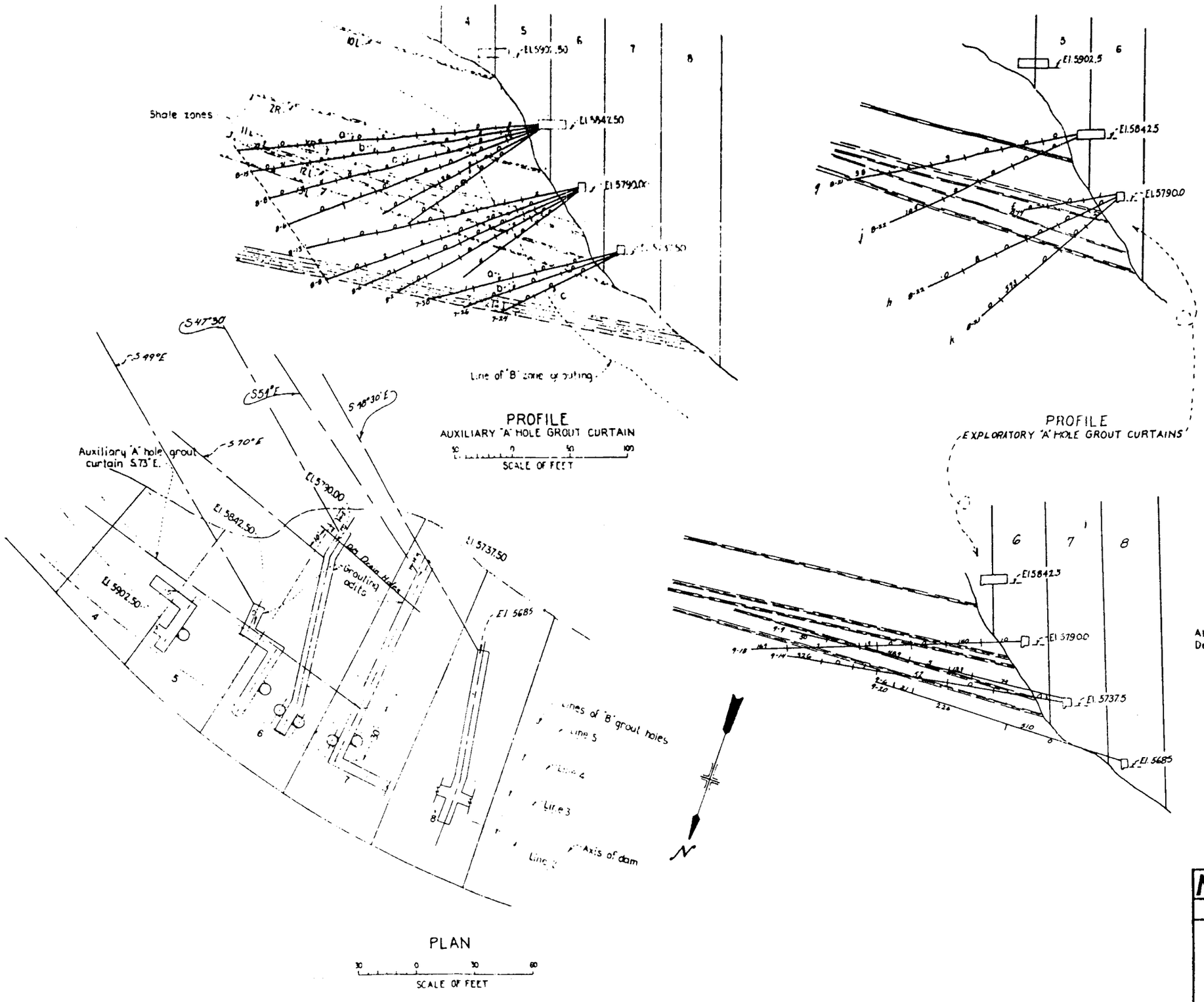


TABLE OF DRILL HOLES

HOLE	DIP	APPROX. LENGTH	BEARING
5a	4°30'	260'	S 73°E
5b	8°30'	250'	"
5c	14°	240'	"
5d	20°	230'	"
5e	27°	150'	"
6a	12°	240'	"
6b	19°	230'	"
6c	23°	200'	"
6d	28°	180'	"
6e	35°	120'	"
7a	13°	170'	"
7b	18°	140'	"
7c	25°	112'	"
5f	35°	135'	"
6f	10°	80'	S 70°E
6h	25°	180'	S 70°E
6k	40°	150'	S 70°E
5g	12°	200'	S 49°E
5j	24°	175'	S 49°E
6l	0°	230'	S 47°30'E
7g	+15°30'	240'	S 54°E
7h	+10°	240'	S 54°E
8a	+18°30'	220'	S 48°30'E

NOTES

All drill holes shall have a bearing as shown on plan view. Depths of holes shown in table are minimums. As drilling progresses and grout takes become apparent, dips and depths of holes may be changed as well as intermediate drill holes added, where required.

MARKED PRINT

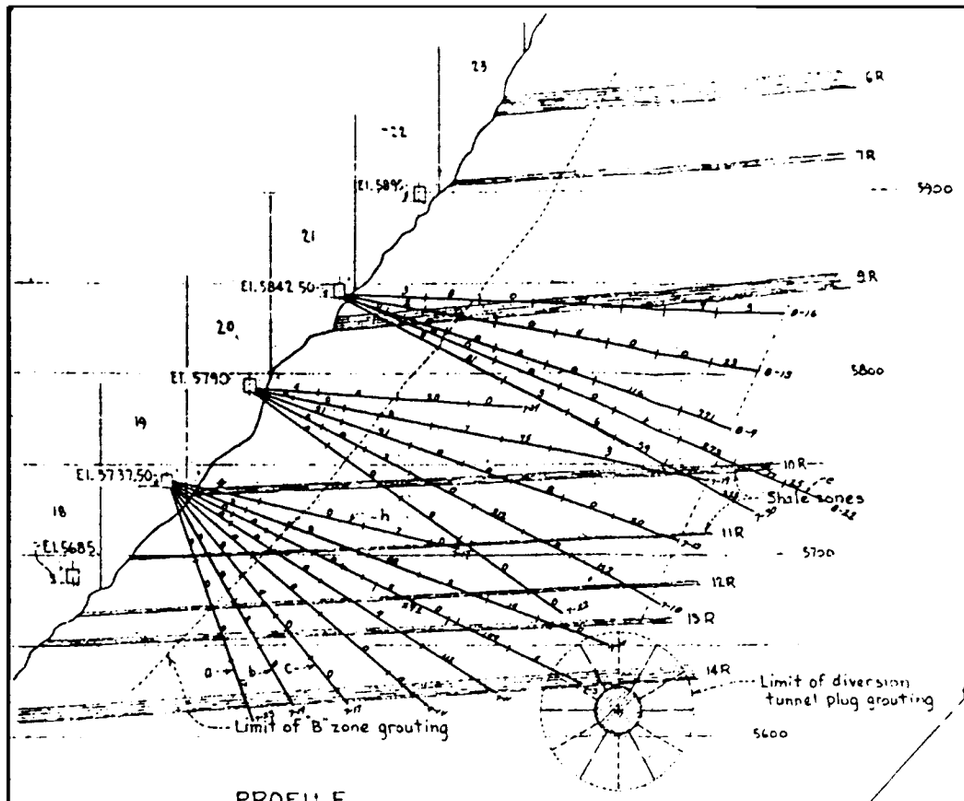
PROGRESS DRAWING

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
GREEN DIVISION-FLAMING GORGE UNIT-UTAH-WYO.

FLAMING GORGE DAM
LEFT ABUTMENT
AUXILIARY "A" HOLE GROUT CURTAIN

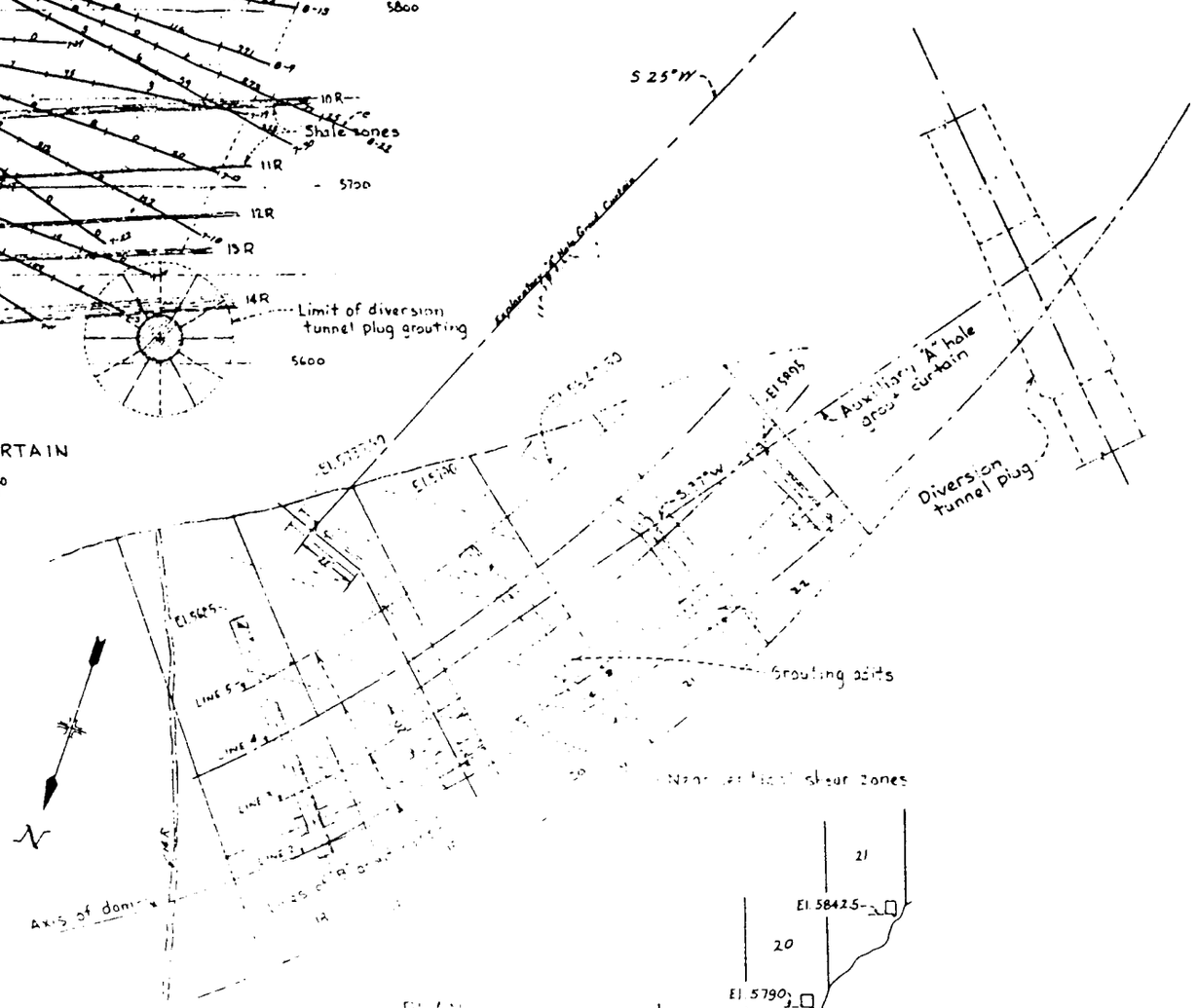
DRAWN H.N.C. SUBMITTED May Jno
TRACED RECOMMENDED R. J. Barrow
CHECKED [illegible] APPROVED F. V. Lindseth
ACT. CHIEF DESIGNING ENGINEER
DENVER, COLORADO, JULY 15, 1963 591-D-1360 A

FIGURE 19

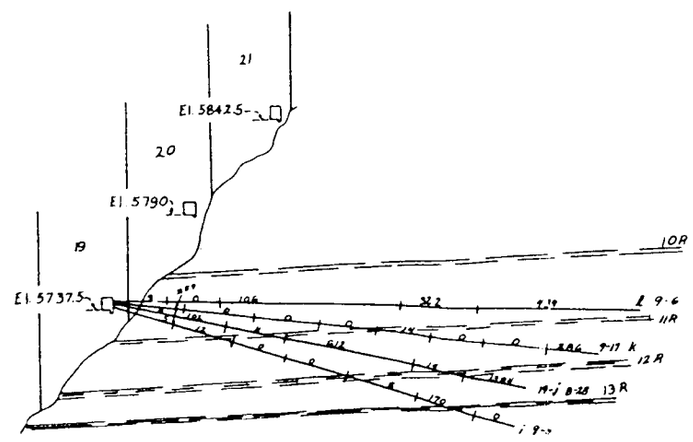


PROFILE
AUXILIARY "A" HOLE GROUT CURTAIN

SCALE OF FEET



PLAN



PROFILE
EXPLORATORY "A" HOLE GROUT CURTAIN

TABLE OF DRILL HOLES

HOLE	DIP	APPROX. LENGTH
19 a	70°	140'
19 b	60°	135'
19 c	50°	154'
19 d	40°	189'
19 e	32°	215'
19 f	25° 30'	255'
19 g	20°	260'
19 h	13°	165'
20 a	35	210'
20 b	27°	260'
20 c	19°	250'
20 d	11°	260'
20 e	4°	150'
21 a	27°	260'
21 b	18°	230'
21 c	10°	240'
21 d	2° 30'	250'
21 e	22° 30'	300'
19 i	17°	239'
19 j	12°	240'
19 k	7°	280'
19 l	2°	300'

NOTES.

All drill holes shall have a bearing as shown on plan

Depths of holes shown in table are minimums. As drilling progresses and grout takes become apparent, dips and depths of holes may be changed as well as intermediate drill holes added, where required.

MARKED PRINT

PROGRESS DRAWING

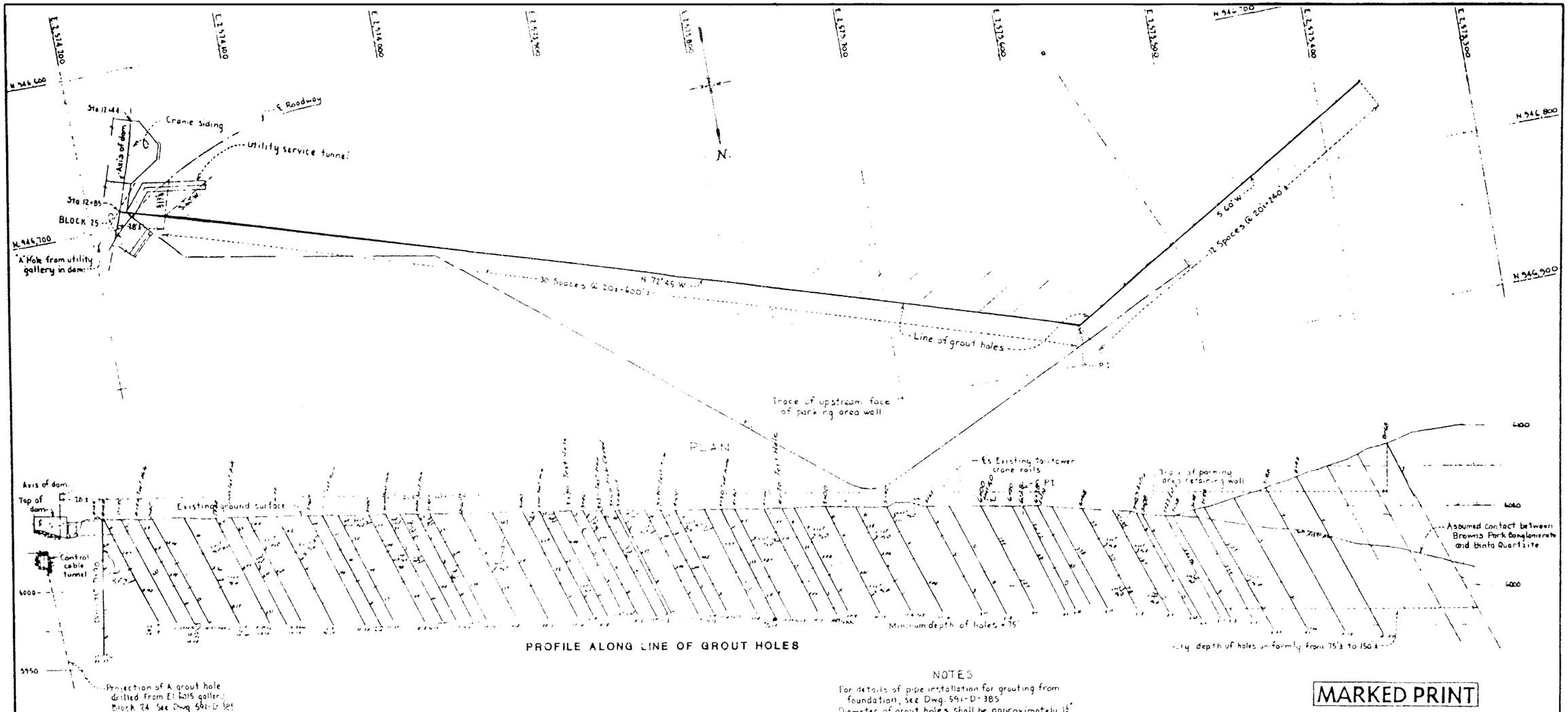
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
GREEN DIVISION FLAMING GORGE UNIT - UTAH-WYO

FLAMING GORGE DAM
RIGHT ABUTMENT
AUXILIARY "A" HOLE GROUT CURTAIN

DRAWN
TRACED
CHECKED
SUBMITTED E. R. Schultz
RECOMMENDED
APPROVED

DENVER, COLORADO MAY 24 1963

FIGURE 20



NOTES

For details of pipe installation for grouting from foundation, see Dwg. 591-D-385

Diameter of grout holes shall be approximately 1½"

All grout holes shall have a dip of 60° and bearing of S 40° W., unless otherwise noted.

Spacing of holes has been shown at 20± centers. Intermediate holes closer than 20' centers may be required, dependent upon high grout takes and depths and number of these holes shall be determined by the contracting officer

Grouting foundation above El. 6045 not required.

FOR LOCATION OF PARKING AREA AND SWITCHYARD, SEE FIGURE 3.

ALSO SEE BARRIER CURTAIN PLOTS ON FIGURES 4 & 8

MARKED PRINT

CONTRACT NO 2-07-DV-00148

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

FLAMING GORGE DAM
RIGHT ABUTMENT
SWITCHYARD BARRIER CURTAIN

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

RIGHT ABUTMENT SWITCHYARD BARRIER CURTAIN

REF. DWG. 591-D-1313A

FIGURE 21

of part of the right abutment during construction is shown with logs of holes DH-5, 120, 121 and 122 on figure 22.

(c) Foundation drainage. The modified design of the foundation drains drilled from the foundation gallery is shown on figure 11 of appendix C and the drains as drilled are shown on figure 203 of appendix D. The design of the left and right abutment downstream drainage curtains are shown on figures 12 and 13 of appendix C. The drainage holes in the right abutment that were drilled from the diversion tunnel are shown on figure 14 of appendix C.

10. Specifications. The specifications were complete insofar as the general performance of the work. The specifications required an apparatus for mixing and placing grout in continuous, uninterrupted flow at any pressure up to a maximum pressure of 500 psi. A circulating grout header at the hole was required, which had not been required on the other Bureau projects in this review. Grout holes were required to be tested with clean water under continuous pressure up to the required grouting pressure. Provisions as directed by the contracting officer were made for both descending-stage grouting (packers were not specified) and ascending-stage grouting with packers set at the top of each stage to be grouted. Payment for only one grouting connection hook-up per hole was to be made regardless of the method of grouting or the actual number of hook-ups required. Core drilling was required at the direction of the contracting officer.

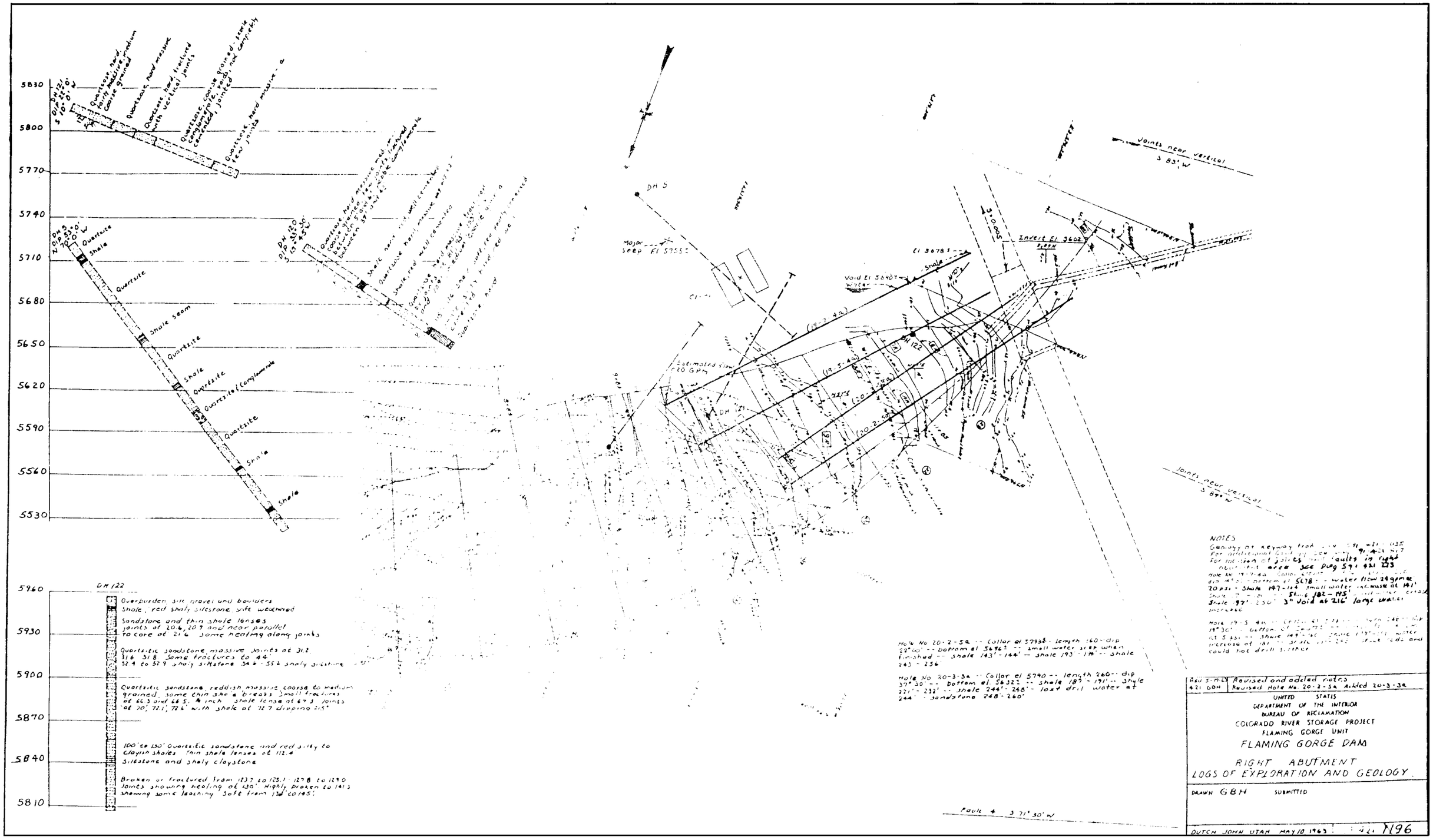

5830
5800
5770
5740
5710
5680
5650
5620
5590
5560
5530
5960
5930
5900
5870
5840
5810
DH 122
Overburden silt gravel and boulders
Shale, red shaly siltstone soft weathered
Sandstone and thin shale lenses
joints at 206, 207 and near parallel
to core at 216 some healing along joints
Quartzitic sandstone massive joints at 312,
316 318. Some fractures to 44°
324 to 329 shaly siltstone 346 - 356 shaly siltstone
Quartzitic sandstone reddish massive coarse to medium
grained some thin shale breaks small fractures
at 66.5 and 68.5; 1/4 inch shale lense at 67.5 joints
at 70', 72.1', 72.6' with shale at 72.7 dipping 25°
100' to 130' Quartzitic sandstone and red silty to
clayey shales thin shale lenses at 112.4
Siltstone and shaly claystone
Broken or fractured from 123.7 to 125.1 - 127.8 to 129.0
Joints showing healing at 130' Highly broken to 141.3
showing some leaching Soft from 134' to 145'
DH 5
Major seep El 5755±
Estimated flow 20 GPM
Joints near vertical S 83° W
Invert El 5602
Shale
Void El 5640±
Water
Joints near vertical S 87° W
Fault 4 S 71° 30' W
Hole No 20-2-5a -- Collar el 5793± length 160 - dip
22° 00' -- bottom el 5676± -- small water seep when
finished -- shale 143' - 144' -- shale 193 - 194 -- shale
243 - 256
Hole No 20-3-3a -- Collar el 5790 -- length 260 -- dip
37° 30' -- bottom el 5632 -- shale 187 - 191' -- shale
221' - 232' -- shale 244' - 248' - lost drill water at
244' - sandstone 248 - 260'
NOTES
Geology at keyway trench
For additional
For location of joints and faults in right
abutment area see Dwg 591 821 123
Hole No 19-7-4a Collar
dip 4° -- bottom el 5678 -- water flow 24 gpm at
70 ft - Shale 147 - 154 small water increase at 141'
Shale
Shale 97' - 150' 3" Void at 216' large water
increase
Hole 19-5 4a -- Collar el
19° 30' -- bottom el
at 5 ft -- Shale
pressure at 181 -- Shale
could not drill further
Rev 3-19-63 | Revised and added notes
4-21 GDH | Revised Hole No. 20-2-5a Added 20-3-3a
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER STORAGE PROJECT
FLAMING GORGE UNIT
FLAMING GORGE DAM
RIGHT ABUTMENT
LOGS OF EXPLORATION AND GEOLOGY
DRAWN GBH
SUBMITTED
DUTCH JOHN UTAH MAY 10 1963
196

FIGURE 22

11. Grouting methods and procedures. The monthly drilling and grouting reports, which contain the summaries of the daily drilling and grouting reports, could not be found for this project. This review was based on information in the Technical Record of Design and Construction and the available profiles of the grouted holes.

(a) Grout mixes. In the B-holes drilled from top of rock in the valley bottom, the water-cement ratios ranged from 5:1 to 1:1. In the B-holes drilled from transverse adits the water-cement ratios ranged from 5:1 to 1:1 with many of the holes being finished with mixes of 5:1 or 4:1. The water-cement ratios for A-line drilled from the foundation gallery and transverse adits are not presented in the available records. In the right abutment deep A-holes, it was reported that much of the grout pumped had a 3:1 water-cement ratio.

(b) Grout-injection pressures. The available records indicate the following pressures were used:

Consolidation grouting from rock surface (B-holes) - 100 psi maximum

Consolidation grouting from transverse adits (B-holes) - from 100 to 150 psi with maximum pressures of - - 175 psi

Deep curtain cutoff grouting (A-holes)- - - - - not given

Auxiliary A-holes - - - - - - - - - - - - - - - - not given

Right abutment deep A-holes - - - - - - - - - up to 450 psi

Right abutment beyond end of dam- - - - - - up to 150 psi

(c) Spacing of holes and closure. The specifications required the B-holes to be drilled and grouted until there was a maximum spacing between holes of 20 feet. During construction,

the area covered in the valley bottom and the depth of the holes were materially increased, but the 20-foot spacing was maintained. The B-holes drilled from the transverse adits were drilled on lines 20 feet apart and at various angles to give coverage from one adit level to the next.

The deep curtain A-holes were to be drilled and grouted to a maximum spacing of 5 feet with provisions for drilling closer if considered necessary. The records indicate that the holes were drilled and grouted to the maximum spacing of 5 feet and that many intermediate holes were also drilled and grouted.

Closure criteria were not given.

