GR-86-4

ANALYSIS OF UTILIZATION OF GROUT AND GROUT CURTAINS — HUNGRY HORSE DAM

February 1986
Engineering and Research Center

U.S. Department of the Interior
Bureau of Reclamation
Division of Research and
Laboratory Services
Concrete and Structural Branch

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16. ABSTRACT

The foundation grouting program at Hungry Horse 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– HUNGRY HORSE 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

HUNGRY HORSE DAM

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ANALYSIS OF UTILIZATION OF GROUT AND GROUT CURTAINS

HUNGRY HORSE DAM

I. INTRODUCTION

1. General description. Hungry Horse Dam is located in Northwestern Montana on the South Fork of the Flathead River about 5 miles above the confluence with the Flathead River. The dam is a concrete gravity-arch structure, referred to sometimes as a thick-arch structure. It has a height of 564 feet above the lowest point in the foundation. The crest is at elevation 3,565 feet, and the crest length is 2,115 feet. The crest width is 24 feet and the maximum base thickness is 320 feet. See figure 1.

The spillway is the morning-glory type with a steep-inclined lined tunnel through the right abutment which discharges downstream of the power plant. There are three 96-inch round river outlets on the right bank. The power outlets consist of four 162-inch-diameter penstocks through the center of the structure. The power plant is located at the downstream toe of the dam.

The contract for the construction of the dam was awarded on April 12, 1948 and the work was completed on July 15, 1953. The work area was protected by crib-filled cofferdams and the river was diverted through a tunnel in the right abutment. The diversion tunnel was unlined except for the downstream 250 feet which was later used as part of the spillway tunnel.

The reservoir is operated normally near the crest of the spillway ring gate (El. 3,560), thereby maintaining a high