(d) Refusal criteria. The criteria for refusal in the specifications were as follows:

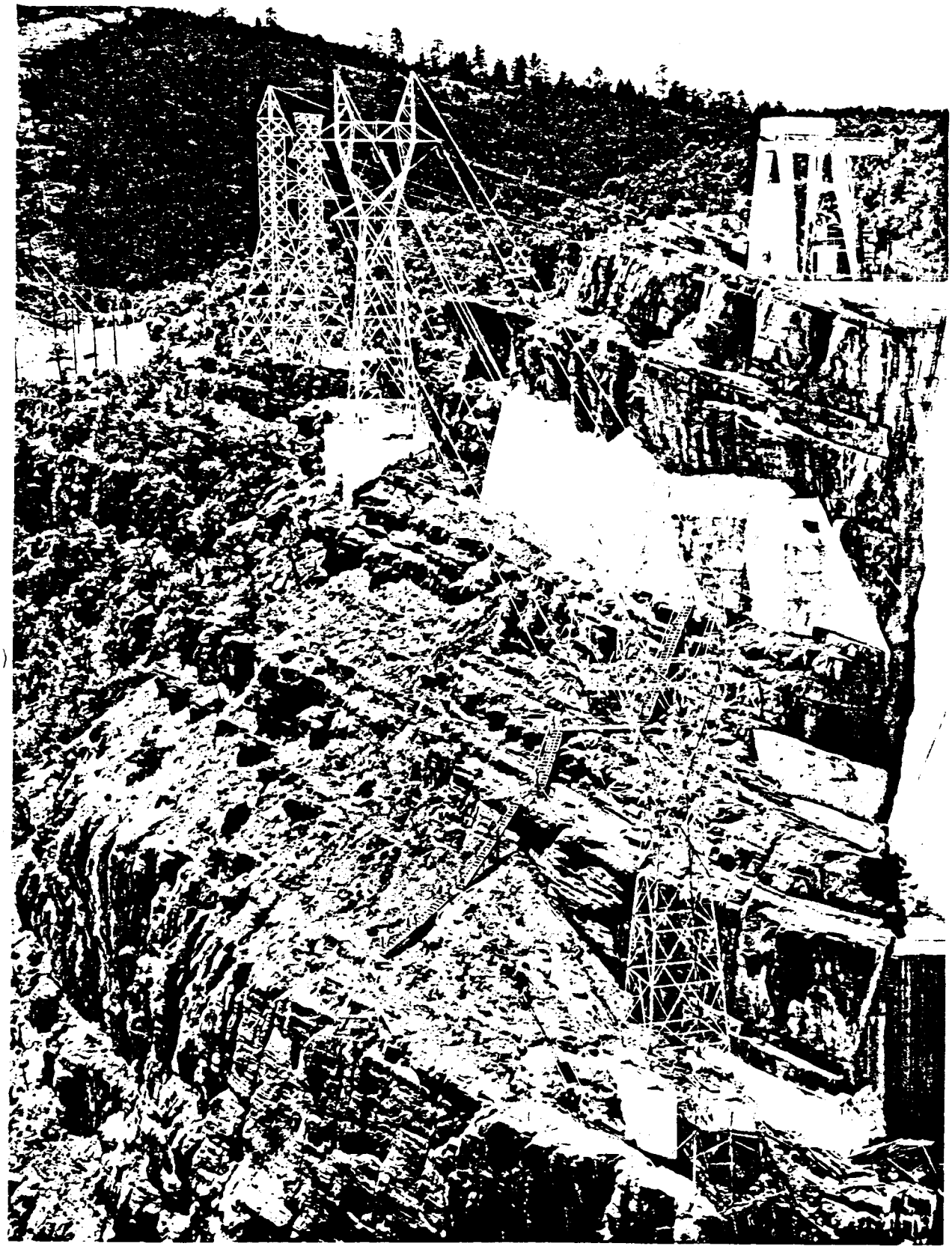
> "The grouting of any hole shall be continued until the hole or grout connection takes grout at the rate of less than 1 cubic foot of grout mixture in 20 minutes if pressures of 50 pounds per square inch or less are being used; in 15 minutes if pressures between 50 and 100 pounds per square inch are being used; in 10 minutes if pressures between 100 and 200 pounds per square inch are being used."

(e) Final backfill of holes. The specifications imply that holes will be backfilled under measurement for payment by indicating that payment for the cement "required to fill permanent pipes" will be made on the basis of the number of sacks used. However, there is no specific requirement for final backfilling of holes, nor is there any statement in the available records regarding sounding and final backfilling of completed grout holes.

V. ANALYSIS

12. Preconstruction geologic investigations. The preconstruction geologic investigations adequately determined that the site was suitable for an arch dam with a height of approximately 500 feet. The preconstruction geology reports anticipated seepage from the Cart Creek area based on the well-developed fault and joint systems located and mapped between Cart Creek and Green River as shown on figure 6. The geology report also discussed the springs existing downstream from the dam site and the probable consequences of reservoir head on seepage downstream from the dam. Seepage itself was not as much as a problem as was the possibility of building up high cleft pressures in the already thin wedge of rock remaining immediately downstream from the right abutment keyway foundation for the arch. Such high pressures if not controlled could have been detrimental to the stability of that abutment of the dam. The thin wedge of rock at the top of dam on the right abutment is shown in the photograph on figure 23.

Only one hole, DH 118, extended to the full depth of the A-holes; most other holes in the valley bottom did not extend to the full depth of the B-holes. There were insufficient explorations along the axis on the abutments to develop a reliable excavation line. The drift on the right abutment was located near the base of the abutment and the drift on the left abutment was located 150 feet downstream of the axis. Additional drifts were needed at the principal shale layers on both abutments to locate rock that was capable of supporting the structure and rock that was groutable.



P591-421-4942 FLAMING GORGE UNIT - COLORADO RIVER STORAGE PROJECT - UTAH-WYOMING

Flaming Gorge Dam - Right abutment showing transmission circuit to Switchyard, on towers C1-T1, C1-T2, C1-T3 and transmission line tower DTH for Vernal 138KV lines.

Figure 23

Highly weathered rock is not considered to be groutable because the joints and openings are often filled with residual soil-like materials that cannot be adequately removed before grouting. Several B-holes on the right abutment had high grout takes even though extensive excavation was accomplished beyond the design excavation line. In block 21, one hole in the heel line took 701 sacks at the concrete rock contact and 2,151 sacks in the next stage. One hole in block 23 took 2,207 sacks in the top stage and another hole took 1,787 sacks in the top stage. If Flaming Gorge Dam were being constructed under today's contracting rules, the cost increase due to the numerous changes in the foundation excavation and grouting programs would have been much greater than the 25 percent increase incurred. A detailed exploration program was needed at the final design site to obtain adequate information for design and to reduce construction changes.

13. Design. The A-line in the valley bottom had a vertical depth of about 220 feet, which is approximately 50 percent of net head. A review of the grout takes in the lowest stages of the valley bottom indicates that the depth was probably adequate. Other than the depth in the valley bottom, extensive changes in the design of the B-hole and A-hole grouting were required. Even the modified design developed for the A-line during construction was changed extensively during the actual installation. Inadequacies in the designed grouting and drainage systems were recognized as conditions were revealed during construction. It was most appropriate that the design continued through construction and that several changes in the drainage and grouting systems were

made by Bureau engineering and construction personnel with guidance from the board of consultants.

It is considered that abutment grouting and drainage tunnels as had been previously used at Kortes Dam and Hungry Horse Dam would have permitted more effective grouting and drainage than was achieved from the galleries, adits and top of abutment.

14. Grouting methods and procedures. (a) Specifications. It is considered that the specifications should have contained more specific requirements on the size of mixing equipment, grout pumps, and grout pipe through the packer, and that the specifications should have had a requirement for a specific time for the water pressure tests, and should have a requirement for final backfilling of holes. With the contracting climate at that time, the specifications were adequate as there are no recorded disputes resulting from the specifications.

(b) Grout mixes. As the data for individual holes are not available, specific comments on the mixes used cannot be made.

(c) Grout-injection pressures. The grout-injection pressures used in both the B-holes and in the A-holes are considerably in excess of what would be calculated from a balance of effective forces. The use of a gage pressure of 100 psi with a packer set at the surface in the B-holes is sufficient to lift and split almost any rock and may have been partly responsible for the leaks discussed on page 341 of appendix D.

The concrete in the dam was required to have a height of 100 feet within a radius of 200 feet before an A-hole could be grouted. The first A-hole grouting was accomplished from the

lowest foundation gallery, El. 5,737, in February 1962. As the concrete in the dam proper was completed in November 1962 the height of concrete undoubtedly was much greater than 100 feet. The use of a maximum gage pressure of 450 psi may have been the reason a packer could not be set in holes 12-4 and 14-2. The need for such high pressures is questioned as they could have lifted the dam and displaced blocks of rock in the abutments. This latter possibility was recognized in the grouting of the right abutment deep A-holes where the pressures were reduced.

(d) Spacing of holes and closure. Although there are no written rules on record on the ultimate spacing of holes, the board of consultants did provide guidance in their report dated June 8, 1960 as follows:

> "The Board is of the opinion that the consolidation grouting should extend to a minimum depth normal to the foundation surface of 50 feet and that the spacing of the grout holes should be such to provide a thorough degree of consolidation."

In reviewing the B-hole grouting for September 1960 on figure 12, it is considered that to meet the Board's guidance extra holes would have been needed in many areas particularly near the downstream half of blocks 11 through 16 where many adjacent holes took in excess of 200 sacks. In block 14, adjacent holes in lines 7 and 8 took 715 and 783 sacks, respectively, without an extra hole being provided between them. The need for the closer spacing of holes was also expressed in the grouting report on page 341 of appendix D by the following statement:

> "It is believed that consolidation of the foundation rock was satisfactory but that a closer spacing of holes would have been

required to completely "close-out" the B-hole grouting to provide an effective cutoff against seepage."

The unit take values of grouting from primary to final closure holes are not shown in the review data. A review of the profiles of the A-hole grouting on figures 16, 17 and 18, indicates many cases where additional split-spaced holes should have been drilled and grouted adjacent to large take holes to tighten the curtain. In other cases, split holes were not drilled to sufficient depth to tighten the curtain. Some examples of holes requiring additional splitting or drilling to greater depth for a tight curtain are as follows:

Left Abutment - (1) Holes 7-2 and 7-4 in Block 7 drilled to 95 feet and 82 feet respectively should have been drilled to 120 feet to tighten hole 7-3 with a grout take of 94 sacks in the bottom zone.

(2) Hole 4-8 in Block 4 should have been split to 2-1/2-foot centers to full depth of 250 feet to tighten curtain in bottom zone where take was 104 sacks. One side of this hole was split but only to a depth of 120 feet.

(3) Hole 2-22 should have been split on both sides to a depth of 230 feet to tighten bottom zone with a take of 216 sacks.

Right Abutment - (1) Hole 10 of the extended curtain had high-grout takes in practically every zone but was split 10 feet on one side but not on the other. This left a 20-foot gap in the extended curtain.

(2) There is a 20-foot gap in the extended curtain between holes 4 and 5. Hole No. 4 had high grout takes in two zones.

(3) Hole 6, on the extended curtain had high-grout takes in practically every zone but was split only to 10-foot spacing.

Valley bottom - (1) Hole 14-3 drilled to a depth of 231 feet had a take of 1,228 sacks in the lower zone but 10-foot split holes were drilled to a depth of only 110 feet.

(2) Hole 12-4 drilled to a depth of 181 feet had a grout take of 409 sacks in the lower zone. The 10-foot spaced, split holes 12-2 and 12-6 were only drilled to a depth of only 95 to 98 feet respectively.

(e) Refusal criteria. Although the records do not indicate the field criteria used to determine when a hole had reached refusal, it is assumed that the requirements of the specifications were followed. It is not considered necessary to stay on a hole accepting 1 cubic foot of grout mixture for 20 minutes. Regardless of the pressure, if a hole does not accept 1 cubic foot of grout mixture in 5 to 10 minutes, the hole should be considered at a point of refusal.

(f) Final backfill of holes. There is evidence that some of the holes were not filled with solid grout. As indicated on page 341 of appendix D many of the previously grouted "criss-cross" A-holes returned grout as holes from the opposite direction were grouted. Field evidence backed up by laboratory tests have shown segregation and sedimentation occurs within the grout hole itself which often leaves the center of a vertical grout hole or

the top of an angled grout hole with a continuous void filled with water. This is often the condition when the hole is filled with the same grout mix used to complete the last stage of grouting. Filling grout holes takes a special effort involving placement of a tremie pipe to the bottom and filling the hole with thick grout having a water-cement ratio of not more than 0.8:1.

15. Unexpected geologic conditions encountered during grouting. Many of the unexpected geologic conditions were encountered in the excavation. A travel report dated March 21, 1960 from L. G. Puls, Chief Designing Engineer and W. H. Irwin, Chief, Engineering Geology Branch, describes a condition on the right abutment above shale zone 10R. In the central portion of the excavation there was a zone of slightly sheared, closely spaced, near-vertical joints, and immediately upstream there were several vertical soil-filled joints ranging from a fraction of an inch to 2 or 3 inches wide with one joint 6 to 8 inches wide. These soil-filled joints, which started at the top of the shale layers, are called relief joints, and should have been expected in a deeply eroded valley having abutments composed of sandstone interbedded with shale zones.

The three-fold increase from 52,000 to 154,318 sacks of cement used in the grouting indicates that the rock had more voids than had been expected during design.

16. Grout takes as related to geology. The high-grout takes in the valley bottom and abutments should have been expected from the geologic conditions. As the valley was eroded, the sides tended to move inward leaving relief joints in the sandstone.

This inward movement also caused compression in the valley bottom resulting in shearing of the rock and buckling or springing of the beds.

The foundation geology map on figure 7 shows numerous near vertical joints and shears running longitudinally across the valley floor from one abutment to the other. On the surface these features appeared tight and partially cemented, but combined with partings along bedding planes they afforded avenues of communication between grout holes. Water flows occurred from several of the valley grout holes. Grout communication occurred between several of the grout holes as shown on the valley section of the A-line grouting profile on figure 17. Initial consolidation or B-hole grouting apparently tightened up zone 1 of the A-line grout curtain as most of the grout takes were in zones 2 and 3 as shown in the tabulation on figure 24. Pressures over 100 psi were used in the B-hole grouting, which could have lifted some rock although the numerous vertical joints probably provided pressure relief and venting to the surface in most cases. While the B-hole grouting did not completely "close-out" the area, the A-hole curtain has evidently provided an effective cutoff against seepage in this area.

High grout takes should have been expected in the abutments because of the relief joints. Although the March 13, 1961 inspection of the abutment keyways by Puls and Irwin indicated that the joints were tight beyond a depth of 5 to 10 feet back of the face, the drill-water losses and high grout takes in the B-holes and A-holes indicate that open joints must have been more

A Comparative Study of Water Loss in Exploratory Drill Holes And Grout Program – Flaming Gorge Dam

Only Drill Holes in The General Area of the Dam Foundation Excavation And With Noticable Amount Water Loss During Percolation Tests Are Here used and Tabulated.

	Drill Hole	Elev.	Packer Depth At	Water Lost	Pressure	T.D.	Remarks.
LEFT ABUTMENT	D/S. D.H. 105	6033.3	10', 20'	20 gpm, C of P	100 psi	50'	Porous from 12' to 17' and 20' to 30' in S.S. and Sh
	D/S D.H. 100	5872.6	15, 20', 75'	11 gpm – 16 gpm	150 psi	123'	Greatest loss = 11 g.p.m. to 16 g.p.m.
			85'				
	℄ D.H. 104	5970.5	20', 25', 30', 40', 45'	20 gpm C of P	100 psi	100'	Lost drill water at 28' 10" open joint Br. at 4'
	℄ D.H. 106	5681.1	32'	11 gpm	150 psi	52'	Lost 11 g.p.m. Bedrock at 12'
	℄ D.H. 108	5641.5	47'	20 gpm C of P	75 psi	70.5'	Bedrock at 35.5' Lost drill water at 45'
			42	18 gpm C of P	65 psi		
IN RIVER BED	D/S D.H. 110	5600.9	Hole is tight although some soft zones present			62'	Bedrock at 27'.
	D/S D.H. 111	5588.4	35'	19 gpm. C of P	110 psi	63'	Packer wouldn't hold 75 psi. at 41', 46' Br at 28'
	D/S D.H. 112	5589.6	Hole is tight below 52'			78'	38' to 52' packer wouldn't hold. Br. at 32'
	D/S D.H. 113	5599.4	34'–44'	9 gpm	150 psi	65'	Bedrock at 30' Hole is fairly tight.
	D/S D.H. 114	5808.0	Lost drill water from 58' to 68'			170'	Hole drilled 40° from horizontal
							Tight below 68'
	U/S D.H. 126	5601.6	50, 55,	20 gpm C of P	150 psi	66'	Bedrock at 31' Water back flow from 40', 50', 55'
	U/S D.H. 127	5591.5	38', 43'	20 gpm C of P	85 psi	66'	" " " 31' Water flowed from hole for
			Lost water at 47'	Back Flow at 56'			8 hrs. after completion at 4 g.p.m.
	U/S D.H. 128	5600.2	Packer wouldn't hold from 30'–51.5'			64'	Br at 28' Small back flow at 40'
RIGHT ABUTMENT	D/S D.H. 122	5958.1	20'–30'	18 gpm C of P	100 psi	150'	Br at 7' Lost return water at 138'
			35', 40', 45'	18 gpm C of P	150 psi		
	℄ D.H. 6	6077.7	15', 25	12 gpm C of P	25 to 75 psi	300'	Tight below 92' Bedrock on surface
			Lost water at 60'	C of P at 65' and 185'			
	℄ D.H. 123	6051.3	11' to 20'	17 gpm C of P	65 psi	56	Br at 5' Lost drill water at 11' and 19'
			20' to 30'	18 gpm C of P	65 psi		
	U/S D.H. 124	5725.1	20', 25', 30'	18 gpm C of P	63 psi	51'	Bedrock at 16'
			35'	18 gpm. C of P	85 psi		
			40, 45'	20 gpm C of P	100 psi		
	U/S D.H. 125	5656.4	20', 25', 30', 35'	18 gpm C of P	75 to 100 psi	53'	Br at 18' Water back flow 8 gpm at 45' and
							10 g.p.m at 48' No water return 27', 44'
TOE AREA River bed	D/S D.H. 109	5601.9	67'	17 gpm C of P	150 psi	72'	Br. at 37'
	D/S D.H. 119	5601.6	54', 65'	23 gpm	150 psi	65	Br at 29.5 Maximum lost of water 23 gpm.
	D/S D.H. 118	5582.6	60', 70'	16 gpm, 11 gpm.	150 psi	237'	" " 32' Hole tight below 70'
	D/S D.H. 116	5608.6	30'	20 gpm		63	" " 27'
	Lt. Abut. D.H. 107	5649.4	26', 31', 53', 60', 70'	19 g.p.m. C of P	150 psi	80'	Br at 5' Lost drill water at 53' and 65'
	Rt. Abut. D.H. 120	5719.8	10' to 20'	18 to 20 g.p.m. C of P	150 psi – 90 psi	125'	Bedrock on surface
			80', 100', 105'	20 to 18 g.p.m. C of P	135 to 150 psi		Lost drill water at 20'

Grout Result To Date

(1) Consolidation "B" Hole Grouting – Block 10 thru Block 16. Base of Dam

D.H. Location	No. of Holes	Max. Take for Single Hole	Total Take IN Sacs (cement)
Line 1, Heel	16	192 sacs	1055 sacs
" 3, near Axis	17	138 "	675 "
" 7, Toe	16	783	2812 "
" 8, near Toe	17	715	1899 "
Block 17, Heel line	6	At elev. 5632.5	149 "
Line 1	3	" " "	0 "
" 4	6	" " "	85 "
" 7	4	" " "	52 "
Block 2 Line A	3	Hole located on Br.	85 "
" B	4	" " " "	1640 "
" C	6	" " " "	412 "

(2) Foundation "A" Hole Grouting Drill Hole Divided into Thirds

No. of Hole	Length	Lower 1/3	Middle 1/3	Upper 1/3	Total
10-4	117'	100 sacs	0 sacs	4 sacs	104 s
10-8	121'	267 "	2 "	0 "	269 sacs
10-12	126'	2 "	1 "	0 "	3 "
11-2	137'	0 "	53 "	0 "	53 "
11-6	153'	20 "	14 "	3 "	37 "
11-10	168'	18 "	30 "	23 "	71 "
11-9	246'	271 "	3 "	3 "	277 "
12-1	252'	154 "	111 "	10 "	275 "
12-4	181'	409 "	"	26 "	435 "
13-2	235'	"	9 "	20 "	29 "
13-8	225'	11 "	0 "	5 "	16 "
13-4	250'	118 "	13 "	2 "	133 "
14-2	231'	17 "	1 "	0	18 "
14-3	242'	1228 "	103 "	10 "	1341 "
15-2	249'	18 "	45 "	3 "	66 "
15-3	216'	0 "	0 "	14 "	14 "
15-6	175'	0 "	127 "	3 "	130 "
16-1	179'	0 "	0 "	0 "	0 "
16-4	162'	727 "	428 "	16 "	1171 "
16-8	158'	26 "	0 "	0 "	26 "
16-10	155	232 "	16 "	8 "	256 "
Total Sacks:		3618	956	147	4724

Note: 12-4 means Block 10 Hole No 4

Note: Depending on location, depth, type and characteristics of hole grout pressures vary from 0 to 400 psi.

P.S.I. = pound per square inch T.D. = Total Depth g.p.m = gallons per minute C of P = Capacity of pump D/S = Down Stream (toe area) U/S = Up stream (heel area)
℄ = Near Dam Axis S.S. = Quartzitic Sandstone Sh = Shale Br = Bedrock, Uinta Formation Rt or Lt. Abut = Right or Left Abutment

FIGURE 24

prevalent than found in the tunnels. High takes in the B-holes occurred most often at the higher elevation adits where the relief joints would have been expected to be wider.

A previous study has been made to compare water losses in exploratory drill holes and grout takes at Flaming Gorge Dam by unidentified Bureau personnel. A tabulation of this study is presented on figure 24. To expand the study, the locations of the exploratory drill holes were plotted on figure 12, and a water-take profile was drawn on figure 25. A review of these data indicate: (1) high water takes and drill-water losses occurred throughout the valley bottom, although not in all exploratory holes; and (2) high grout takes occurred in B-holes near DH 118, DH 119, DH 109, DH 111, and DH 113 where high water takes had occurred. These data indicate that there is a relation between grout takes and water takes in exploratory drill holes.

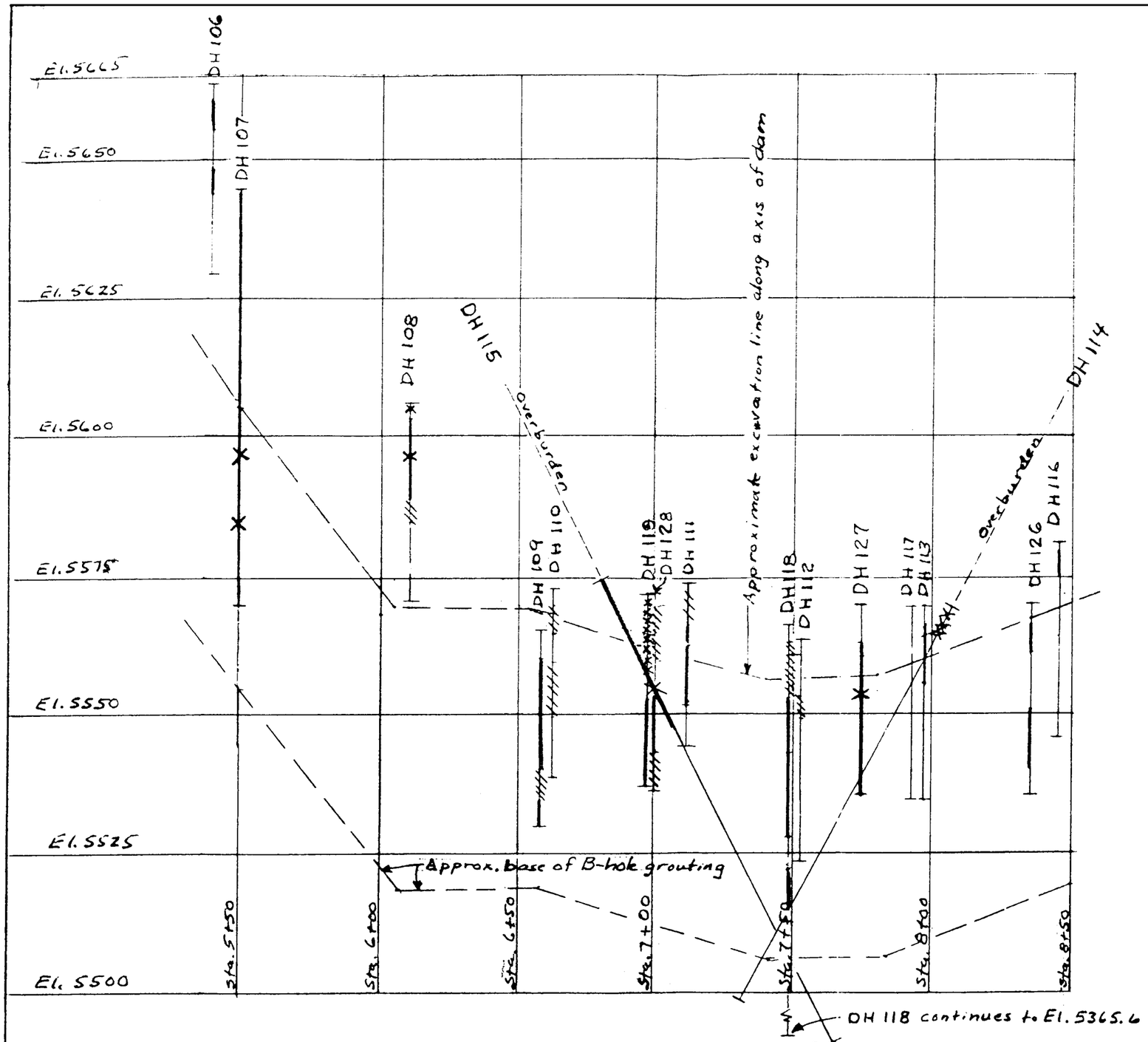
A similar review was made of the information shown on figure 22. The detailed logs for DH 5 and DH 122 were not available. High water takes occurred for the full depth of DH 120 and DH 121 except in the shale layers. Occasional high grout takes occurred in the B-holes (see figure 14) particularly in the near horizontal holes within the elevation ranges of DH 120 and DH 121.

17. Evaluation of dam uplift pressures and drain flows. The uplift pressures at the base of the dam and the drain flows are influenced by the following factors:

(1) Foundation geology.

(2) Depth of excavation.

(3) Grouting - A-hole and B-hole.



WATER-TAKE PROFILE - VALLEY BOTTOM
(LOOKING DOWNSTREAM)

EXPLANATION

- DH111 — PRECONSTRUCTION DRILL HOLE
- TOP OF ROCK WHEN HOLE WAS DRILLED
- ZONE OF ZERO OR LOW WATER TAKE (5 GPM OR LESS)
- COULD NOT HOLD PACKER
- ZONE OF HIGH WATER TAKE
- LOST DRILL WATER PARTIAL OR TOTAL
- BOTTOM OF HOLE

NOTES:

1. ALL HOLES PROJECTED ALONG RADIAL LINES TO AXIS OF DAM.
2. FOR LOCATION OF DRILL HOLES, SEE FIGURES 4 & 12.
3. CLASSIFICATION OF WATER TAKE BASED ON WATER-PRESSURE TESTS. HIGH WATER TAKES USUALLY EXCEEDED CAPACITY OF PUMP (18-20 GPM) AT MAX. PRESSURE OF 150 PSI.

SCALES:

VERTICAL 1" = 20'

HORIZ. 1" = 40'

CONTRACT NO. 2-07-DV-00148

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

FLAMING GORGE DAM

WATER-TAKE PROFILE
VALLEY BOTTOM

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

FIGURE 25

(4) Drainage holes and present condition.

(5) Base width of dam.

(6) Headwater and tailwater levels.

The design of the uplift pressure-measurement system is shown on figure 26. There are three lines of uplift pressure pipes. Line 1 is located about one-third of the way up the left abutment; line 2 is located in the valley bottom; and line 3 is located about one-third of the way up the right abutment. The 2-1/2-inch-diameter pipes were set in the concrete 6 inches above top of the rock. One and one-half-inch-diameter holes were drilled through the pipes at least 3 feet into rock. The holes in sets 1 and 3 were drilled after completion of foundation grouting in their area. The holes in set 2 were drilled after the first lift of concrete was placed on the foundation rock, which would have been after all B-hole grouting was completed in this area from top of rock; but before the A-hole grouting was accomplished, as the A-hole grouting was accomplished from the foundation gallery at the higher level. Typical details of the uplift pressure pipe system are shown on figure 27.

The uplift-pressure measurements from 1972 to 1981 are plotted on figures 28, 29 and 30. Profiles for lines 1, 2 and 3 through the dam showing uplift pressures for December 11, 1981, are shown on figures 31, 32 and 33. The scale for line 2 is only about one-half of that for lines 1 and 3.

A review of the plots indicates that the pipes upstream of the A-line (A row) have readings that parallel the pool readings and

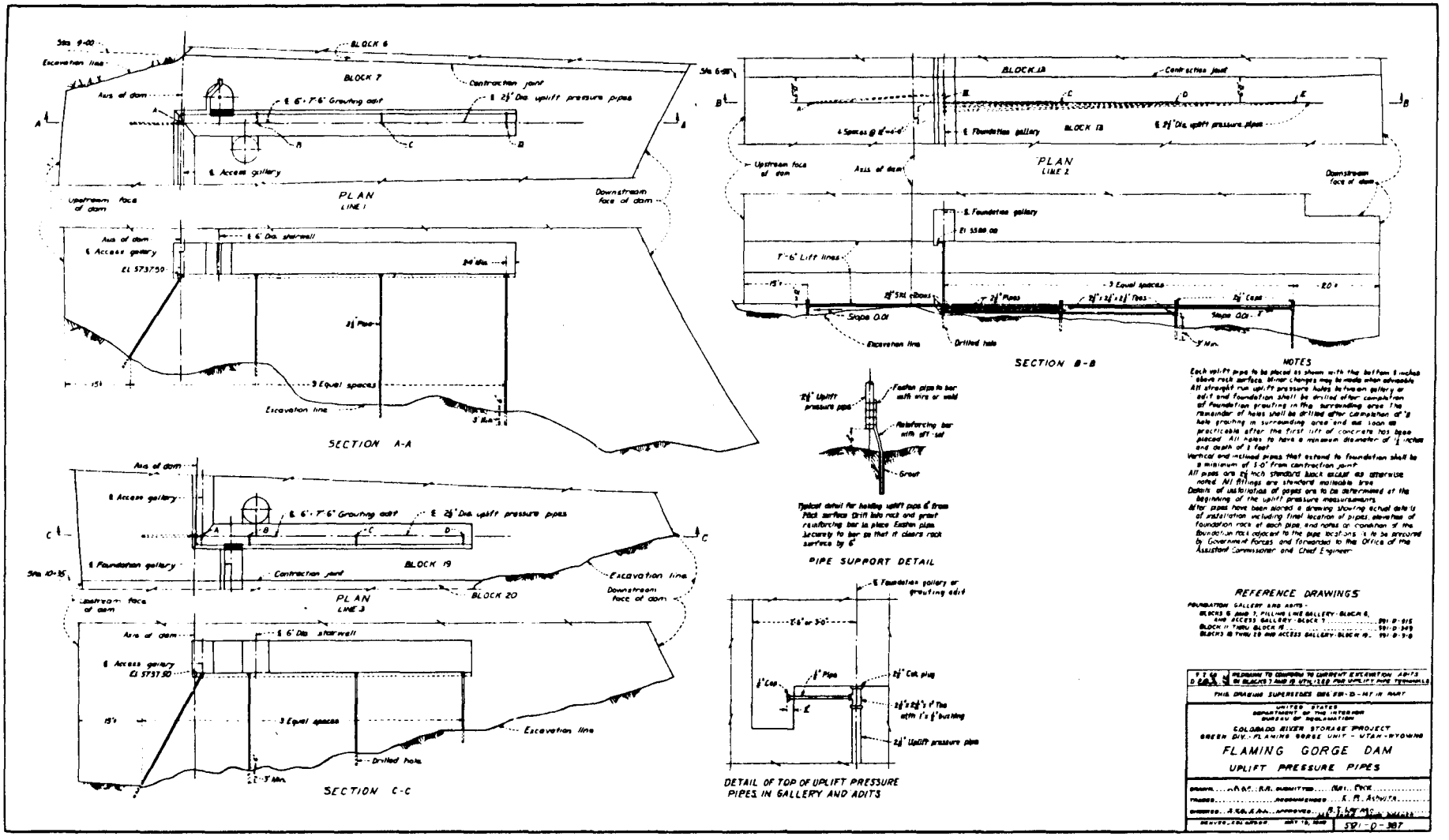
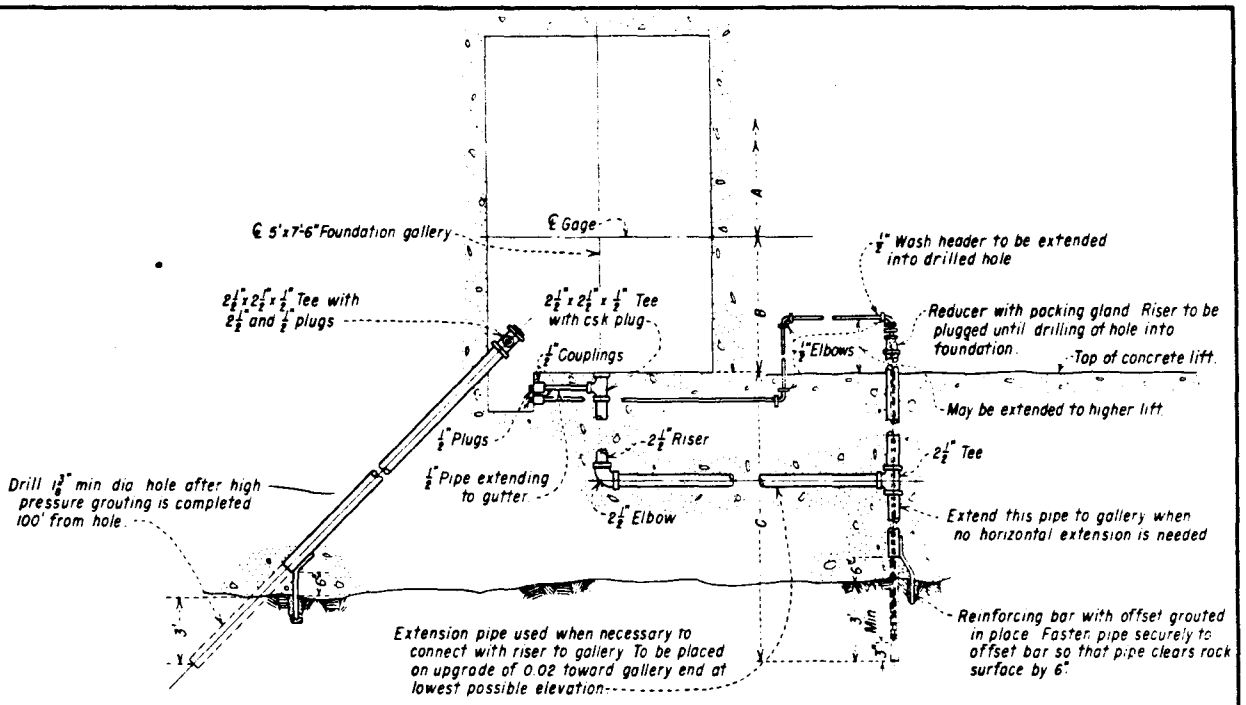
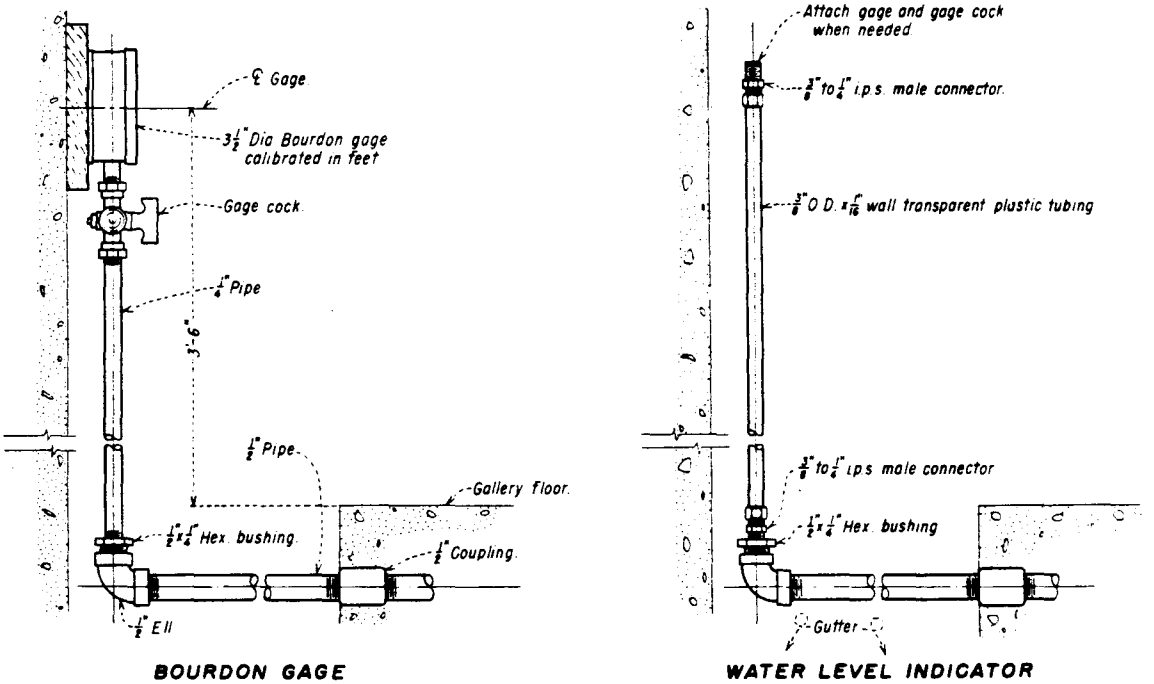

PLAN
LINE 1
SECTION A-A
PLAN
LINE 2
SECTION B-B
PLAN
LINE 3
SECTION C-C
PIPE SUPPORT DETAIL
DETAIL OF TOP OF UPLIFT PRESSURE PIPES IN GALLERY AND ADITS
NOTES
REFERENCE DRAWINGS
FLAMING GORGE DAM
UPLIFT PRESSURE PIPES
COLORADO RIVER STORAGE PROJECT
591-D-387

FIGURE 26



PIPE DETAILS



BOURDON GAGE **WATER LEVEL INDICATOR**

GALLERY INSTALLATIONS

NOTES

All pipes should be standard steel except as otherwise noted.
All fittings should be standard malleable iron except as otherwise noted.
Use gage when water level in pipe rises into region "A"; use water level indicator when water level in pipe is in region "B"; make soundings when water level in pipe is in region "C".
Bourdon gage capacity should be twice reservoir head for pipes upstream of foundation drains and two-thirds of reservoir head for pipes downstream of foundation of drains.
Insert 1" length of ¼" O.D. x 1/64" wall brass tube into ends of plastic tubing to strengthen tubing for compression connectors.
Wash header to be used to wash uplift pressure pipe during proximity grouting.

D 12-26-62 ADDED WASH HEADER, FITTINGS AND RELATED NOTES. REVISED SIZE AND SHAPE OF GALLERY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
STANDARD DESIGNS

STRUCTURAL BEHAVIOR EQUIPMENT
UPLIFT PRESSURE PIPE SYSTEM
TYPICAL DETAILS

DRAWN J.T.R. SUBMITTED J.J. Hammond
TRACED W.W.G. RECOMMENDED K.B. Keener
CHECKED [signature] APPROVED W.E. Blomgren
ACT. CHIEF ENGINEER
DENVER, COLORADO, MAY 18, 1953 40-D-5291

FIGURE 27

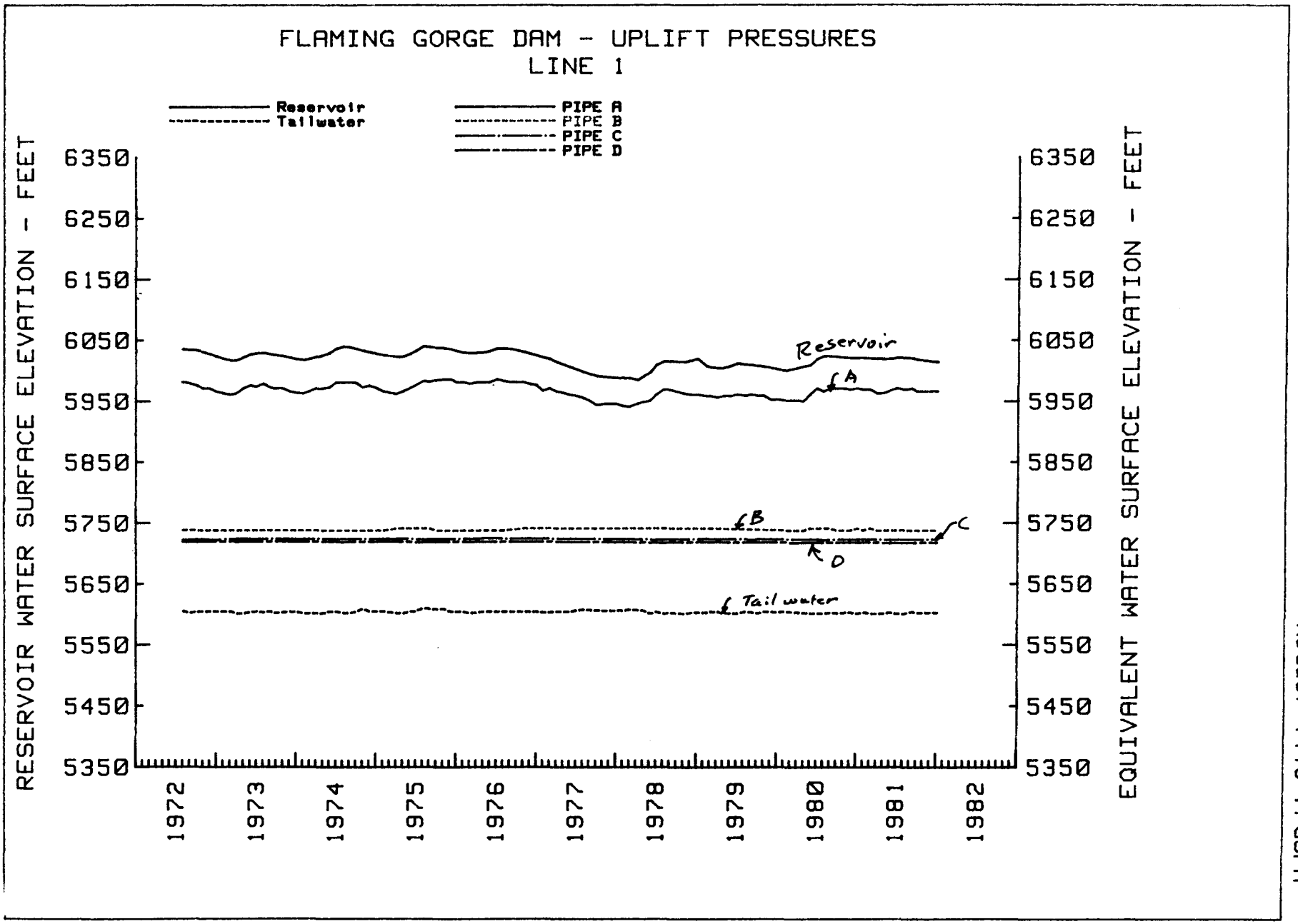


FIGURE 28

ENGINEERING & RESEARCH CENTER - STRUCTURAL BEHAVIOR SECTION

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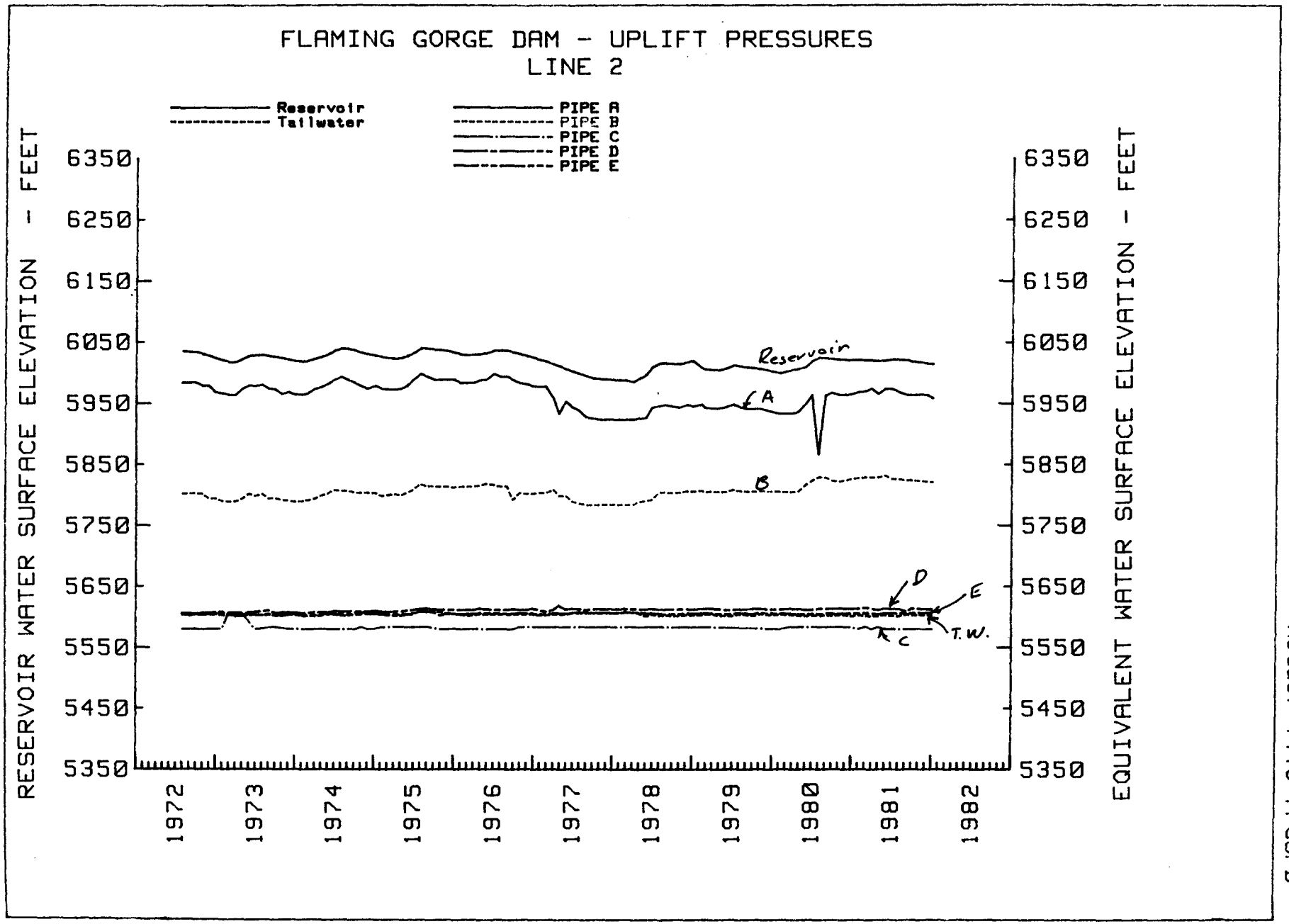

FIGURE 29

ENGINEERING & RESEARCH CENTER - STRUCTURAL BEHAVIOR SECTION

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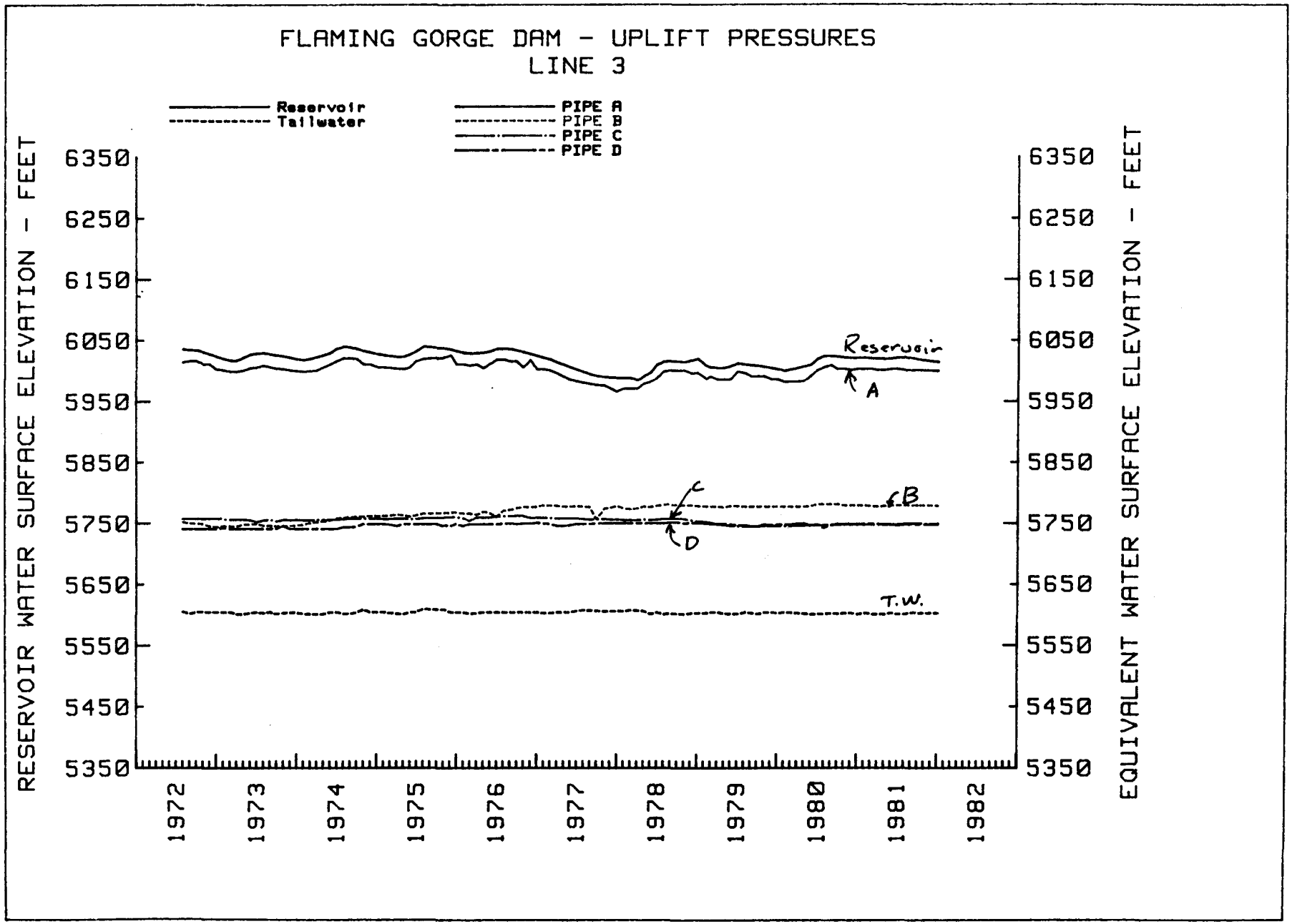

FIGURE 30

ENGINEERING & RESEARCH CENTER - STRUCTURAL BEHAVIOR SECTION

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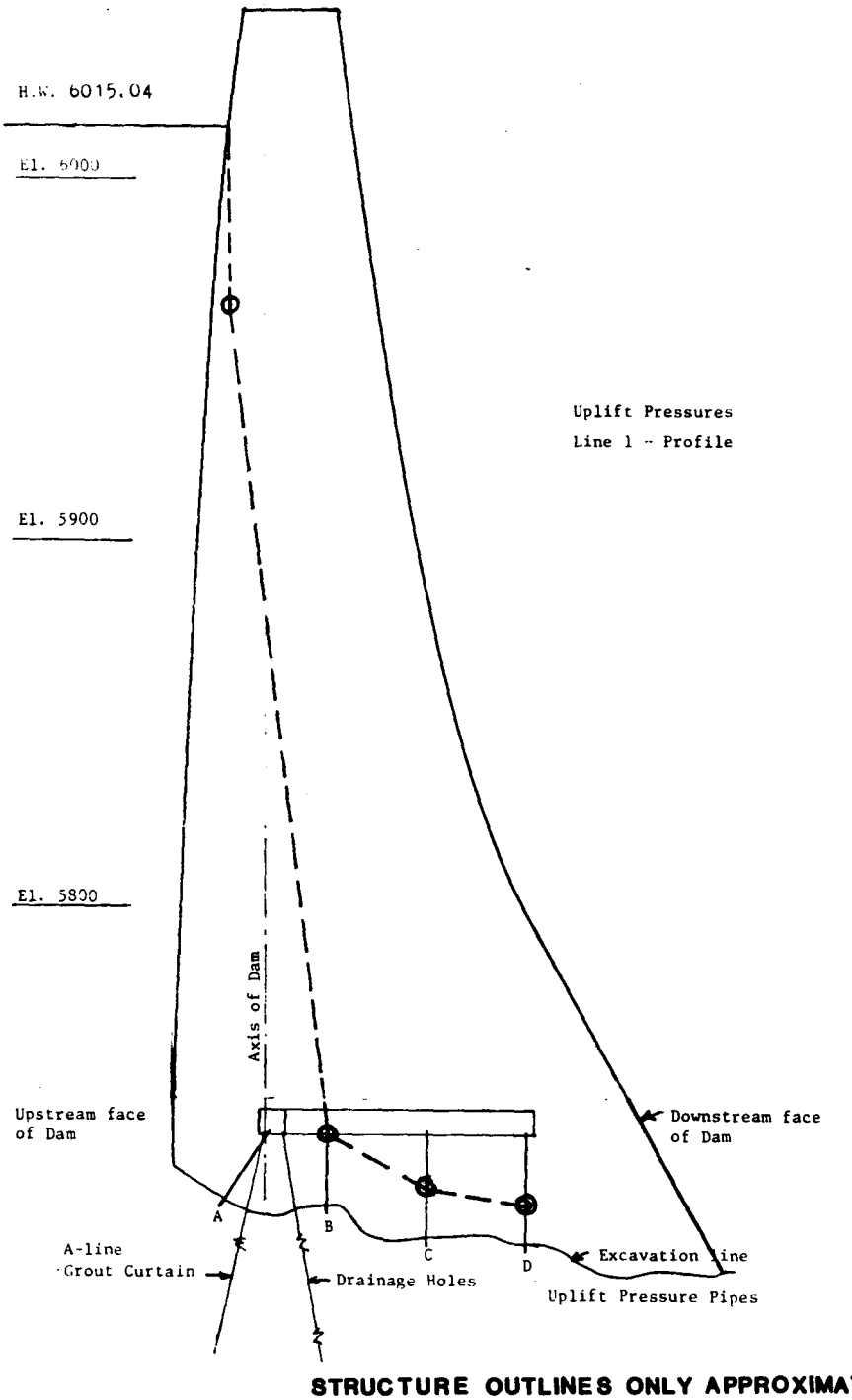


STRUCTURE OUTLINES ONLY APPROXIMATE

CONTRACT NO. 2-07-DV-00148

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

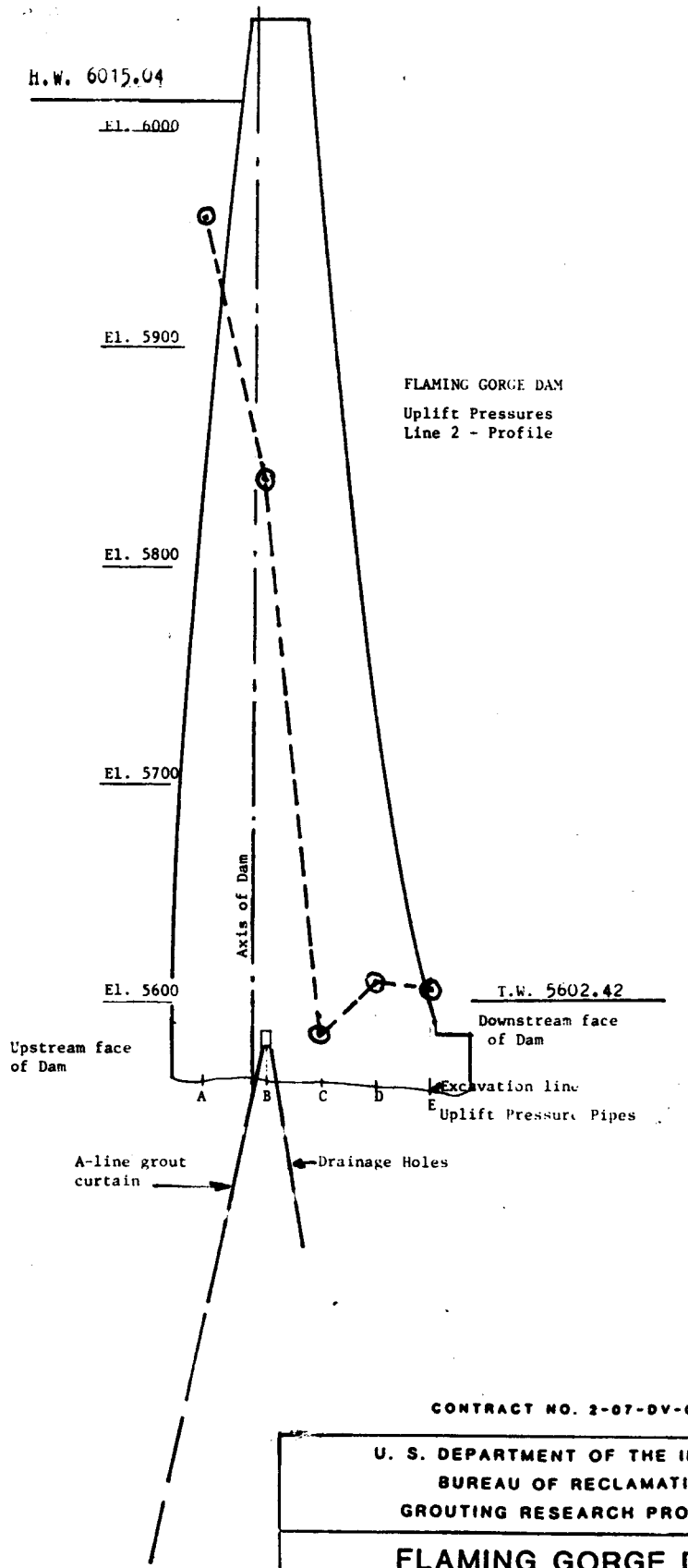
FLAMING GORGE DAM

UPLIFT PRESSURES
LINE 1 PROFILE 12-11-81

REF. DWG. 591-D-387

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

FIGURE 31



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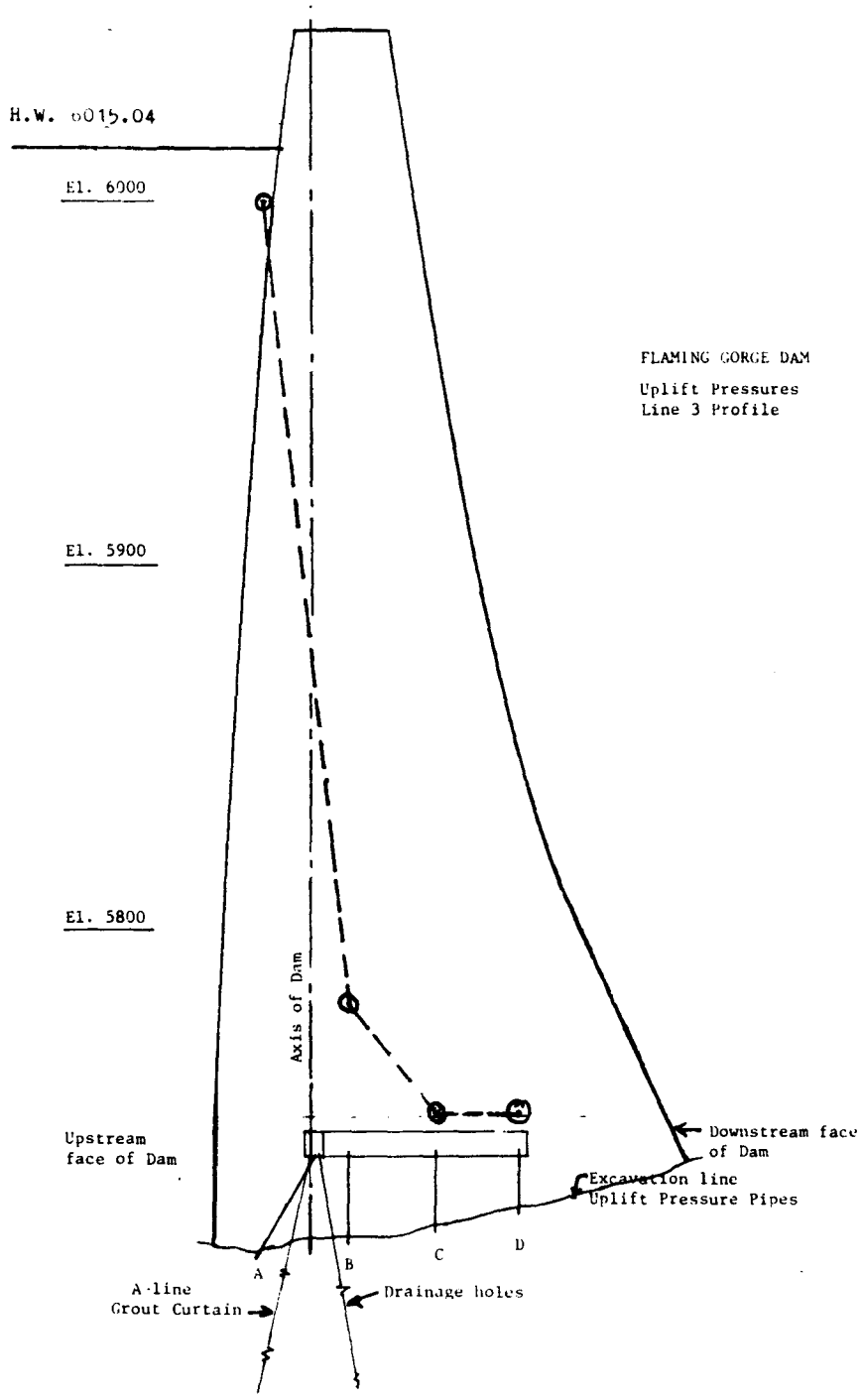
GROUTING RESEARCH PROGRAM

FLAMING GORGE DAM
UPLIFT PRESSURES
LINE 2 PROFILE 12-11-81

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

FIGURE 32



FLAMING GORGE DAM

Uplift Pressures
Line 3 Profile

CONTRACT NO. 2-07-DV-00148

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

FLAMING GORGE DAM

UPLIFT PRESSURES

LINE 3 PROFILE 12-11-81

PREPARED BY: CLAUDE A. FETZER

JULY 1982 CONSULTING GEOTECHNICAL ENGINEER

FIGURE 33

are from about 20 to 60 feet below the pool levels. As these pipes are located only 15 feet from the upstream side of the dam, these readings are as expected. The next downstream pipes (B row) are located downstream of the A-line and of the drainage holes except in line 2 where the B pipe is between the A-line and the drainage holes. The readings for the B pipes are markedly lower than for the A pipes - 277 feet for line 1, 140 feet for line 2, and 220 feet for line 3. The reduction from the B pipes to C pipes is also substantial - 15 feet for line 1, 240 feet for line 2, and 30 feet for line 3. The C and D readings for lines 1 and 2 are almost identical; whereas, there is a 32-foot rise from C to D in line 2, and a slight reduction of 7 feet from D to E in line 2.

The substantial reduction between pipes B and C in line 2 is attributed to the influence of the drainage holes. The elevation of reading C is almost equal to the elevation of the discharge point of the drainage holes in the foundation gallery. The slightly higher readings found in D and E further downstream in line 2 probably indicate that pipe C represents a limited area where the pressure is at the elevation of the drain which is even below tailwater. Other areas in the valley bottom along a line parallel to pipe C could have readings at or above the reading in pipe D. The low reading in pipe C of line 2 not only indicates the importance of installing drainage holes but also the real need to keep the drain holes cleaned out at all times.

One of the trends found on figure 30 is that differential water levels between pipes A and B of line 3 have decreased from about 260 feet in 1972 to 220 feet in 1981. The decrease in

differential has been due to a rise in the water levels of pipe B. This change could be due to leaching out of the grout curtain or to clogging of the drainage holes; however, there has not been an increase in the water levels in pipes C and D. Although the trend is not changing at a rapid rate, it should be observed and evaluated.

On figure 31, the water levels in pipes C and D are below the elevation of the drainage-hole discharge point shown for the foundation gallery. It is considered that the water levels found in C and D may be influenced by drainage holes discharging at a lower elevation in the foundation gallery or from seepage discharging through joints or fractures in the rock downstream. It would probably be possible to lower the water levels in pipes C and D in both lines 1 and 3 by drilling drainage holes from the abutments downstream of the dam. However, the present water levels are undoubtedly below the uplift values assumed in design and the need for additional drainage holes can probably not be justified.

The uplift pressure measurements are essentially made at top of rock and reflect the uplift pressure at the base of the dam. However, the uplift pressures at depth below top of rock are not known; and if stability analyses were required for weak planes below top of rock, uplift pressure measurements along those planes would be needed. Sealed piezometers would be required to measure the uplift along a selected weak plane below top of rock.

The drain flows for 1977 to 1981 are plotted on figures 34 and 35, and the locations of the measuring stations are shown on figure 36. The flows from the left and right galleries are only

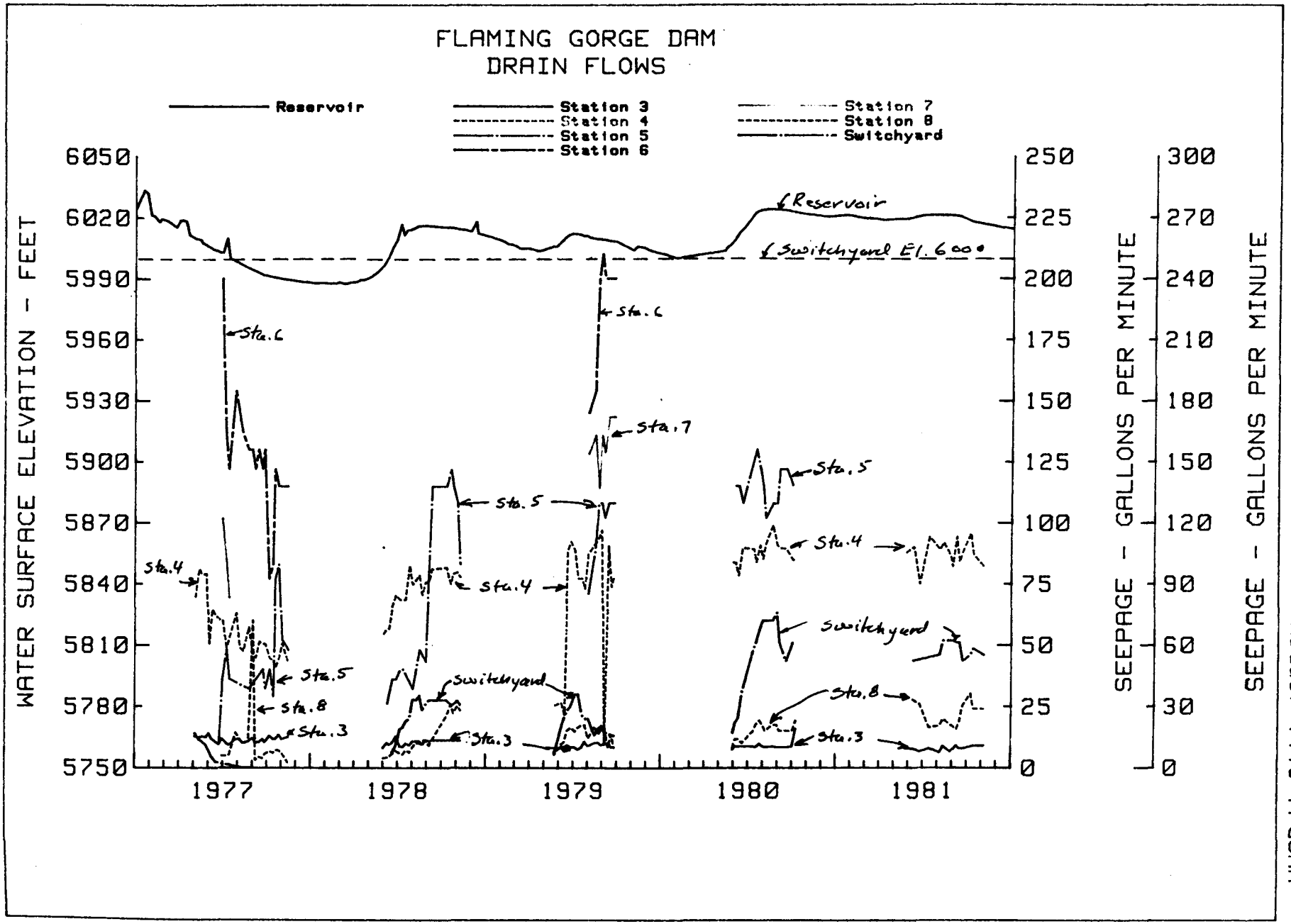


FIGURE 34

ENGINEERING & RESEARCH CENTER - STRUCTURAL BEHAVIOR SECTION

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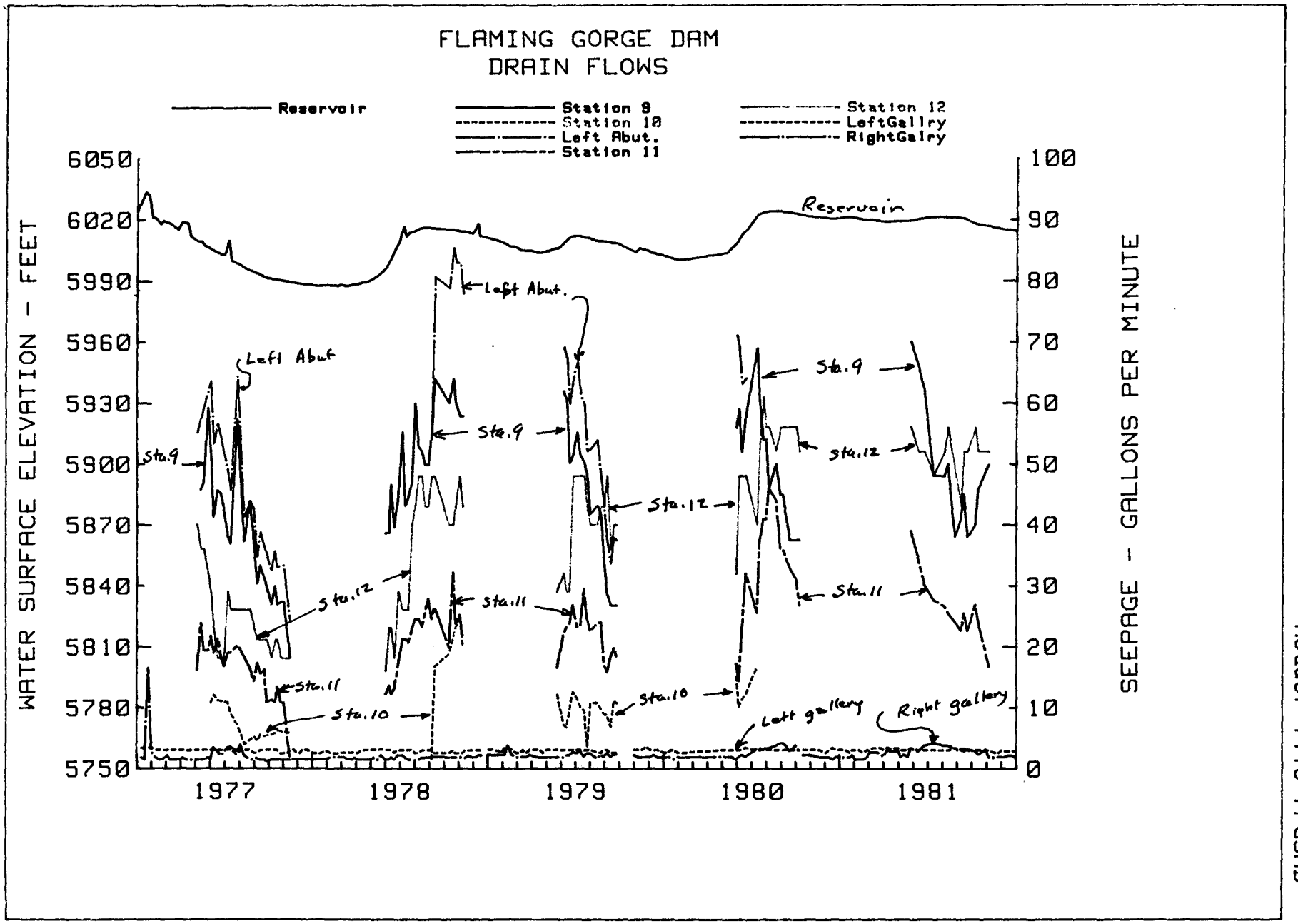
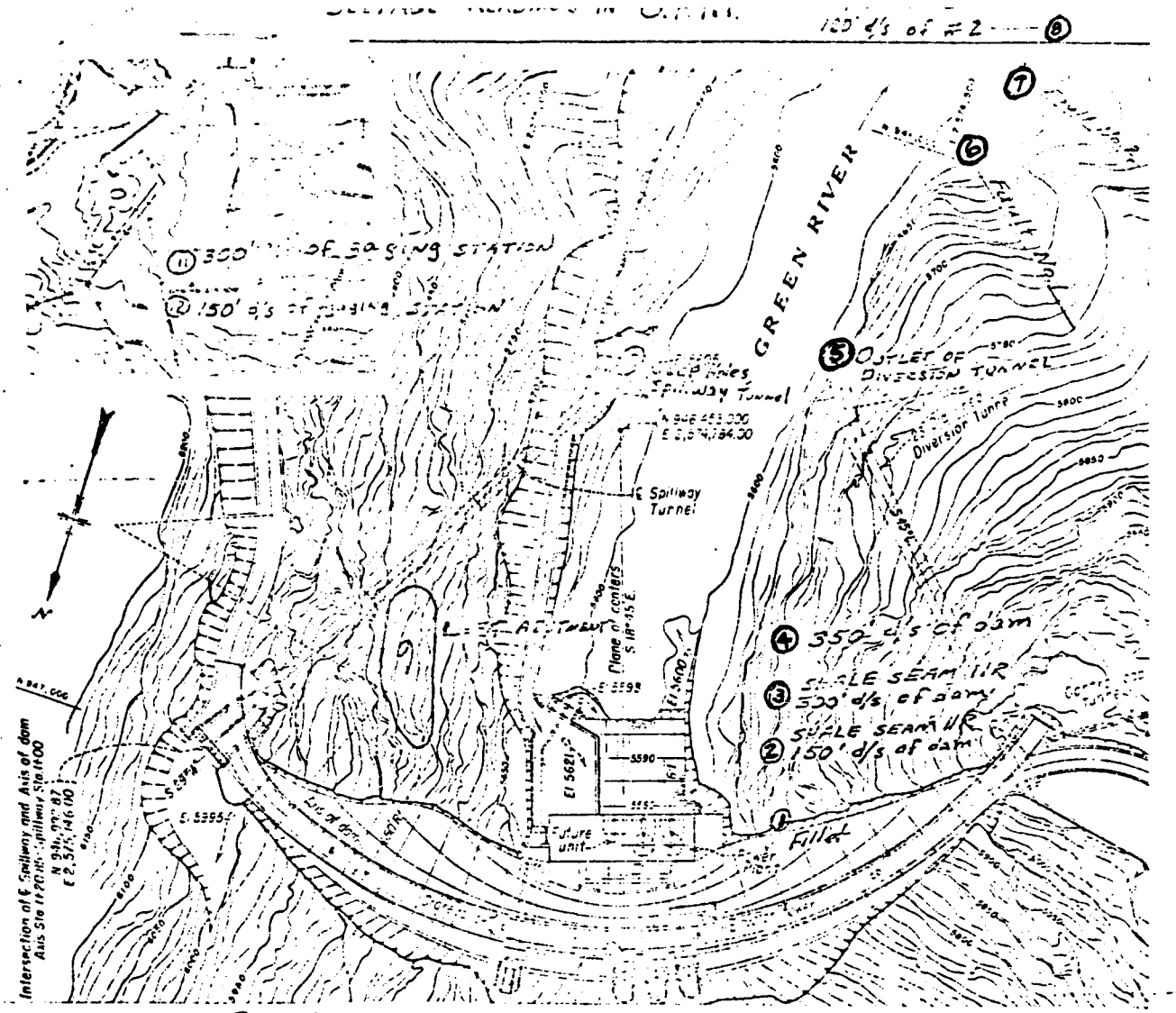


ENGINEERING & RESEARCH CENTER - STRUCTURAL BEHAVIOR SECTION

FIGURE 35

Header file 7FGSMB



Station	Date 3-2-82 Res. 6010.80	Date 3-9-82 Res. 6010.65	Date 3-17-82 Res. 6010.47	Date 3-23-82 Res. 6010.57	Date 3-31-82 Res. 6010.71
1					
2					
3					
4					
5					
6					
7					
8					
Switchyard					
Right Abut. Totals					
9					
10					
Left Abut. Totals					
11					
12					
Galleries left	2.52	3.02	2.52	3.37	3.37
Galleries right	2.19	2.19	2.19	2.19	2.19

TCE & Shale!

NOTE: All readings are in GPM-measured

FIGURE 36

about 2 to 3 gpm, which are very low flows for a head differential of about 400 feet. These data indicate that the overall dam foundation is relatively impervious. The B-hole grouting, which covered the entire valley bottom on 20-foot centers to a depth of 50 feet, undoubtedly contributed substantially to the overall tightness of the valley bottom foundation, and the high takes in the B-hole grouting in the abutments also contributed to the tightness of the rock along the abutment contacts.

The effectiveness of the switchyard barrier curtain (see figures 3, 4, 6 and 21) is difficult to measure. The elevation of the switchyard has been plotted on figure 34. The available data on the plots indicate that seepage is measured at the switchyard only when the reservoir level is above the switchyard, and the flows increase rapidly when the reservoir level rises above El. 6,000. Even if the barrier curtain were absolutely tight, flow around the curtain could occur as the drill holes along and beyond the end of the curtain (DH-140, 157, 161 and 162) had frequent drill-water losses and high takes in the percolation tests. Flow could also occur across the peninsula from Cart Creek. Additional grouting along the barrier curtain may not reduce the flows into the switchyard as the water table in the peninsula is fed from three sides and probably rises and falls with the reservoir level. Installation of a few open standpipes across the curtain and beyond the end of the curtain is probably warranted to check the water-table slope and fluctuations with pool.

A review of the plots for the other stations on figures 34 and 35 does not indicate that the flows are increasing with time for equivalent reservoir levels.

It is concluded that the grouting and drainage systems have adequately controlled the foundation seepage and uplift pressures at the base of the structure. However, open tube and sealed piezometers are needed to obtain data on water levels and pore pressures at various levels in the right abutment in connection with stability studies for the abutment. These data are especially needed in the winter time when the downstream seepage outlets may be frozen.

VI. RECOMMENDATIONS

18. Flaming Gorge Dam.

(1) Foundation drains be cleaned at appropriate intervals.

(2) Sealed piezometers be installed about El. 5,530 below the first shale layer in the valley bottom.

(3) Under Phase II for determination of grout deterioration, drill holes from lowest grouting gallery at locations where B-hole grout takes were high between Stations 7+20 and 7+80.

(4) Install open pipes from top of blocks 25 and 26 to measure the water levels in the sandstone above and below shale zone 6R, and install sealed piezometers in shale zones 6R and 7R. This recommendation is made to determine if additional drainage or other measures are needed to assure the stability of this abutment during a design earthquake and/or at maximum water surface.

(5) Install open stand pipes to measure effectiveness of barrier curtain.

19. Other large concrete dams.

(1) Preconstruction geologic investigations be thoroughly completed for final dam design including sufficient abutment adits to define excavation line and sufficient drill holes with adequate depth to define limits of grouting and drainage.

(2) Pumps used in pressure testing of exploratory holes during preliminary investigations and in the design phase be of sufficient capacity to give a true picture of foundation conditions. A hole accepting 20 to 23 gallons per minute at the

capacity of the pump may well represent a hole (or zone in a hole) that would actually accept 50 to 75 gallons per minute if tested with a pump of adequate capacity. Bid quantities for drilling footage and cement could be better estimated if pressure tests were accurate and reliable.

(3) Preconstruction geologic report include an evaluation of geologic conditions as related to the design of the grouting and drainage including relationship of water-pressure tests, bedding, jointing, etc. to depth, angle and spacing of holes and estimate of grout takes.

(4) Specifications include specific minimum requirements for mixing and grouting equipment, range of grout mixes, and methods of sounding and final backfilling of completed grout holes.

(5) Geologist and engineer establish maximum safe pressures for grouting for each reach of the dam and for each packer setting.

(6) Results of grouting operations be continuously evaluated by the site geologist and engineer to determine appropriate changes and the need for additional investigations. The design must continue through construction on all projects.

APPENDIX

APPENDIX A

Dutch John, Utah
June 8, 1960

Mr. Grant Bloodgood
Assistant Commissioner and Chief Engineer
Bureau of Reclamation
Denver Federal Center
Denver 25, Colorado

Dear Mr. Bloodgood:

Pursuant to instructions contained in a letter from Mr. B. P. Bellport, Acting Assistant Commissioner and Chief Engineer, dated May 12, 1960, addressed to each member of the Flaming Gorge Consulting Board Messrs. Hammond, Vanderwilt, and Burwell assembled in Denver on the morning of June 5 where they were joined by Mr. E. R. Schultz, Head, Concrete Dam Section. From Denver they were flown to Dutch John, Utah, via Bureau plane arriving in the early afternoon. There they were joined by the following personnel of the Bureau:

Denver Office:

L. G. Puls, Chief Designing Engineer

W. H. Irwin, Chief, Engineering Geology Branch

Region 4 Office:

F. M. Clinton, Regional Director

W. F. Peterson, Regional Engineer

J. N. Murdock, Regional Geologist

Flaming Gorge Project Office:

Jean R. Walton, Project Construction Engineer

James R. Granger, Assistant Project Construction Engineer

Herbert W. Senne, Engineer

Darrel W. Hansen, Engineer

William R. Groseclose, Engineer

George T. L. Wongwai, Project Geologist

Sylvester J. Turley, Engineer (6-7-60 only)

Messrs. Julian Hinds and R. E. Davis, the other members of the Board, were unable to attend this meeting. Therefore, the "Board" as used in this report refers only to the undersigned members.

During the afternoon of June 5th the Board made general observations of the damsite area from the upstream cofferdam and from the top of the left abutment.

On Monday, June 6th, following a briefing by Messrs. Walton and Puls, the Board began its examination of the damsite area. During the morning, detail inspections were made of the topography and bedrock conditions of the right abutment with special attention given to the shale beds and jointed condition of the foundation rock exposed in the right abutment keyway. Also during the morning an inspection was made of the river section where only limited areas of bedrock have been uncovered.

During the afternoon of June 6th the upper part of the left abutment keyway, including the foundations for the thrust block and the spillway intake were examined in detail, giving special attention to the affects of the shale beds on foundation stability. A visit was also made to the downstream cofferdam where the condition of the valley walls downstream of the keyways and the alignment of the spillway tunnel were observed.

During the morning of June 7th the Board visited the aggregates processing plant on Henry's Fork and examined the partially stripped aggregate deposit area.

The afternoon of June 7th and all of June 8th were spent in the preparation of the report that follows. The subjects suggested for the Board's consideration in your letter of May 24, 1960, to Mr. Julian Hinds, are dealt with herein in the order in which they are listed in the letter.

1. Examine the completed foundations for the dam and powerplants and appraise their adequacy

The Uinta formation, the foundation rock, is well exposed in the keyway excavations and, as shown in surface exposures, consists predominently of hard, strong quartzite and a number of red shale beds. The excavations for the keyways brought out the presence of more and thicker red shale beds than had been anticipated. Also, the open vertical joints extended to greater depth than had been expected. Keyway excavation deeper then had been planned originally was necessary in both abutments because of the greater quantity of shale and the greater depths of joint openings, and these conditions will require suitable treatment as brought out under item two.

The Board desires to call attention to the fact that the value of the modulus of elasticity used in computing the arch stresses was 2,000,000 p.s.i. Although this value was approved by the Board in its report of October 23, 1958, it is now of the opinion that this value is too high. In view of the conditions disclosed by the keyway excavations,

the Board is of the opinion that the modulus of elasticity might well be as low as 1,000,000 p.s.i.

The rock formations and structural conditions exposed at the base of the keyway excavations can be expected to extend over the area of the dam and powerhouse foundation in the floor of the canyon which had not been uncovered.

In the left abutment keyway, adjacent the spillway intake excavation and in the thrust block area, open joints and fractures indicate movement of quartzite blocks on shale bedding. It is believed that blasting and water saturation of the shale has contributed to the loosening and creep of rock that has occurred. The Board agrees with Bureau engineers that the rock which has been unduly loosened in the thrust block area should be removed especially where down-dip rock support is lacking. In the area above and adjacent the spillway intake, where the presence of a shale bed and joints suggest possible movement, prestressed rock bolts and other means for stabilizing the rock should be employed.

The relatively close proximity of the planned spillway tunnel to the open canyon wall is a matter of concern. Heavy blasting during construction of the spillway conceivably could loosen rock masses in the canyon wall so as to endanger not only the spillway tunnel but also structures in the canyon below. The Board recommends close supervision of construction of the spillway and especially of the drilling and blasting patterns.

In the right abutment above shale bed 6R at approximate elevation 5945, the rock mass which will support the arch is relatively thin. While the Board is of the opinion that his mass is capable of resisting the arch thrust, it is important that a careful analysis be made to determine its stability under the condition of maximum loading including earthquake. In addition, studies should be made to determine what measures could be undertaken to prevent any future deterioration of the mass that might reduce its stability.

With the exceptions noted and with the treatment outlined under item two, the Board is of the opinion that the dam and power-plant foundations will be satisfactory and that the mass strength of the foundation rock will be adequate to support the loads and stresses of the dam as planned.

2. Review proposed treatment of shale zones in abutment keyways and foundation grouting program

The excavation of the abutment keyways for the Flaming Gorge Dam has disclosed bedrock conditions that are less favorable than those that were known or could be anticipated from the results of the exploratory investigations completed at time of the Board's previous meeting in October 1958. Not only has a greater number of shale beds of appreciable thickness been encountered than was expected, but wide zones of creep of substantial thickness, in which the rock joints have been opened appreciably, have been found in both abutments, necessitating considerably deeper excavation than was originally planned. These discoveries made necessary a more elaborate plan of foundation

treatment than was originally proposed.

The proposed treatment of the shale beds, outlined in the book of technical data furnished the Board, provides for concrete cutoffs in the thicker shale beds at the heel and toe of the dam. The depth of the cutoffs into the abutments is to be determined by the character and thickness of the beds and the reservoir head to which they will be subjected. Eight such beds, three in the left abutment and five in the right, having thicknesses of from two to ten feet, are to be treated. The proposed minimum depths of the cutoffs vary from 25 feet in the upper part of the abutments to 75 feet near the base of the abutments.

The Board considers this treatment of the major shale beds suitable and desirable and desires to point out that it will also provide additional information on foundation conditions inside the abutments which will be useful in planning the program of consolidation grouting. The Board recommends that the cutoff drifts in the upper part of the abutments be increased from a minimum depth of 25 feet to a minimum depth of 50 feet.

In addition to the cutoff treatment, the Board recommends that the keyway slopes at the shale crossings be made vertical so that no part of the dam will rest directly on shale.

The open-jointed condition of the foundation rock in the abutment keyways necessitates a thorough job of consolidation grouting over the entire area of the keyways. As the abutment rocks in their

jointed condition, would be difficult to consolidate and are sensitive to grout pressures if unconfined, the Board is in complete agreement with the Bureau's proposal to accomplish the consolidation grouting from transverse adits located in the dam at 50-foot intervals of elevation. This method will increase the costs substantially, but is considered absolutely necessary.

The Board is of the opinion that the consolidation grouting should extend to a minimum depth normal to the foundation surface of 50 feet and that the spacing of the grout holes should be such as to provide a thorough degree of consolidation.

Concerning the curtain grouting, the Board is of the opinion that it will be necessary to extend it to a considerably greater depth than was originally proposed, particularly in the upper part of abutments. This curtain should be extended to include the narrow right abutment spur and saddled.

3. Comment in general terms on the adequacy of the strength and durability obtained in laboratory tests of concrete

The Board has reviewed the results reported in the book of technical data on the tests of the limited number of laboratory specimens prepared with cement from Devils Slide, Utah, Idealite pozzolan from Laramie, Wyoming, and aggregates from the Henry's Fork deposit. While the Henry's Fork aggregates are far from ideal, even when beneficiated by the heavy media process, the test results indicate that with beneficiation of the No. 8 through 1½-inch fractions, no difficulty should be experienced in producing a concrete

of specification strength using this aggregate, and containing two sacks of Devils Slide cement and one sack of Idealite pozzolan.

The Board notes the increase in compressive strength obtained by the use of a water-reducing agent and would have no objection to the use of such an agent in order to reduce the cement requirement by a small amount.

The Board notes the detrimental affect of pozzolan on durability indicated by the limited number of tests. Should further testing of large specimens bear out the preliminary results, the Board would agree that consideration should be given to omitting the pozzolan in the exterior mixes.

Respectfully submitted,

/s/ John W. Vanderwilt

/s/ Edward B. Burwell, Jr.

/s/ John H. Hammond, Acting Chairman

UNITED STATES
DEPARTMENT OF THE INTERIOR
Bureau of Reclamation
Commissioner's Office
Building 53, Denver Federal Center
Denver 2, Colorado

June 15, 1960

Mr. Grant Bloodgood
Assistant Commissioner
and Chief Engineer
Bureau of Reclamation
Denver, Colorado

Dear Mr. Bloodgood:

As you were advised in advance, I was unable to attend the Board of Consultants' meeting for Flaming Gorge Dam at Dutch John, Utah, on June 5, 1960. Accordingly, by mutual agreement, I visited the site on June 13 and 14, and conferred, in Denver, on June 15, with Mr. John H. Hammond and various members of the Bureau's office staff.

Preparatory to this visit I was furnished a copy of the Technical Data Report of May 16, 1960; also, a copy of the letter report, dated June 8, 1960, covering the visit of other Board Members, June 6-8, 1960.

These documents were carefully reviewed and the site was inspected with the following personnel:

Denver Office:

Grant Bloodgood, Assistant Commissioner
and Chief Engineer

Max Ford, Designing Engineer

Region 4 Office:

C. H. Carter, Assistant Regional
Director

Flaming Gorge Project Office:

Jean Walton, Project Construction Engineer
James R. Granger, Assistant Project Construction Engineer
George T. L. Wongwai, Project Geologist
S. J. Turley, Engineer
Others from time to time

I was furnished excellent topographic and geological data in the form of drawings and photographs, and was given full assistance in viewing all features of the project.

I have the advantage of seeing further work accomplished at the site subsequent to the June 6-8 visit of the other Board Members, and of reviewing stress studies made in Denver as suggested at the June 6-8 meeting.

After careful review of the site and study of available data, I reach the following conclusions:

1. Although the basic foundation rock is of excellent quality, it is fractured and jointed to a greater depth than originally expected. This has resulted in greater depth of excavation than originally contemplated, which will cause an increase in cost, but in no way affects the safety of the dam when completed in accordance with present plans.

2. The shale bands, revealed by excavation to be more prominent than originally contemplated, are not of sufficient magnitude to seriously affect stresses, and their orientation is not such as to endanger abutment sliding. When treated as recommended in the Technical Data Report and as suggested in the Board Report of June 8, they will involve no hazard to the safety of the dam.

The Board Report of June 8 is endorsed in all respects with the following suggested clarifications:

1. At the bottom of Page 3 and the top of Page 4 there appears the sentence: "In view of the conditions disclosed by the keyway excavations, the Board is of the opinion that the modulus of elasticity might well be as low as 1,000,000 psi."

It is understood that this was not intended as a recommendation that the designs be changed to this basis immediately, but rather that the effect of such a possibility on dam stresses be evaluated. With this interpretation, I full agree.

Partially completed computations discussed in Denver June 15, 1960, indicate that the affect on arch stresses will be moderate and generally favorable at critical points.

2. In the comment near the middle of Page 4 on the anchorage of unstable blocks on the left abutment, it should be pointed out that these unstable blocks are entirely outside the dam abutments, and that their stabilization is for subsidiary purposes. After careful inspection of this area, I fully concur in the proposed anchorage.

3. Subject to the above clarification, the Board Report is fully endorsed both as to matters herein discussed and to others not mentioned.

Sincerely yours,

/s/ Julian Hinds

Julian Hinds

I concur:

Edward B. Burwell, Jr.

John J. Hammond

John W. Vanderwilt

APPROVED: June 27, 1960

/s/ Grant Bloodgood
Grant Bloodgood
Assistant Commissioner
and Chief Engineer

October 16, 1961

Mr. Grant Bloodgood
Assistant Commissioner & Chief Engineer
U. S. Bureau of Reclamation
Denver Federal Center
Denver 25, Colorado

Dear Mr. Bloodgood:

To begin the assignment outlined in your letter of August 25, 1961 the members of the Board of Consultants for Flaming Gorge Dam assembled at Dutch John, Utah late Thursday afternoon October 12, 1961. The following four days were devoted to field inspections and analysis of information contained in the book of technical data, furnished by your office, pertaining to the items listed on the sheet enclosed with your letter and an additional item presented by Mr. O. L. Rice to the Board.

Several members of the Bureau staff assisted in the course of the work by providing information and supplementing data requested by the Board. They included among others:

O. L. Rice, Acting Chief Designing Engineer, Denver, Colorado

C. H. Carter, Assistant Regional Director, Region 4, Salt Lake City, Utah

E. R. Schultz, Head, Concrete Dams Section, Denver, Colorado

J. R. Walton, Project Construction Engineer, Flaming Gorge Unit, Dutch John, Utah

F. S. Dallon, Assistant Project Construction Engineer,
Dutch John, Utah

W. A. Behling, Office Engineer, Dutch John, Utah

R. C. Borden, Chief, Civil Engineering Division, Dutch John, Utah

G. Wongwai, Project Geologist, Dutch John, Utah

S. J. Turley, Engineer, Head, Laboratory & Aggregate Plant Branch,
Dutch John, Utah

After a short meeting with Mr. Walton, in his office, Thursday afternoon, it was decided to postpone briefing until Friday morning in favor of a quick trip to the dam site for general orientation and to observe the stage of construction.

Friday morning, October 13, the Board members convened in the Project Construction Engineer's office. Mr. Walton outlined the status of construction progress and Mr. Rice gave a brief resume' of the design and related studies of the Flaming Gorge Dam project. The items of the work assignment were reviewed and original item two was subdivided for clarity making a total of eight items to be considered.

Following the briefing, the Board proceeded to the right abutment to inspect the location of the proposed concrete retaining wall designed

to buttress the overhang of the cliff and protect shale bed 6R from further weathering; to observe the cutoff concrete backfills along this shale bed in the abutment; and to study the parking area and switchyard where a grout curtain is planned. In the afternoon the Board inspected the cutoff drifts along shale bed 9R in the right abutment; observed mass concrete placement and compaction practices; and inspected the completed excavations and rock bolting for the thrust block and spillway intake structure.

On Saturday morning inspections were made of the aggregate processing plant, the recently placed concrete lining in the spillway outlet tunnel, and the rock formation exposed in the tunnel beyond the lined section. Also the powerplant structure was visited. Board member Davis also inspected the batching and mixing plant.

Sunday morning a cableway skip was provided for a close view and inspection of both abutments. The skip was slowly lowered or raised, close to each abutment and as requested stopped to allow for more detailed observation and discussion of the features being observed. Special attention was given to the right abutment, above shale bed 9R. These observations from the skip provided a far better perspective of rock structures in relation to the dam abutments than would have been possible had work been limited to inspections from the ground.

The balance of the time, Saturday afternoon, Sunday and Monday, October 14, 15 and 16, was devoted to discussion of features observed, review of technical data furnished by the Bureau, and in preparation of this report.

The eight items upon which the Board's views are presented, after the revision and addition referred to above are as follows:

1. Examine the completed foundations for the thrust block and the spillway intake structure and appraise their adequacy.

2. Examine the abutments of the dam and appraise their adequacy,

3. Examine the area upstream of the right abutment in the vicinity of the parking area and switchyard and comment on the adequacy of the proposed extended grout curtain.

4. Review the foundation treatment program for the dam and appraise its adequacy.

5. Review the results of the latest trial-load analysis of the dam.

6. Review concrete test results of interior mass concrete to date.

7. Comment on compaction practices for mass concrete.

8. Review temperature history of mass concrete placed during this construction season and comment on methods now being used for thermal control of concrete.

ITEM 1

Examine the completed foundations for the thrust block and the spillway intake structure and appraise their adequacy.

The Board considers the completed foundations for the left abutment thrust block and the spillway intake structure as satisfactory. The anchorage which has been installed to stabilize the upstream-dipping massive slabs of rock in the areas above and on the left side of the spillway intake should effectively overcome the tendency for these slabs to slide on the shale seams that underlie them. There still remains a mass of fractured rock, resting on a thin upstream dipping seam of shale, located in the canyon wall above and immediately downstream of the spillway gate structure. As a slide or large rock fall at this location would endanger the gate structure, it would be prudent to install some anchorage in the questionable mass.

ITEM 2

Examine the abutments of the dam and appraise their adequacy.

In its report of June 8, 1960 the Board made specific mention of the relatively thin mass of rock that will provide the arch support in the right abutment above shale bed 6R at approximate elevation 5945 and emphasized the importance of making a careful analysis to determine its stability under conditions of maximum loading including earthquake. Subsequently, the magnitude of the arch thrust on the abutment rock above shale bed 6R was reduced substantially by the omission of the

the grouting in the end joint above elevation 6000.

Unfortunately, because of the difficulty of obtaining suitable samples for testing, the values of shear strength for 6R and other thinner beds of shale that lie above 6R were not obtained. The stability analysis which was made was based on assumed values of cohesion and tangent ϕ for the shale in 6R of 500 psi and 0.8 respectively. The results gave a safety factor of 24 for the condition of failure up the dip of the shale and 32 for the condition of failure along a horizontal plane cutting both shale and sandstone. The values of cohesion and tangent ϕ used in the analysis are considered much too high. Therefore, the Board requested that an analysis be made assuming values of cohesion and tangent ϕ of 200 psi and 0.5 respectively. Such an analysis was made while the Board was at Dutch John and it was found that, for the condition of failure in the shale up-dip the factor of safety, neglecting side shear, is 10 and for failure on a horizontal plane 13. While the assumed values of cohesion and tangent ϕ suggested by the Board may be on the high side, taking into account the fractured condition of the rock, any reasonable reduction in these values would result in a safety factor substantially in excess of 4. Therefore the Board is of the opinion that the rock mass in question is capable of resisting the arch thrust.

The Board concurs in the Bureau's proposal to stabilize the overhanging cliff of sandstone above shale zone 6R downstream of the toe of the dam and to protect this shale from future weathering by the construction of a concrete retaining wall.

The remainder of the right abutment is generally satisfactory although the bench and high abrupt step that exists in the keyway excavation at shale bed 9R is very objectionable. This condition will induce cracking and should, in the opinion of the Board, be rectified by the placement of a plug or fillet of concrete, which would be cooled and then considered as a part of the arch foundation.

Generally good conditions also exist in the left abutment and the dimensions of rock mass downstream of the keyway are quite adequate at all elevations. There does exist, however, at elevation 5900 a bench and abrupt step in the foundation that should be treated in the same manner as suggested for the bench at shale bed R9 in the right abutment to improve the symmetry of the keyway and avoid objectionable cracking.

ITEM 3

Examine the area upstream of the right abutment in the vicinity of the parking area and switchyard and comment on the adequacy of the proposed extended grout curtain.

The proposed extension of the grout curtain beyond the right end of the dam to provide a barrier against the possibility of reservoir leakage through the narrow saddle of the Uinta formation adjacent to and upstream of the switchyard area meets with the Board's approval. However, the curtain line in the narrow part of the ridge as now located is too close to the canyon rim and should be moved back to about the center-line of the ridge. While the Tertiary Browns Park formation which overlies the Uinta west of the narrow saddle appears to have a low permeability, the upper part of the Uinta in this area may have a relatively high permeability, taking into account its fractured condition and the fact that it has an unconformable contact with the Browns Park formation. Should this be found to be the case, as demonstrated by the grout takes along the proposed 300-foot extension of the curtain into the area overlain by the Browns Park formation, it may be found desirable to continue the curtain farther to the west to protect the switchyard. It may also be found necessary to grout to greater depths below the Browns Park-Uinta contact than now proposed.

ITEM 4

Review the foundation treatment program for the dam and appraise its adequacy.

The program of consolidation and curtain grouting outlined in the Book of Technical Data under the heading "Foundation Treatment" meets with the approval of the Board. In view of the fractured condition of the rock disclosed in the upper part of the keyways, the Bureau's proposal to deepen both the "A" hole grout curtain and the drainage curtain in the upper elevations of the abutments is considered desirable.

The depths of the shale cutoffs at the heel and toe of the dam are in accordance with those previously recommended by the Board and are considered satisfactory.

ITEM 5

Review the results of the latest trial-load analysis of the dam.

Trial-load analyses for Flaming Gorge Dam are shown in the data book for the following conditions:

(a) Asymmetrical structure, rock modulus of 2,000,000 psi, including earthquake forces, contours for actual excavation.

(b) Symmetrical structure, based on contours and data furnished the Consulting Board in June, 1960, with earthquake forces, rock modulus of 1,000,000 psi.

These duplicate analyses resulted from a suggestion by the Consulting Board, Dutch John, Utah, June 8, 1960 as follows:

"The Board desires to call attention to the fact that the value of the rock modulus of elasticity used in computing the arch stresses was 2,000,000 psi. Although this value was approved by the Board in its report of October 23, 1958, it is now of the opinion that this value is too high. In view of the conditions disclosed by the keyway excavations, the Board is of the opinion that the modulus of elasticity might well be as low as 1,000,000 psi."

This opinion of the Board was based on a belief that the considerable fracturing of the otherwise excellent rock at Flaming Gorge might cause yielding that would reduce the modulus below the values obtained in the laboratory.

After careful consideration Bureau personnel doubts the justification of lowering the design modulus to a value of 1,000,000 psi and favor retaining the original value of 2,000,000 psi. They have, however prepared stress computations for both these limits and have made other investigations of the effect of the reduced modulus on stresses.

These studies show that changing the design modulus from 2,000,000 psi to 1,000,000 psi would affect the computed stresses as follows:

(a) The computed principal stresses at the downstream abutment would be lowered, which is favorable, but not important, as values for both moduli are within safe limits.

(b) The computed upstream principal abutment stresses would be slightly increased, eliminating the small tensile stresses found for the 2,000,000 psi modulus. This is favorable but unimportant, as these computed stresses are safe in either case.

(c) The computed concrete stresses in the arch would be somewhat increased, but still within allowable limits.

Since all computed stresses for either assumption are within safe limits it is unimportant whether the computed stresses are based on a modulus of 1,000,000 psi or 2,000,000 psi, or some intermediate value. The safety factors of the structure cover this possible variation in modulus.

Consequently, the Board approves the adequacy of the design as shown by the two trial-load analyses presented.

In any case, the Board believes that appropriate steps should be taken to insure placing temperatures and maximum hardening temperatures of concrete placed during the 1962 construction season well below the peak temperatures of concrete placed during the current season.

Respectfully submitted,

/s/ Raymond E. Davis

/s/ John J. Hammond

/s/ John W. Vanderwilt

/s/ Edward B. Burwell, Jr.

/s/ Julian Hinds, Chairman

Approved October 20, 1961

/s/ Grant Bloodgood
Assistant Commissioner
and Chief Engineer

CHAPTER II. SITE INVESTIGATIONS AND GEOLOGY

A. Site Investigations

9. Site Investigations for Flaming Gorge Unit. The preliminary exploration on Flaming Gorge damsite was conducted during the fall of 1949 when six diamond core drill holes were completed. Information obtained from studies of these holes and cores indicated that the general quality of the rock at the damsite was excellent. Data from these holes were not sufficient, however, for final design purposes and a greater amount of detail data was required. Owing to the existence of faults, joint systems, and shale seams in the rock at the damsite, it was necessary to carry out a thorough and rather extensive geological investigation and map a program to obtain sufficient detailed geological information about the foundation deficiencies so that an adequate design could be prepared for the dam and appurtenant structures.

From July 1956 through May 1958, a total of 72 additional diamond drill core holes were drilled at the damsite, on the diversion tunnel alinement, on the spillway tunnel alinement, in the switchyard, and at the domestic water-pumping plant site. Total footage drilled was 1,087 feet in 1949, and 6,457 feet in the years 1956 through 1958. These holes were both angle and vertical holes. All holes were carefully logged and the cores were recovered. Data and cores obtained from these holes were forwarded to the Denver office for consideration by the Chief Engineer and subsequent preparation of design. The location of these holes and logs showing the type and character of the material encountered, together with notes of the percolation test for each hole, are attached to the final geology report. 1/ Seismic studies were also made for the purpose of correlating with the elastic property tests.

Two horizontal drifts were excavated in 1956, one in each abutment. Drift 1 went 24.7 feet into the right abutment along a 3.5-foot shale bed. Drift 2 went 15.5 feet into the left abutment along a soft sandstone layer and followed some prominent joints. Logs of these drifts are attached to a preliminary geological report. 2/ Except for jointing, both drifts encountered rock of good quality.

Percolation tests were made in bedrock in all drill holes at 10-foot intervals. Successive pressures of 25, 75, and 150 pounds per square inch were used in these tests for periods of 5 minutes each. There was a wide range of losses from 0 to 18 gallons per minute (the capacity of the pump), but in practically all the tests it was possible to obtain 150 pounds per square inch of pressure which indicated that the joints were fairly tight. Generally speaking, the most solid rock with the least jointing and attending water loss was in the river channel under the downstream toe of the dam. The greatest losses were high in the abutments where the joints were more open.

The faults, joints, slip planes, and shear zones in the rock in the vicinity of the damsite were not extraordinary or uncommon for this type of terrain; however, they represented a weakness in the rock that required careful consideration and attention. Some of the problems that were anticipated because of this rock weakness were seepage, overbreak or extra excavation to reach sound rock during excavation, and pinning of certain areas to prevent possible movement. The overbreak problem was evidenced during keyway excavation operations and resulted in considerable overrun in order to reach sound bedrock, and this extra excavation eventually had to be filled with concrete during concrete placing operations. The seepage problem was expected and has been relatively minor. Special consideration for this problem included extension of the main cutoff grout curtain deeper into both abutments. As anticipated, the seepage problem became more evident after reservoir storage was initiated in 1962 and seepage water began to show up on both abutments in small but increasing amounts as the reservoir head increased. Pinning of rock areas with anchor bars on both abutments was expected and was performed as required during construction operations in order to assure stability of certain rock areas and to prevent possible rockfalls on the powerplant.

1/Wongwai, G.T.I., "Geology Summary of Flaming Gorge Dam, Flaming Gorge Unit--Utah-Wyoming--Colorado River Storage Project," Bureau of Reclamation, 1963 (unpublished).
2/"Preliminary Geological Report of Flaming Gorge Dam and Reservoir Site," G-109, Bureau of Reclamation, Region 4, Salt Lake City, Utah, January 1957 (unpublished).

The presence of shale seams in the keyway areas indicated that additional geological information was required in order to consider problems of percolation through these seams, rapid deterioration of these seams when exposed to large quantities of water, and weakness of the stratigraphic column of the keyway. Project survey personnel collected data and prepared maps and drawings to show location of all major shale seams in both abutments. These maps and drawings were furnished to the Denver office for consideration in preparation of designs. It was decided that treatment of the shale seams would consist of placing concrete cutoff tunnels in the shale beds at the heel and toe of the dam in order to increase the percolation path of seepage water through the shale and to confine the shale and prevent it from squeezing out under load when it became wet. Additional details of the treatment for shale seams are covered in sections 13 through 16.

B. Geology

10. Regional Geology. The dam and reservoir area is near the east end of the Uinta Mountains. These mountains are a broad anticlinal fold with the older rocks exposed on the crest. Rocks ranging from pre-Cambrian to Tertiary in age occur on both the north and south flanks of this range, dipping away from the crest and exposed in colorful sequence. All are sedimentary in origin and include sandstone, shale, quartzite, and limestone. Faulting is quite prevalent along the northern flank of these mountains and can be observed in several places. A fault of large displacement occurs along the north edge of Hideout Flat, 18 miles upstream, where the Weber sandstone of Carboniferous age rests flush against the much older pre-Cambrian Uinta quartzite.

The course of the Green River as it cuts its way across the eastern end of the Uinta Mountains is one of unusual interest. It meanders in and out of hard formations regardless of structure. On several occasions, it cuts itself out of the hard rock into relatively soft shales, and when it seems it would have had a much easier path to continue in the shale it turns abruptly back into the hard, resistant formations which comprise the center of these rugged mountains. Its location was undoubtedly determined by a much different topography in the past. The present meandering course was doubtless established in a softer uniform sediment, and after cutting down hundreds of feet the river found itself partly astride the resistant formations and partly in the softer sediments. Subsequent erosion has removed all the upper sediments, leaving the river in Red Canyon entirely in the resistant Uinta formation. The dip of the strata is to the northeast and the range from 1° to 75°.

Two maps published by the U.S. Geological Survey (1955 and 1956)--the Manila quadrangle and the Flaming Gorge quadrangle--show detailed structure and areal extent of the formation in the areas north of the damsite. Two other quadrangles--the Dutch John Mountain and Goslin Mountain areas--provide excellent data on the general geology of this region.

11. Reservoir Geology. The reservoir, with a maximum water surface elevation of 6045, rests on sedimentary rocks ranging in age from pre-Cambrian to Tertiary. These formations are predominantly shale and sandstone with minor amounts of limestone, and all are fairly impervious to percolating water. Since the Green River forms the low drainage of this entire area and all ground water is tributary to this stream, there seems no possibility that reservoir seepage will be of any consequence.

12. Damsite Geology. Green River, in its course through the reservoir basin, meanders in a large half circle. At the damsite, its general direction is southeast but it makes a 40° curve to the south at the axis. This curve provides an attractive inlet and outlet for the diversion works and a good slope for the spillway. The profile along the axis of this site is unusually good for a high dam. A shoulder extends out from the right side which makes an excellent location for an arch dam.

Except for a small patch of Tertiary conglomerate on the high right abutment, bedrock at this site all belongs to the Uinta formation of pre-Cambrian age. It consists of quartzite, quartzite conglomerate, quartzose sandstone, and a few thin beds of shale. The general quality of this rock is excellent for all types of construction purposes. The shale occurs in lenticular beds usually less than 2 feet thick and widely spaced so they are insignificant when contrasted with the massive quartzite.

Grain size in the sandstone and quartzite ranges from medium sand to pebbles 2 inches in diameter. The sandstones and quartzites differ only in the degree of cementation. Both are cemented with silica, and where the cementing action has completely filled the pore space, they have formed the quartzites. Where the pore space is only partially filled, they are called quartzose sandstone. Beds of conglomerate are interspersed in all the strata and are well cemented and equally as hard as the quartzite.

Bedding ranges from 2 to 20 feet apart, with many thick, massive ledges 20 to 40 feet high. Structurally, the dip and strike of the strata are ideal to receive the thrust of the dam. The dip is 16-1/2° N. and the strike N. 52° E.

Folding in the area has produced some shearing in the brittle rock. A prominent system of joints parallel to the axis and located on the right abutment is evident. The joints are nearly vertical and are spaced 2 to 4 feet apart. Drilling disclosed that these joints did persist at depth but they were fairly tight. Some healing action has taken place and the crevasses are partially filled by secondary cementation.

Two faults or shear zones occur in the damsite area. Neither of these is involved in the dam itself, but the diversion tunnel crosses one of them. One is located upstream about 82 feet from the face of the dam. It is vertical and trends N. 73° E. The shear zone is 8 to 10 feet wide and consists of broken blocks of quartzite. There is little or no gouge associated with the movement where the fault can be observed.

The other shear zone is located 1,100 feet downstream from the face of the dam and follows a shallow draw up the right abutment. There is no well-defined topographic expression on the left. A sharp change in dip and strike on each side of the draw, together with the talus covering in otherwise hard, well-exposed rock, was strong evidence of its position. It was explored by two core drill holes both of which were angle holes across the shear zone. Both encountered fractured rock. One drill hole was drilled on the spillway tunnel line and showed some fracturing but this was not severe enough to result in heavy ground or serious construction problems. The fractures range from 6 inches to 2 feet apart and the sandstone is leached or ground up to small fragments near the fractures.

(a) *Diversion Tunnel.*--The diversion tunnel crossed the upstream fault between tunnel stations 2+25 and 2+30. Tunnel excavation in the shear zone of the fault area disclosed that the rock was shattered but tight and dry. The tunnel lies in bedded hard quartzite and sandstone with a few thin beds of red ferruginous shale. This rock dips upstream at 16-1/2° and the major part of the tunnel is approximately along the line of maximum dip. Except for jointing, this rock is well adapted to tunnel construction. Where jointing is more severe, steel supports were placed in the tunnel at 5-foot centers. These supports were composed of 8-inch wide-flange beams and were located between tunnel stations 1+25 and 2+55, stations 5+75 and 6+29, and stations 9+33 and 9+63. The rock was brittle and shot well, but it was hard and resistant which dulled drill bits quickly. Several shale beds were encountered during tunnel excavation, but they were so thin and widely separated that they did not influence the conditions in the tunnel to any significant degree. Little or no water was encountered except in the invert of the tunnel and these small seeps were directly tied to river stages.

(b) *Spillway.*--Because of its superior angle for the stilling basin, the left abutment was selected for the spillway. Geologic conditions were suitable for a tunnel-type spillway. The rock was generally of good quality except near the outlet end of the tunnel where it crossed a downstream fault, and in this area there was a zone of broken rock. The left abutment where the spillway is located was exposed to more weathering than the right abutment and contained many unstable boulders and ledges. This unstable material presented no problem for successful operation of a tunnel-type spillway; however, it did present a problem of potential danger to the powerplant area in the river bottom should it come loose and fall. It was therefore removed during construction operations.

(c) *River Channel Overburden.*--It was determined that the foundation rock in the river channel was hard and unweathered. The river fill was essentially sand and well-rounded gravel and boulders of hard quartzite. A few large rock slabs occurred intermingled with the gravel. These slabs represented rockfalls from the cliff adjacent to the river.

CHAPTER III. FOUNDATION TREATMENT

A. Shale Zones

13. General. The foundation rock for the dam (see chap. II) has an approximate dip of 15° upstream and toward the reservoir. The rock ranges from a moderately hard sandstone cemented with silica to a very hard quartzite. Interspersed at various elevations throughout the foundations are zones of varying thicknesses composed of interbedded lenslike layers of shale, siltstone, and sandstone in varying proportions. These zones range from 1 foot or less to 10 or 12 feet in thickness. Individual shale layers within the zones and at other levels in the bedrock sequence are generally of minor thickness and range from paper thin to about 4 feet. For convenience, the more important shale zones are numbered for identification, beginning at 16 in the foundation excavation beneath the river channel and decreasing to 9L and 5R on the left and right abutments, respectively. See figures 4 and 5 for typical identification of shale zones in both abutments.

14. Concrete Cutoffs for Shale Zones. It was deemed advisable to excavate, and backfill with concrete, upstream and downstream cutoff drifts into the abutments for the larger size shale zones. The object of these cutoffs was to increase the path of possible reservoir percolation through the zones and to confine the shale zone materials between the cutoffs. Eight zones were protected by cutoff drifts, as indicated by the following tabulation:

Left Abutment

Zone No.	Average thickness, feet	Minimum depth of cutoffs, feet
*9L	9	25
12L	2	50
14L	12	75

Right Abutment

Zone No.	Average thickness, feet	Minimum depth of cutoffs, feet
6R	5	50
7R	7	50
9R	9	50
12R	3	50
14R	7	75

*Only one cutoff drift, due to minimum reservoir head and reduced thickness of dam.

The required depths for the cutoff drifts were determined by the character of the materials encountered as the excavation for the drifts progressed, by the thickness of the zones, and by the reservoir head to which they will be subjected.

15. Concrete Protective Walls For Shale Zones. For some distance downstream of the toe of the dam the shale zones are exposed, due to scaling and sluicing activities on the abutments during construction of the dam. Immediately above several of the shale zones were massive layers of rock with a predominate near-vertical jointing system. This jointing system, together with the dip of the bedding planes toward the river, would

Figure 4. --Left keyway of the dam. P591-421-2302. June 2, 1960.

Figure 5. --Right keyway of the dam. P591-421-2308. June 7, 1960.

contribute to possible instability of rock masses if the support of the underlying shale zones were removed. To safeguard against weathering of certain of the shale zones and consequent undercutting of rock masses it was decided to construct concrete protective walls.

In all, six protective walls were constructed for the protection of the following shale zones:

Numbers 9L, 10L, 6R, 7R, 9R, and 14R

All of the walls, with the exception of the 9L wall, were designed and constructed for the protection of the shale zones in the areas downstream of the toe of the dam. The 9L wall is located upstream of the spillway intake structure on the left side of the intake channel, extending upstream until the shale zone dips below the floor of the channel.

All of the walls were designed to a minimum thickness. Anchor bars, 1-3/8 inches in diameter, were utilized wherever possible to increase the stability of the walls. Drainage systems were installed to take care of any seepage in the rock that might tend to build up hydrostatic head behind the walls. All of the walls were of nominal size except for shale zones 6R (fig. 6) and 14R. At these locations the walls not only covered the shale zone but were extended a considerable distance above the tops of the shale zones to provide additional stability for the overlying rock.

16. Anchorage of Rock Masses Adjacent to Shale Zones. There were several areas adjacent to the downstream toe of the dam where rock masses might have a tendency to slide, due to the jointing system and the dip of underlying shale zones or seams toward the river. These areas, with the exception of an area adjacent to the upstream portal of the spillway tunnel, were anchored in place by 1-3/8-inch-diameter reinforcement bars, grouted into the rock. The bars were embedded to various depths, depending on the locations of the cracks and seams that might affect the stability of the rock masses.

Figure 6.--Downstream rock protective wall, on right dam abutment, for shale zone 6R. Transformer circuit towers C1-T1, C1-T2, and C1-T3, with double-circuit-tower type DTH in the left background. P591-421-4942, August 1, 1963.

The area adjacent to the upstream portal of the spillway tunnel required more elaborate anchorage than was designed for other locations. A pattern of 1-3/8-inch-diameter reinforcement bars, 94 in number, were grouted in place for the rock slabs on the left side of the portal structure. Subsequent excavation and cleanup revealed the need for additional anchorage, principally on the left side, and at a localized area on the right side. In order to induce a downward weight component to the rock slabs, prestressed 1-1/2-inch-diameter anchor bars were installed, 59 on the left side of the portal and 5 to the right of the portal. The prestressing was obtained by embedding and grouting the bars a minimum depth of 10 feet below the questionable bedding plane on which the rock slabs might slide, then stressing the bars by applying a torque through a nut and plate arrangement at the rock surface. Load cells were installed on three of the anchor bars to obtain a correlation between applied torques and induced tensions (sec. 43). It was desired to obtain induced tensions in the bars of approximately 18,000 pounds per square inch.

The torquing and consequent tensioning of the bars equipped with load cells furnished the basis for torquing the remainder of the bars and also furnished valuable data for any future similar installations. Information obtained is listed as follows:

(1) With the threads clean and dry and with no lubricants it was not possible to obtain the desired tension with a reasonable torque. A torque of 550 foot-pounds induced an average tension in the bars of approximately 6,000 pounds per square inch.

(2) The use of hardened steel washers to reduce the friction losses at the nut and bearing plate increased average tensions in the bars to about 10,000 pounds per square inch.

(3) To obtain the approximate desired tensions of 18,000 pounds per square inch it was necessary to lubricate thoroughly and torque to approximately 700 foot-pounds. A temporary lubricant of engine oil was satisfactory, but for a more permanent type of lubricant a mixture of white lead, graphite, and engine oil was used.

(4) Some relaxation was noted in the tensioning of the bars equipped with load cells in the first few weeks. After an elasped time of about a year and retensioning, very little if any reduction in tensioning has been noted for the subsequent 2 years.

B. Grouting

17. Basic Grouting Plan. The grouting plan as formulated by the specifications was very similar to standard Bureau procedures for foundation grouting that have been established through past practices. The plan was to consolidate the foundation rock beneath a specified upstream portion of the dam by drilling a pattern of relatively shallow holes and grouting under comparatively low pressures prior to concrete placement in the area. This program was identified as B-hole drilling and grouting. Later, as specified, a high-pressure grout curtain would be obtained by drilling deep holes from the foundation gallery and grouting under high pressures. These holes were identified as A-holes. See sections 173 through 185 for foundation drilling, grouting, and drainage.

Four lines of B-grout holes were specified, with the lines on about 20-foot centers and the upstream line directly under the upstream heel of the dam. Grout holes in the lines were to be on about 20-foot centers and drilled normal to the rock surface to approximate depths of 25 feet.

The spacing of holes for the high-pressure A-hole curtain was to be determined to a certain extent by the grout take. The specifications indicated a spacing of 5 feet, with depths of holes ranging from a maximum depth of about 200 feet in the river channel to a depth of approximately 50 feet near the top of either abutment. It was assumed that the maximum depth of hole should bear a direct relation to the maximum reservoir head at the location of the hole. This relation, roughly, is that the depth of hole should be 40 percent of the hydraulic head. At and near the bottom of the canyon, the holes had been planned to extend to curtains 75, 150, and 200 feet in depth where applicable.

18. Additional Consolidation Grouting Requirements. Owing to the condition of the jointing system of the rock after excavation for the dam had been completed, the entire

foundation of the dam required consolidation grouting rather than the upstream portion only, as shown in the specifications. Eight lines of B-grout holes spaced at 20-foot centers were drilled and grouted from the foundation between shale zones 14L and 14R in the central portion of the dam. The depths of holes were increased to 50 feet, twice the specifications recommended depth. Concern was felt about obtaining satisfactory consolidation grouting on the steep abutments by grouting from the foundation. It was therefore decided to incorporate a series of transverse adits at approximately 50-foot intervals in elevation in the gallery system from which consolidation grouting could be accomplished after mass concrete had been placed and contraction joints grouted. Seven grouting adits were formed in each abutment from which B-hole grouting was performed, as shown on drawings No. 591-421-1246 and 591-421-1247*.

The addition of the grouting adits to the gallery system was also beneficial in permitting auxiliary foundation grouting and drainage to reduce seepage near the downstream toe of the dam. These adits can also be used for the above purpose should future additional grouting or drainage be required. Also see section 25.

19. Additional High-Pressure Grouting Requirements. As drilling progressed and water was being stored behind the dam, seeps developed at and beyond the downstream toe of the dam, indicating seepage either through or around the already grouted foundation. Additional holes were drilled and depths were increased along the A-hole curtain to try to intercept the water passages. Auxiliary and exploratory curtains were also drilled and grouted downstream from the main curtain until favorable reductions in seepage become evident. The downstream curtains were drilled from the transverse grouting adits. The main A-hole curtain was drilled from the foundation gallery through block 24 and was extended to the right abutment by drilling from the top of the dam and along the downstream shoulder of the access road. See sections 173 through 185 for additional grouting information.

A grout curtain, beginning at the right end of the dam and extending in a westerly direction in the upstream portion of the parking area, was drilled and grouted to provide a barrier against the possibility of reservoir leakage through the narrow saddle into the switchyard area.

Figures 7 through 10 and drawings No. 591-421-1248* through 591-421-1250* show some of the high-pressure grout holes "as drilled" indicating locations, lengths, and grout takes.

C. Drainage

20. Basic Drainage Plan. The drainage plan as shown in the specifications was based in general on standard Bureau practice. A change from most previous jobs was the drilling of the drainage holes on the abutments sloping toward the gallery whenever possible--thereby making the holes self-draining. The depth of the drainage curtain at the foundation of the dam was established at 60 feet, varying to 30 feet near the top of the dam. Metal pipe and fittings were to be embedded in the floor of the galleries or the vertical gutters in the stairway shafts so that the drainage holes when drilled would be on approximately 10-foot centers at the bottom of the curtain.

21. Additional Drainage Requirements. It became evident during the construction of the dam and initial reservoir storage that the control of seepage downstream of the dam would require a combination of both extensive grouting and drainage programs (see secs. 173 through 185). The drainage program as formulated for the specifications was followed to approximately elevation 5675, to which a drainage curtain about 60 feet deep was drilled. The curtain above elevation 5675 was deepened to about 100 feet in order to be more effective in intercepting seepage.

After completion of grouting, including the auxiliary and exploratory programs, it was decided to drill downstream drainage curtains on both abutments from the transverse grouting adits to try to intercept the increasing seepage as the reservoir rose behind the dam. Drainage holes were also drilled into areas where seepage was evident on the right abutment, from locations in the diversion tunnel downstream of the tunnel plug.

Figures 11 through 14 show the drainage holes as drilled.

*Not included.

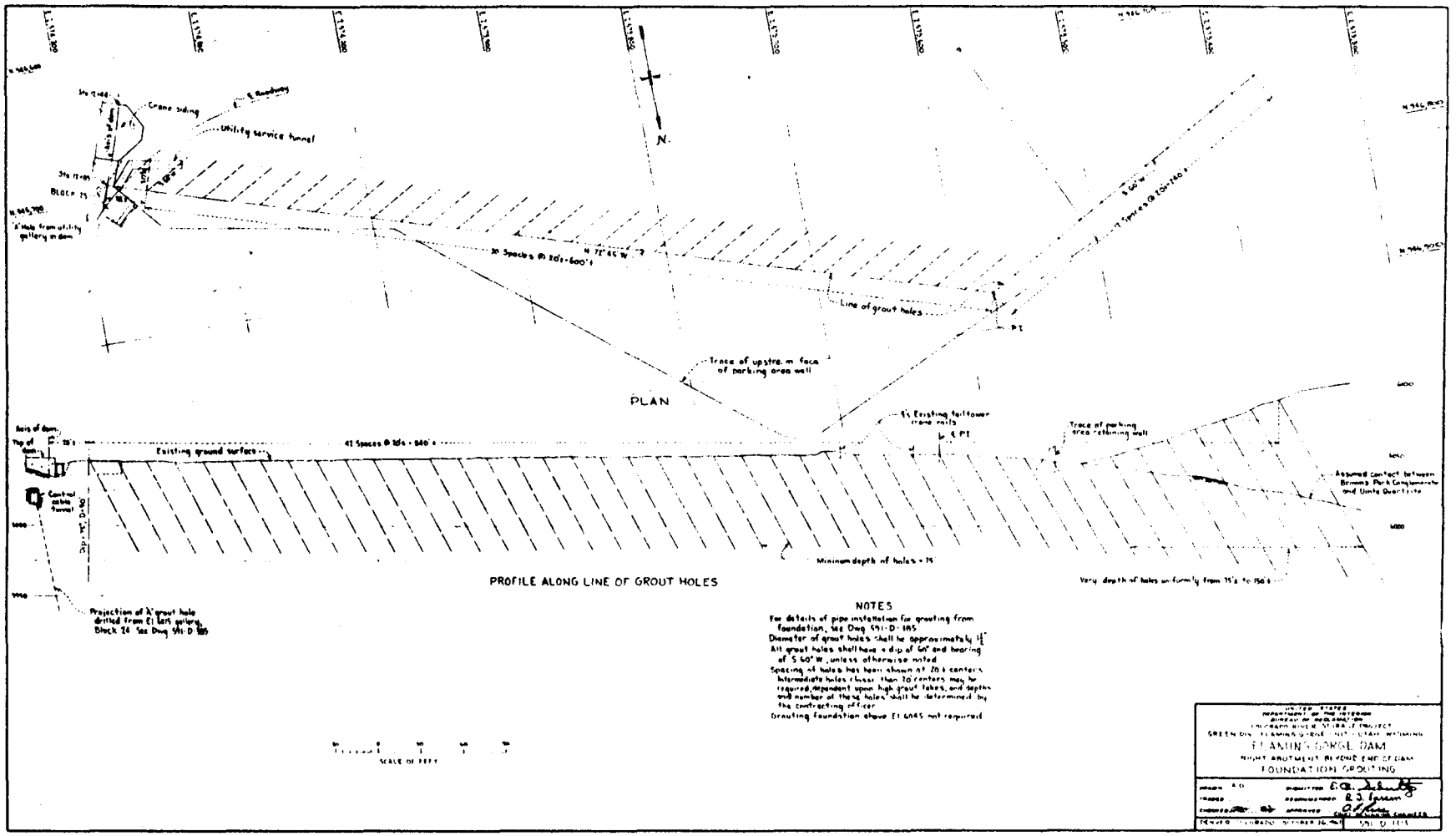


Figure 7. --Foundation grouting of right abutment beyond end of dam.

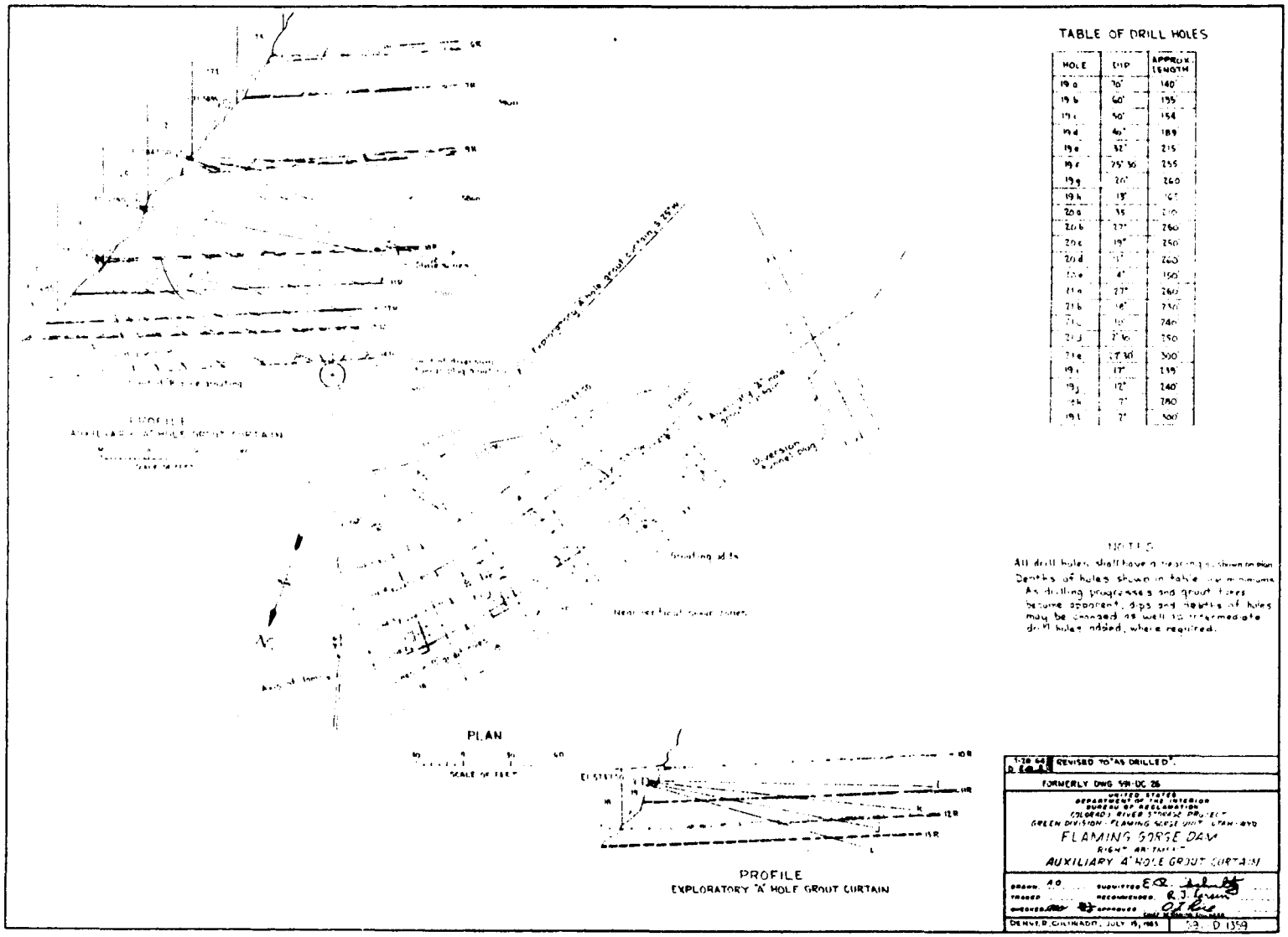


Figure 8. --Right abutment auxiliary A-hole grout curtain.

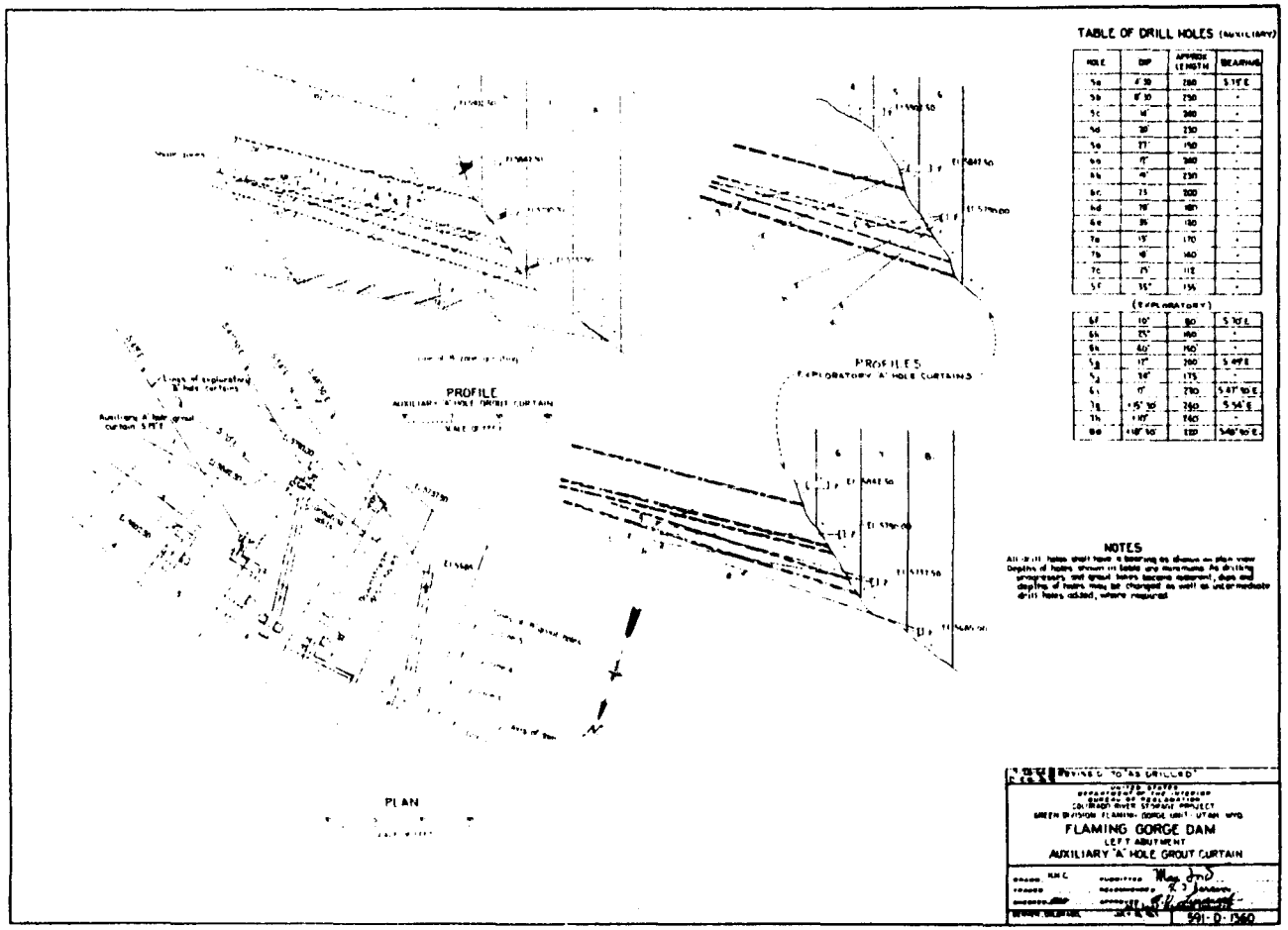


Figure 9.--Left abutment auxiliary A-hole grout curtain.

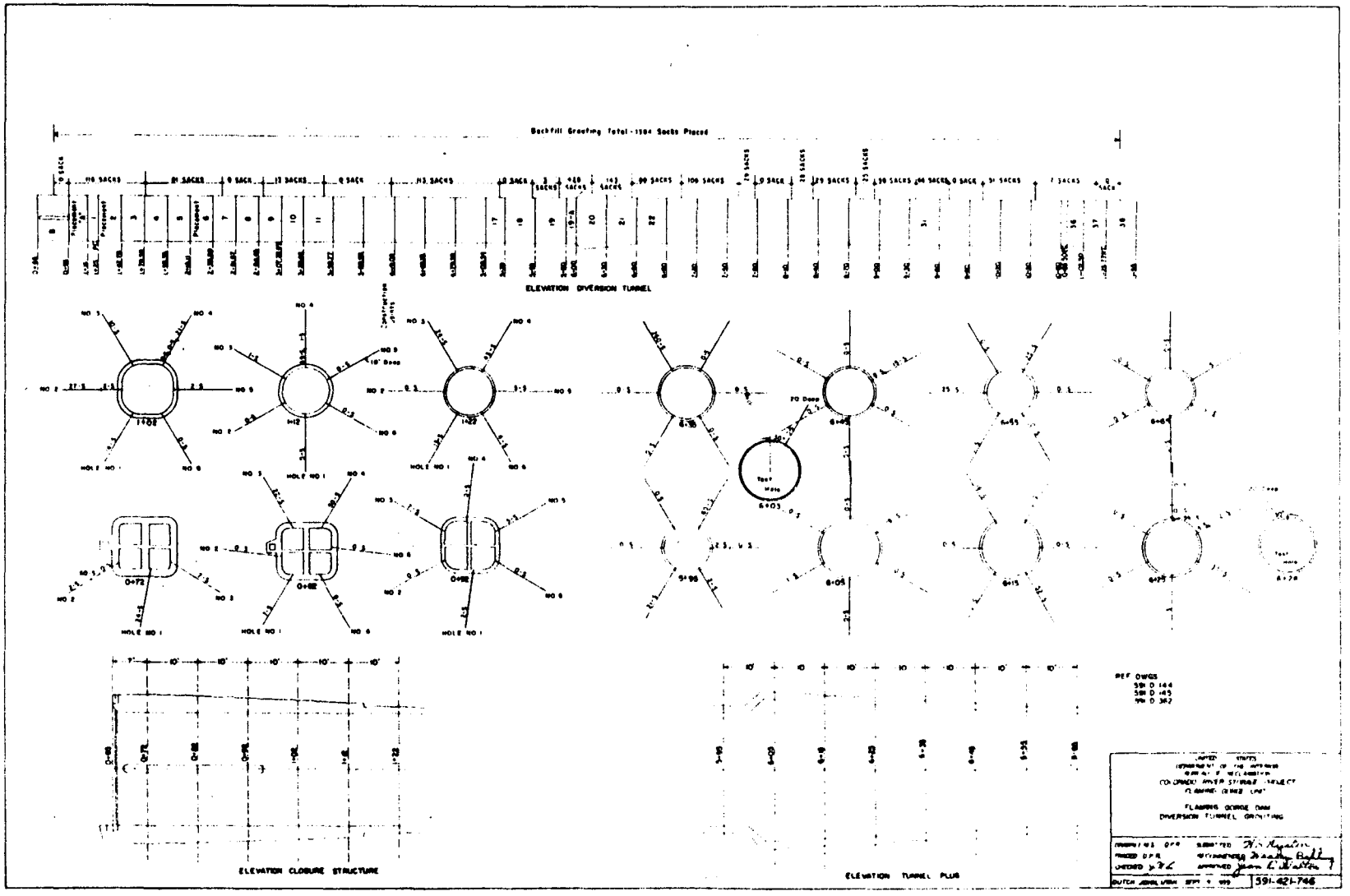


Figure 10. --Diversion tunnel grouting.

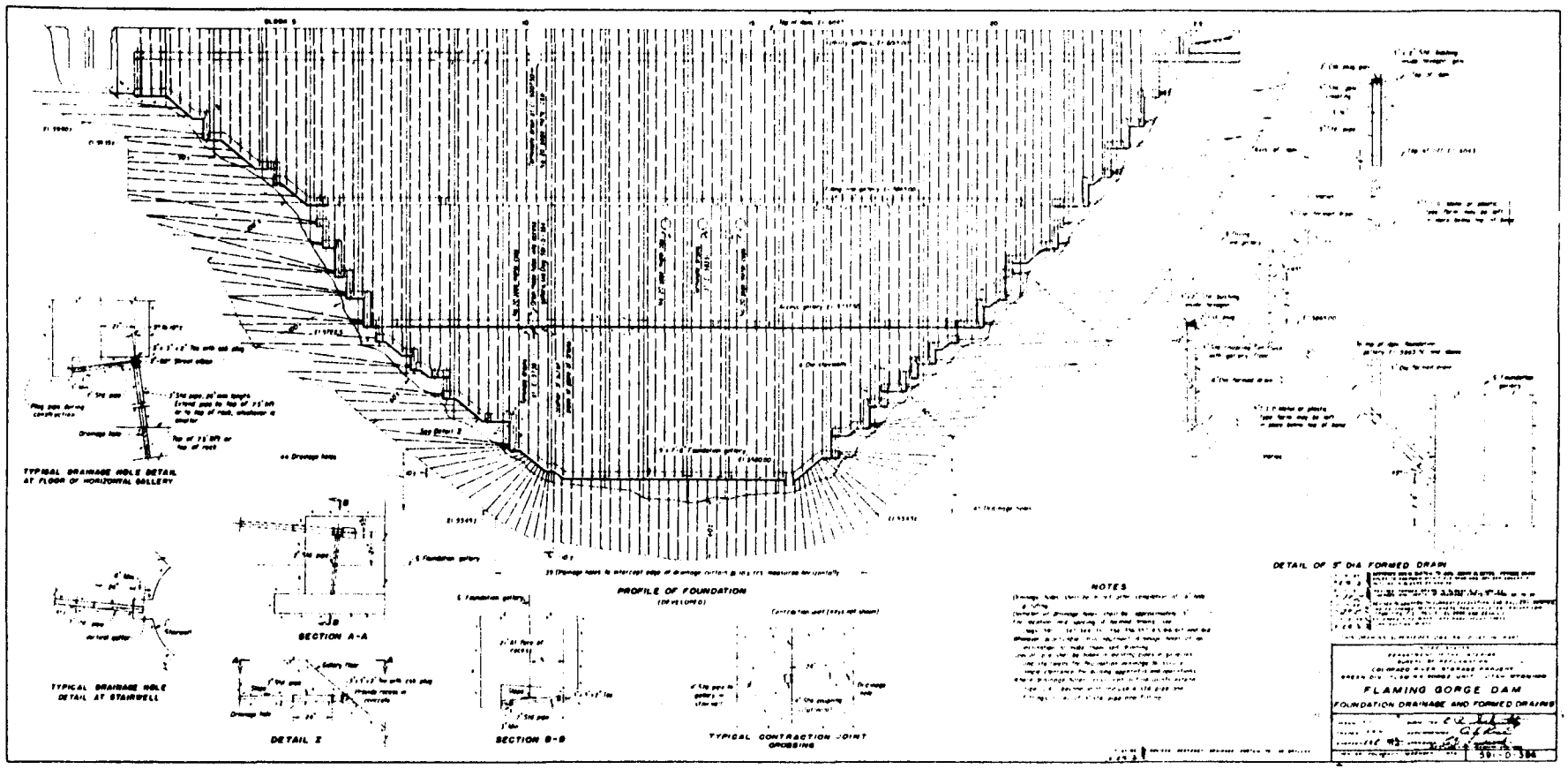


Figure 11. --Dam foundation drainage and formed drains.

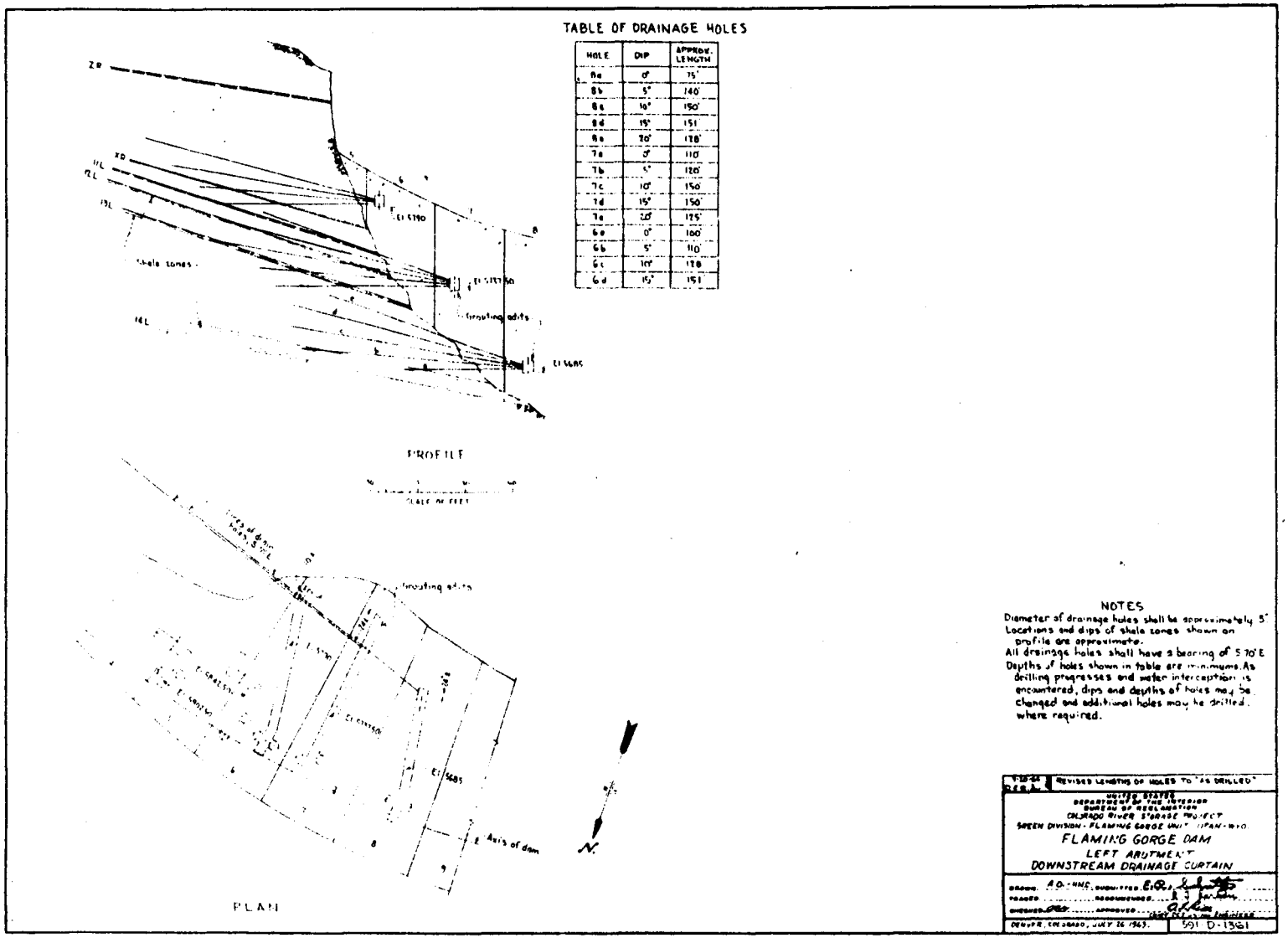


Figure 12. --Left dam abutment--Downstream drainage curtain.

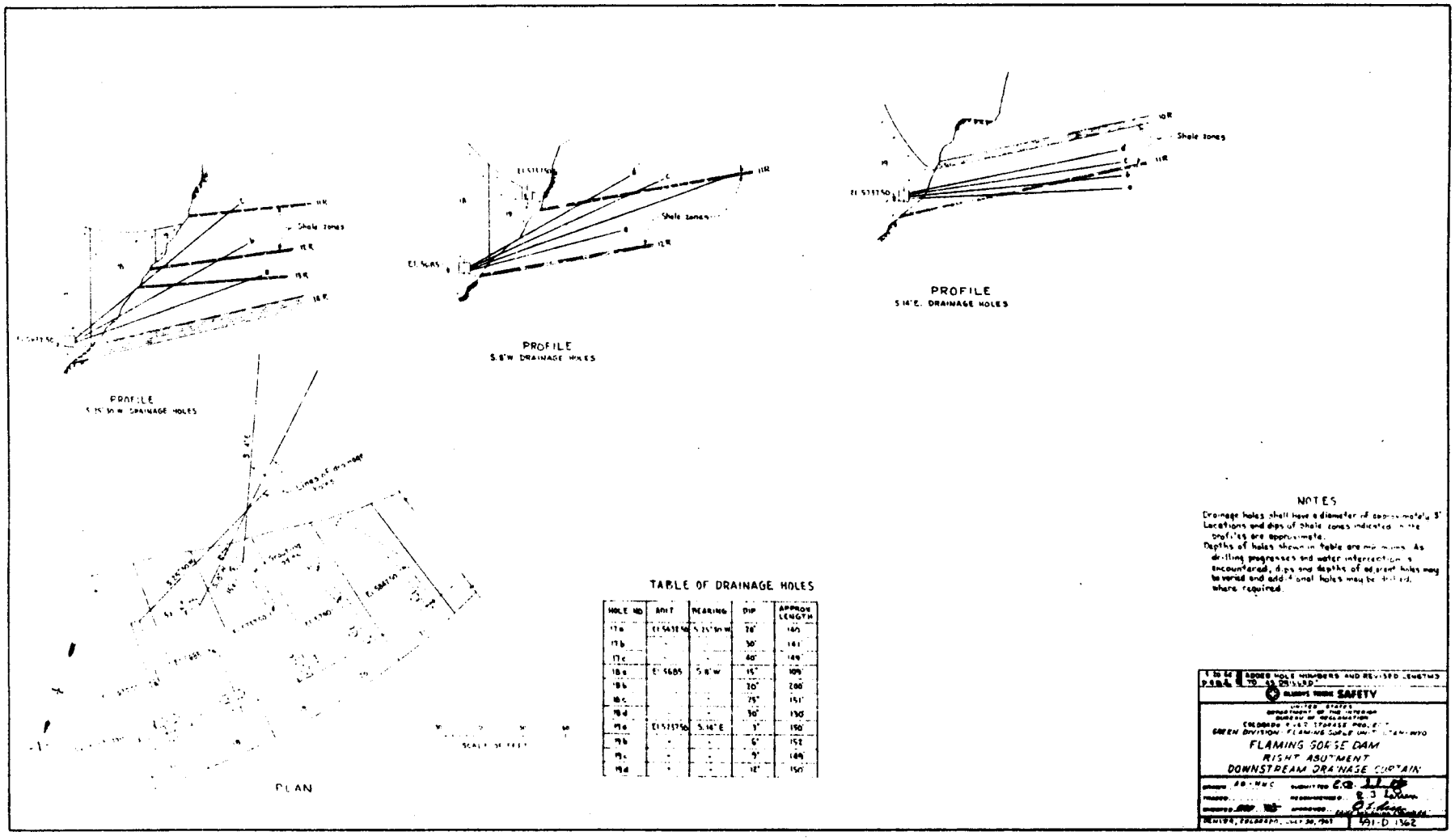


Figure 13. --Right dam abutment--Downstream drainage curtain.

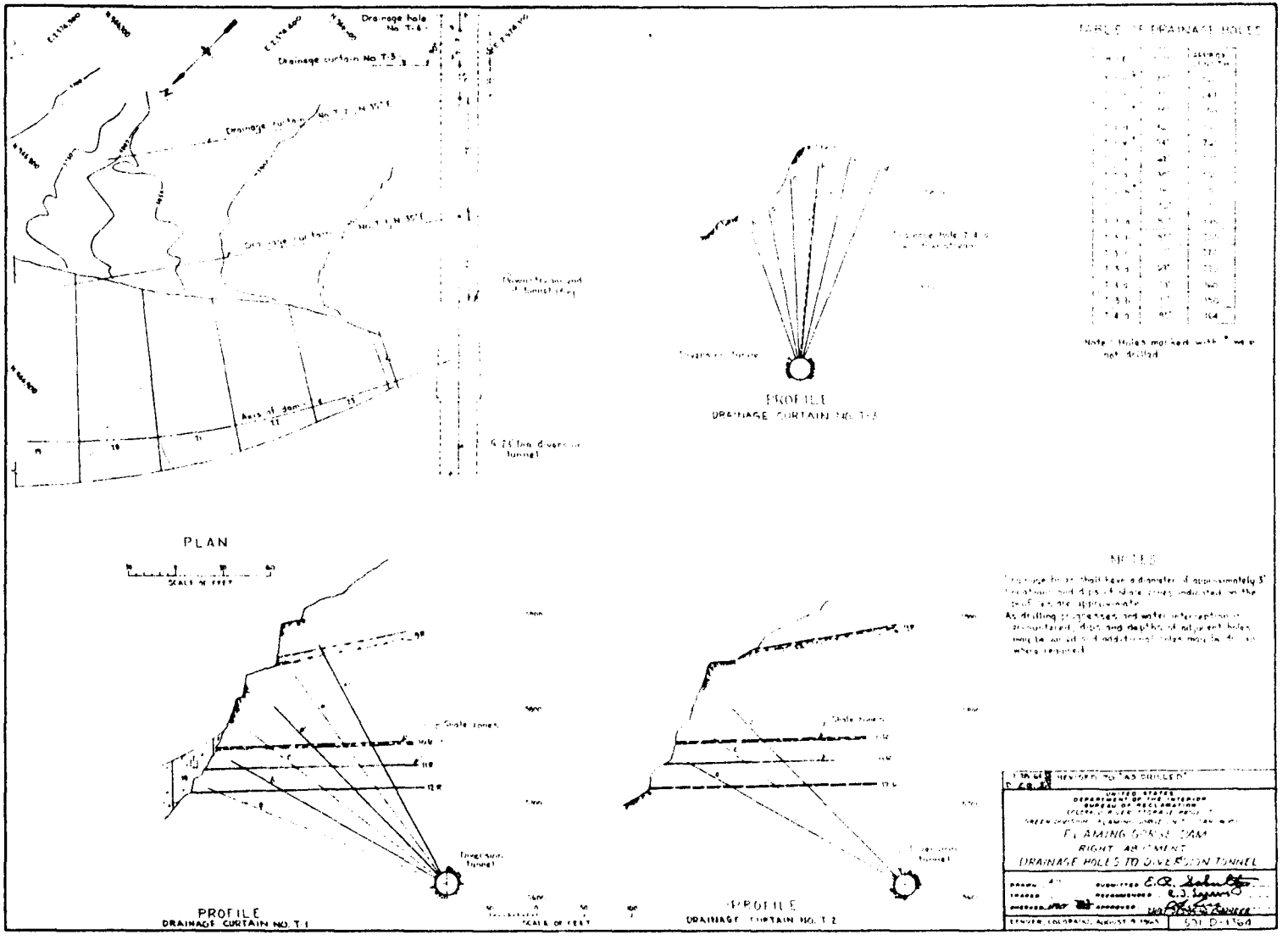


Figure 14. --Right dam abutment--Drainage holes to diversion tunnel.

APPENDIX D

E. Foundation Grouting, Drilling, and Drainage

173. Introduction. Flaming Gorge Dam is a thin-arch concrete structure 502 feet high with a crest length of 1,285 feet. The foundation rock was drilled and grouted, and foundation drainage was provided by holes drilled into the rock. The general plan followed in pressure grouting the rock foundations is outlined below:

(1) Drilling and grouting the rock foundations of the dam and spillway-intake structure through low-pressure, shallow grout holes designated as B-holes.

(2) Drilling and grouting the rock foundation of the dam and spillway-intake structure through high-pressure, deep curtain grout holes designated as A-holes.

(3) Drilling and grouting the rock surrounding the upstream and downstream portions of the spillway tunnel and around the diversion tunnel plug.

(4) Grouting the backfill concrete placed in the tunnels excavated for cutoffs in shale seams.

(5) Placing mortar or grout by grouting methods to complete the placement of concrete lining in tunnels.

(6) Drilling and grouting at other locations as directed.

Foundation drainage of the dam was provided by drilling holes into the rock through 3-inch-diameter pipes embedded in concrete from the floor of the dam foundation gallery and powerplant penstock gallery.

All of the drilling and grouting work was performed by Selby Drilling Corp. of Boise, Idaho, a subcontractor of Arch Dam Constructors. The first grouting on the project was in August 1959 when backfill grouting of the diversion tunnel was begun. The foundation grouting was completed in October 1963. No drilling or grouting was performed during the periods from October 1959 through July 1960 and from November 1960 through December 1961.

Figures 200 and 201 show the general plan of the foundation grouting for Flaming Gorge Dam and figure 11 shows the general plan for foundation drainage. The final construction report (see bibliography) also contains many drawings showing the results of the grouting operations.

174. Drilling and Grouting Equipment. Most of the drilling, including core drilling, grout holes, and foundation drainage, was performed with pneumatic drills. Some core drilling, and the first B-holes drilled from the rock foundation were drilled with two-cylinder piston-type drills. All rotary drilling was performed with diamond set bits. Both plug bits and core bits were used. In general, 1-1/2-inch-diameter (EX) core bits were used to drill grout holes through concrete, and plug bits were used in rock.

Portable grout plants which were set up at various locations for different phases of the work, usually consisted of a "Crowe" type mixer, a vertical agitator, and a pump. Mixers and agitators of various capacities were used during the grouting program. Pumps with capacities from 200 to 300 pounds per square inch were used for most of the low-pressure grouting and some of the A-hole grouting. Pumps of higher capacity were used where higher pressures were desired.

175. Diversion Tunnel. Backfill grouting of the diversion tunnel and pressure grouting of the foundation rock at the diversion tunnel plug section and intake structure, were performed in August and September of 1959.

Grouting and vent holes for backfilling the diversion tunnel lining by grouting methods were spotted at high points in the excavated tunnel arch. After the tunnel lining had been placed, the grout holes were drilled through the concrete with both jackhammers and diamond drills. Backfill grout with a 1:1 water-cement ratio was ther pumped into the holes through short plug packers at a maximum pumping pressure of 30 pounds per square inch. A total of 1,384 sacks of cement was placed in the 1,090 lineal feet of tunnel for an average of 1.3 sacks per lineal foot of tunnel. Grout leakage at some construction joints was rather heavy, but was controlled by calking the joints with oakum and wooden wedges.

The upstream closure (intake structure) area and the plug area were the only portions of the tunnel where pressure grouting of foundation rock was performed. One and one-half-inch-diameter grout holes were drilled to a maximum depth of 30 feet in a pattern of radial holes from rings spaced 10 feet apart in these areas. A diamond drill was mounted on a specially constructed crawler platform to drill these holes. A total of 1,085 sacks of cement was placed in 2,438 lineal feet of hole for an average take of 0.44 sack per lineal foot of hole. A mechanical expansion-type grout packer was used to water test and grout these holes. Grout consistencies ranged from a 5:1 to a 1:1 water-cement ratio by volume, and maximum pumping pressures of 100 pounds per square inch were used.

176. Consolidation Grouting in B-Holes. Initially, the B-hole foundation grouting program was to consist of four lines of holes under the heel of the dam, concentric with the upstream face and spaced 20 feet apart with 30-foot-deep holes staggered at 20 feet on centers from adjacent lines. As excavation of the keyways proceeded, bedding and jointing systems in the rock were such as to require deeper excavation than originally planned. The joint systems did not tighten with depth as rapidly as anticipated, and rock

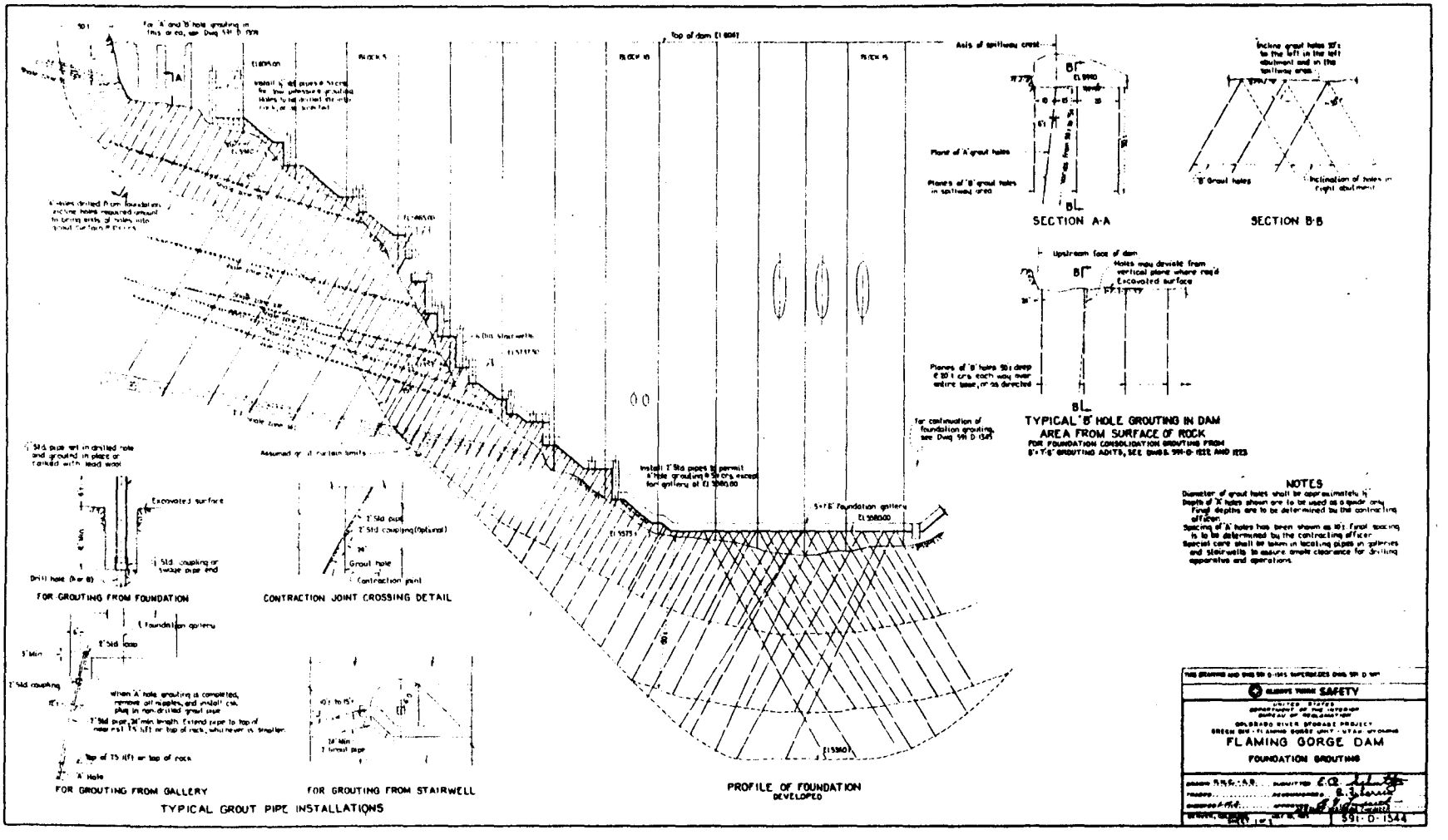


Figure 200.--Dam foundation grouting at left abutment--Locations of grout holes and grouting details.

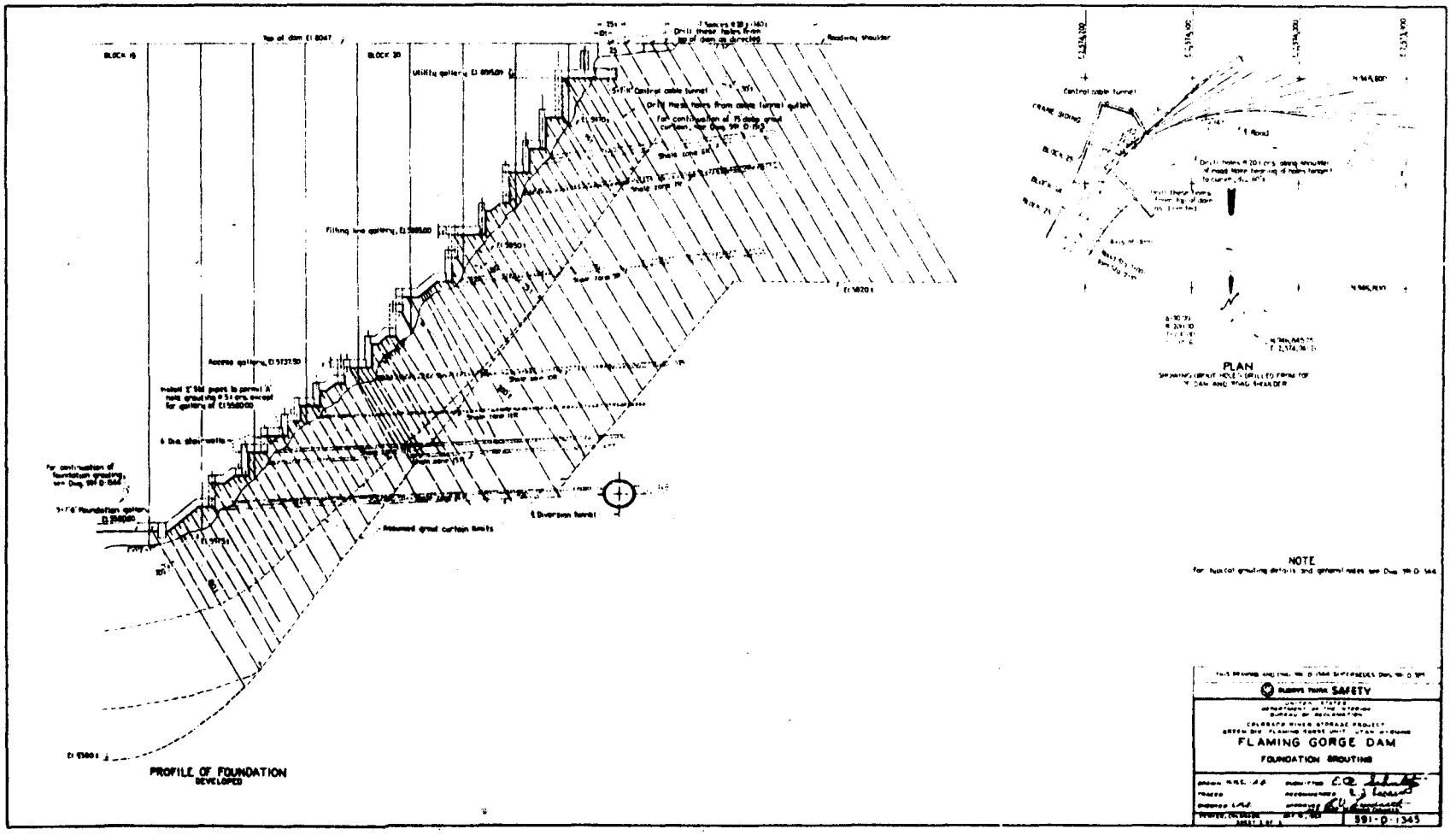


Figure 201.--Dam foundation grouting at right abutment--Locations of grout holes and grouting details.

masses beyond the intended excavation lines were disturbed during blasting operations. These conditions dictated a more extensive consolidation grouting program, and the B-hole pattern was extended to cover the major portion of the dam foundation to a depth of 50 feet below the rock surface.

Subsequently, the B-hole drilling and grouting was performed in two phases. From August 18 to October 8, 1960, the bottom foundation rock was drilled and grouted from the rock surface prior to concrete placement. Consolidation grouting during this period was completed between the base of shale seam 14L on the left abutment and seam 14R on the right abutment and included all of blocks 9 through 16, and portions of blocks 7 and 8 and blocks 17 and 18. The remaining B-hole drilling and grouting, with the exception of holes in the thrust block and spillway intake area, was performed from transverse adits within the dam. The spillway intake B-holes were grouted in July of 1962. Drilling and grouting operations from the adits were begun on June 29, 1962, and were continued until their completion on July 3, 1963.

Holes from the foundation rock were drilled to intersect cracks, shear planes, and shale seams at angles ranging from vertical to 30° from vertical. Considerable difficulty was experienced in grouting these holes because of excessive leakage. The grout appeared to travel through some of the thin shale seams and along the contact planes of larger shale seams, particularly shale seam 16, and then leaked to the surface through the nearly vertical cracks or joint systems. Grout leakage through shale seams which were exposed or surfaced in the foundation keyway of the dam was almost impossible to control because calking would not hold in these seams. It is believed that consolidation of the foundation rock was satisfactory but that a closer spacing of holes would have been required to completely "close-out" the B-hole grouting to provide an effective cutoff against seepage. The A-hole grout curtain established at a later date through this area has resulted in an effective seepage cutoff. Grouting in this area was performed through mechanical packers using grout with consistencies ranging from a 5:1 to a 1:1 water-cement ratio, and a maximum pumping pressure of 100 pounds per square inch. A total of 187 holes and 9,347 lineal feet of rock drilled in this area accepted 10,995 sacks of cement for an average take of 1.17 sacks per lineal foot.

Three lines of B-holes in the thrust block (block 2) and spillway intake area were drilled from the foundation rock, with a few of the higher holes being drilled later through the intake structure concrete. The first few holes in this area accepted large quantities of grout with no noticeable leaks, but most of the remaining holes had a considerable amount of surface leakage from the top grouting stages. A total of 39 holes and 1,906 lineal feet of rock drilled in this area accepted 4,923 sacks of cement for an average take of 2.58 sacks per lineal foot of rock drilled.

The layout and grouting results of B-holes drilled from transverse adits within the dam are shown on figures 29 and 30. In general, these holes were grouted and closed out satisfactorily with very little leakage. Grout consistencies for these holes ranged from a 5:1 to a 1:1 water-cement ratio, with many of the holes being finished with relatively thin 5:1 or 4:1 grout. Pumping pressures at the lower packer settings were generally from 100 to 150 pounds per square inch with maximum pressures of 175 pounds per square inch being used on some holes. A total of 183 holes were drilled, 8,963 lineal feet in concrete and 27,871 lineal feet in rock. These holes accepted 28,757 sacks of cement for an average take of 1.03 sacks per lineal foot of rock drilled.

177. Deep Curtain Cutoff Grouting in A-Holes. Drilling of the first deep curtain cutoff holes from the foundation gallery was started in February 1962. A-hole drilling and grouting was performed continually in conjunction with other drilling and grouting operations until the final hole from the foundation gallery was grouted in September 1963.

Figure 200 illustrates the general layout of the "criss-cross" grouting pattern used in the A-hole grouting. All the holes inclining in one direction were drilled and grouted across the bottom of the dam before any of the holes inclining in the opposite direction were started. Some difficulty was experienced with this type of pattern, in that many of the previously grouted holes returned grout as holes from the opposite direction were grouted. Grouting of A-holes across the bottom portion of the dam to approximately elevation 5737 of the foundation gallery was satisfactorily performed in accordance with original grouting plans.

In February 1963, water seepage was noted on the right abutment at several locations just downstream from the dam fillet between elevations 5605 and 5710. When first noted, the total seepage was estimated to be about 25 gallons per minute. The rate of flow was found to increase with the rising reservoir head. In April 1963, seepage was noted at approximately elevation 5760 just downstream of the left abutment fillet. In an attempt to locate and cut off the seepage on both abutments, a program of deeper A-hole drilling and grouting was begun from block 7 on the left abutment and from block 19 on the right abutment. The deeper holes were started in areas on both abutments which had been previously grouted to a depth of 75 feet into rock on 10-foot centers. Holes were drilled on both abutments to intercept the 14 (14L and 14R) shale zones, and to a maximum depth of 260 feet. It was found that a satisfactory grout closure could not be obtained from these deeper holes at 10-foot spacing, and subsequently, grouting of both abutments above approximately elevation 5700 was completed with holes spaced at 5 feet on centers. The deeper cutoff curtain had no noticeable effect on the rate of seepage from either abutment. Other measures were taken in an attempt to control the leaks, and are discussed in sections 19, 178, and 179. A total of 268 A-holes were drilled from the landings, inclines and shafts of the foundation gallery and from the spillway intake through 41,944 feet of rock and concrete. These holes accepted 31,964 sacks of cement for an average take of 0.76 sack per lineal foot drilled. The average grout take in the foundation rock would be slightly higher than the above figure by considering only the total depth of rock drilled and neglecting the footage which was drilled through concrete.

178. Auxiliary A-Holes. In May 1963, four deep exploratory holes were drilled from the transverse grouting adits in blocks 19 and 20 in an attempt to intercept water seepage on the right abutment. The only hole which encountered any large amount of water was hole 19-7-4a which flowed approximately 24 gallons per minute. These holes were temporarily left open and then grouted in conjunction with other holes in the area.

Auxiliary A-hole grout curtains were drilled and grouted from transverse adits on both abutments. The left abutment auxiliary curtain holes accepted very little grout. Holes from the right abutment auxiliary curtain accepted more, but a satisfactory grout closure was obtained. The seepage was not affected by the auxiliary grout curtains on either abutment.

Additional exploratory deep holes were then drilled from the transverse adits farther downstream and nearer the toe of the dam. Some of these holes from the left abutment returned grout in the seepage areas, but no reduction in flow was noted, and it was felt that control of the left abutment seepage could not be obtained by grouting from transverse adits in the dam. Four exploratory holes were drilled from the block 19 adit at elevation 5737.5. These four holes leaked grout from seepage areas near the dam, and produced a decided reduction in the seepage. The total flow, within approximately 200 feet of the dam, was reduced to approximately 4 gallons per minute. Seepage further downstream on the right abutment was not reduced.

A total of 49 auxiliary and exploratory deep grout holes from both abutments were drilled through 10,090 feet of rock and concrete. These holes accepted 9,953 sacks of cement for an average take of 0.99 sack per lineal foot drilled. The general layouts for these holes are shown for the right and left abutments respectively on figures 8 and 9.

Water seepage on the right abutment probably originates from the Cart Creek area or follows a circuitous route deep in the abutment beyond the depths of the various grout cutoffs from the dam. This flow appears downstream on the right abutment through a system of vertical joints in the massive sandstone and travels down-dip (upstream) above various shale zones until it encounters the grouted foundation area near the dam and is forced to the surface. It is felt that the left abutment seepage is not directly connected with the reservoir, but is rather the same water which flowed from a spring at the base of the left abutment prior to construction of the dam. A combination of the rising groundwater table and the intersection by the dam and grout curtain with the down-dip groundwater flow apparently forces seepage to the surface downstream from the dam.

179. Right Abutment Deep A-Holes. When water seepage was encountered on the right abutment, the depth of the A-hole cutoff curtain was extended in an attempt to intercept the flow. The deeper holes drilled and grouted from the foundation gallery are discussed in section 183. Provision was made at the same time for additional deep holes to be

drilled from the roadway beyond the dam in order to extend the cutoff curtain on the right abutment. The general layout of this extended grouting pattern is shown on figure 201.

When large grout takes were encountered in the first hole drilled from the roadway, it was decided to extend the depth of these holes even further, and most of the holes were finally drilled to a depth of 410 feet. In all, 17 holes were drilled from the roadway shoulder, and 2 holes were drilled from the control cable tunnel. Grout takes in this area continued to be rather heavy. Drilling water loss in many of these holes required the drilling and grouting to be performed in stages. In addition to drilling deeper holes, it was necessary to drill extra holes at 10-foot spacing in order to obtain a satisfactory grout closure throughout much of this area. Several of the holes leaked grout from the abutment rock downstream from the dam, but most of the holes were grouted at depth with no leaks observed. Although maximum pumping pressures of up to 450 pounds per square inch were used in some holes, the heavy grout takes in this area required that particular care be taken to avoid displacement of the foundation rock. Pressures were reduced and grout consistencies thickened to avoid the possibility of displacement. Much of the grout pumped in these holes had a 3:1 water-cement ratio by volume. A total of 36,715 sacks of cement was placed in the 19 holes which were drilled a total of 7,038 lineal feet for an average take of 5.22 sacks per lineal foot. All drilling and grouting operations in this area were completed between June and October 1963.

180. Right Abutment Beyond End of Dam. A grout cutoff curtain was extended from the end of the dam through the right abutment parking area roughly parallel to the reservoir rim just upstream from the dam. Much of the surface rock through this area is shattered, and grout takes were heavy in the relatively shallow holes. The layout for this curtain is shown on figure 7. The first hole in this area was drilled on October 30, 1962, and most of the curtain was completed by the end of February 1963 when outside drilling and grouting operations were discontinued because of cold weather. When activities were resumed in June 1963, drilling difficulties were encountered in attempting to drill through the extensive sandy overburden found in the few remaining holes at the upstream end of the pattern, and as a result the last three holes in the pattern were not drilled.

Because of the shattered condition of the rock in this area, there was considerable grout leakage to the surface in the immediate area of some of the holes as they were grouted. Fifty-nine holes in this curtain were drilled a total of 4,568 lineal feet and accepted 21,631 sacks of cement for an average take of 4.73 sacks per lineal foot. Most of this grout was placed in the top 50 feet of the holes, and although pumping pressures as high as 150 pounds per square inch were used, most of the grouting was performed with low pressures and heavy grout because of the extensive leaks.

181. Spillway Tunnel. Foundation grouting and backfill grouting in the spillway tunnel were performed in August through October 1962.

The five grout rings, each with six radial holes 20 feet deep, located near the upper end of the spillway tunnel had an average take of 1.48 sacks per foot. Several leaks from this grouting occurred in the exposed shale seams in the canyon wall adjacent to this area. Most of the remaining holes in the elbow and horizontal sections of the tunnel were relatively tight. A total of 72 foundation grout holes in the tunnel were drilled 1,401 lineal feet and accepted 1,272 sacks of cement for an average take of 0.91 sack per lineal foot. Maximum pumping pressure for this grouting was 100 pounds per square inch.

Backfill grouting was performed only in the horizontal section of the tunnel between stations 4+60 and 6+81. A total of 162 sacks of cement was placed for an average take of 1.36 sacks per lineal foot of tunnel. Drilling in the spillway tunnel was accomplished from a rubber-tired jumbo, and the grout plant was set up near the spillway intake area.

182. Shale Seam Backfill Grouting. Mortar was placed by grouting methods to complete the concrete backfill of tunnels excavated in shale seams for the purpose of constructing cutoffs. Five tunnels on the left abutment and 10 on the right abutment were treated in this manner. Following is a summary of cement placed in backfill grouting these tunnels:

Shale seam tunnel	Sacks placed	Shale seam tunnel	Sacks placed
9L	3	6R Downstream	25
12L Downstream	78	6R Upstream	1,470
12L Upstream	32	7R.Downstream	27
14L Downstream	122	7R Upstream	3,668
14L Upstream	286	9R Downstream	105
14R Downstream	48	9R Upstream	893
14R Upstream	0	12R Downstream	166
		12R Upstream	0

A total of 6,923 sacks of cement was placed.

The large grout takes in the 6R, 7R and 9R upstream tunnels indicate that some grouting of the foundation rock was accomplished in this area in addition to backfilling the tunnels. This possibility is further borne out by the condition of the foundation rock near the heel of the dam high on the right abutment. The rock in these areas was broken and intersected by vertical cracking or shear zones. Low pressures and heavy grout were used in backfilling the shale seam cutoff tunnels.

183. Foundation Drainage. Foundation drainage for the dam was provided by diamond drilling 3-inch-diameter (NX) holes into the foundation rock downstream from the grout cutoff curtains. Drainage holes from the spillway intake structure are shown on figure 202. Drainage holes from the foundation gallery in the dam are shown on figure 203.

The holes above approximately elevation 5670 were drilled deeper than originally planned after the depth of the grout cutoff curtain was increased in these areas. Additional drainage was provided downstream from the auxiliary A-hole patterns by holes drilled from the transverse adits. These holes are shown for the left and right abutments, respectively, on figures 12 and 13. In addition to the above holes, 11 vertical drain holes were drilled from the penstock gallery in the powerplant structure on the dam. Wherever possible, the foundation drain holes were drilled to be self-draining. The B-holes drilled from transverse adits downstream from the A-hole curtain were not backfilled and provide some additional drainage. Several of the drains have shown a tendency to fill up and plug off with mineral deposits. This will probably result in a continuing maintenance problem to keep the drain holes clear.

In addition to the foundation drainage holes from the dam, 12 holes were drilled from the diversion tunnel in an attempt to intercept the right abutment drainage. These holes are shown on figure 14. Drilling of the diversion tunnel drainage holes was performed with a diesel-powered diamond drill from a truck-mounted jumbo. The total waterflow from the diversion tunnel was about 115 gallons per minute with most of this flow coming from the drainage holes drilled in line T-3.

184. Miscellaneous Drilling and Grouting. In September 1962, the concrete lining of the control cable tunnel was backfill grouted. A total of 240 sacks of cement was placed in approximately 220 lineal feet of tunnel for an average take of 1.09 sacks per lineal foot of tunnel.

A gravel drain placed on the foundation rock under the tailrace training wall was grouted and took 100 sacks of cement. The subcontractor also performed various other minor drilling operations under this contract.

185. Summary. The following tabulation is a summary of the drilling and grouting operations quantities:

Feature	Drilling, lineal feet	Cement placed, sacks
B-holes drilled from rock	9,347	10,995
B-holes from transverse adits	36,834	28,757
B-holes, spillway intake and thrust block	1,906	4,923
A-holes	41,944	31,964
Auxiliary A-holes	10,090	9,953
Right abutment deep A-holes	7,038	36,715
Right abutment parking area cutoff	4,568	21,631
Spillway tunnel	1,401	1,272
Diversion tunnel	2,438	1,085
Grout hole redrilling (one-half of total)	1,877	--
Diversion tunnel backfill	--	1,384
Spillway tunnel backfill	--	162
Shale seam tunnel backfill	--	6,923
Training wall drain	--	100
Control cable tunnel backfill	--	240
Foundation drainage holes (NX) including uplife pressure holes	18,652	--
Drain holes to diversion tunnel (NX)	2,542	--

Total foundation grouting footage . . . . . 117,443 feet
Total drain hole footage . . . . . 21,194 feet
Total cement for grouting (excluding contraction joint grouting) . . . 156,104 sacks

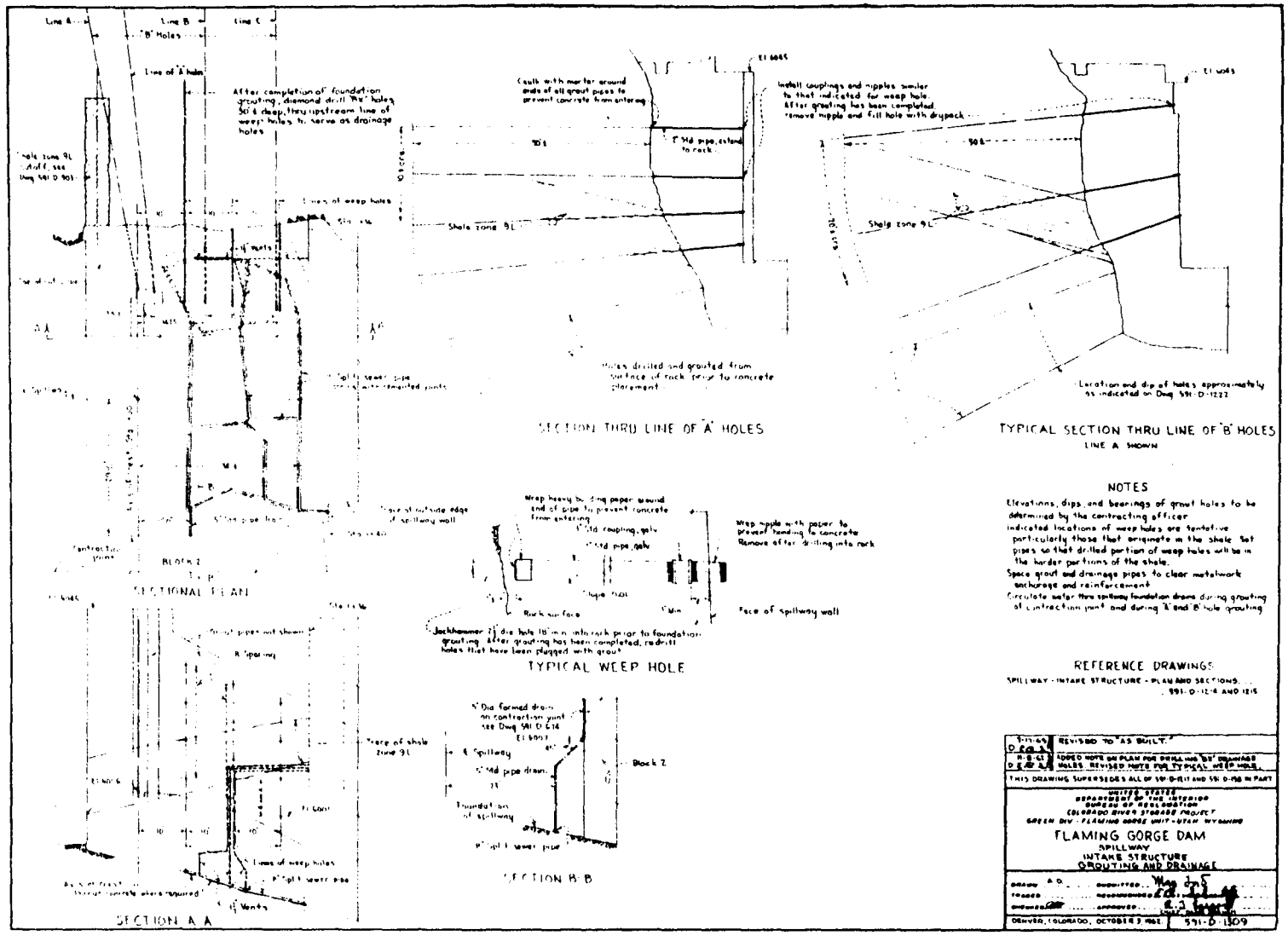


Figure 202. --Spillway intake structure--Grouting and drainage system.

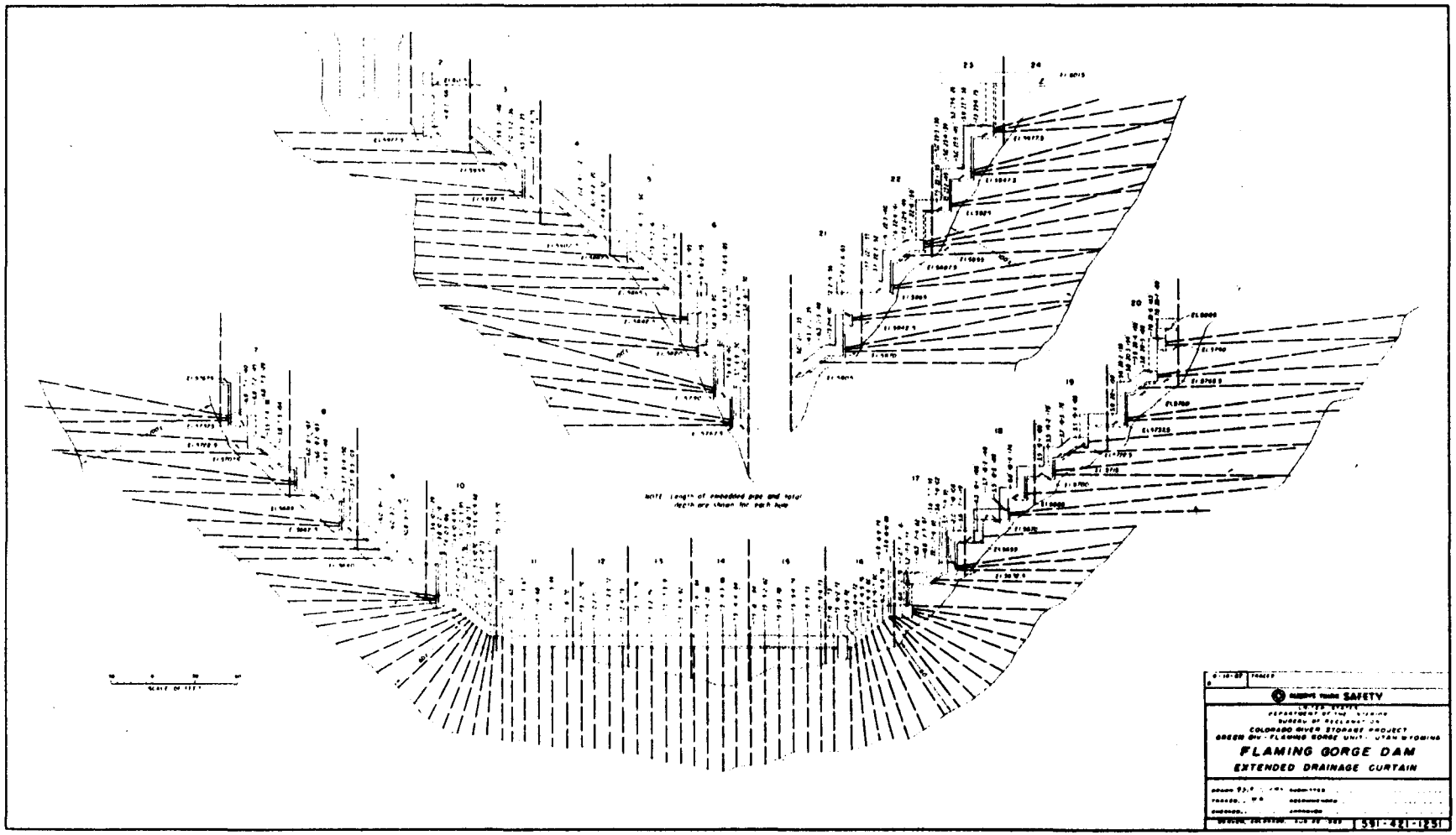


Figure 203. --Extended drainage curtain under dam.

APPENDIX E

DOCUMENTS REVIEWED

1. Technical Record of Design and Construction, Flaming Gorge Dam and Powerplant, Bureau of Reclamation, Denver, Colorado, January 1968.

2. Technical Data Workbook, Flaming Gorge Dam, prepared by Robert Spencer, Goodson & Associates, Inc., June 26, 1981.

3. Specifications No. DC-5045, Flaming Gorge Dam and Powerplant.

4. Summary of bids, award of contract and execution of the work, Flaming Gorge Dam and Powerplant.

5. Final Report, Foundation Drilling, Grouting and Drainage, Technical Installations, Concrete Cooling Contraction Joint Grouting, Bureau of Reclamation, Region 4, June 1964.

6. The Geology of The Flaming Gorge - Final Report, 1963.

7. Speedletter dated March 4, 1960, from Assistant Commissioner and Chief Engineer to Project Construction Engineer, Dutch John, Utah, subject: "Additional Excavation for right abutment blocks 21, 22, and 23 Flaming Gorge Dam".

8. Travel report dated March 21, 1960 from L. G. Puls and W. H. Irwin to Assistant Commissioner and Chief Engineer, subject: "Inspection of keyway excavations - Flaming Gorge Dam".

9. Travel report dated May 4, 1960 from L. G. Puls, O. L. Rice, W. H. Irwin and E. R. Schultz, subject: "Inspection of the keyway excavations for Flaming Gorge Dam".

10. Letter report dated June 8, 1960 from John W. Vanderwilt, Edward B. Burwell, Jr., and John H. Hammond, Acting Chairman of Board of Consultants, to Mr. Grant Bloodgood, Assistant Commissioner and Chief Engineer, subject: "Report on Foundation Adequacy and Design Considerations of Flaming Gorge Dam".

11. Letter report dated June 15, 1960 from Julian Hinds (member of board of consultants), to Mr. Grant Bloodgood concerning site visit of June 13 and 14.

12. Travel report dated June 15, 1960 from L. G. Puls, W. H. Irwin, and E. R. Schultz, subject: "Inspection of Flaming Gorge damsite".

13. Travel report dated September 6, 1960 from Oscar Rice and Samuel Thompson, subject: "Inspection of foundations and abutments for Flaming Gorge Dam and Powerplant".

14. Travel report dated March 30, 1961 from L. G. Puls and W. H. Irwin, subject: "Inspection of foundation excavations, Flaming Gorge Dam".

15. Letter report dated October 16, 1961 from Board of Consultants (Raymond E. Davis, John J. Hammond, John W. Vanderwilt, Edward B. Burwell, Jr., and Julian Hinds, Chairman) to Grant Bloodgood, Assistant Commissioner and Chief Engineer.

16. Draft SEED Report, Flaming Gorge Dam, undated.

17. Geologic logs of drill holes Nos. 100 through 121, 123 through 128, 138, 139, 140 and 156 through 162.

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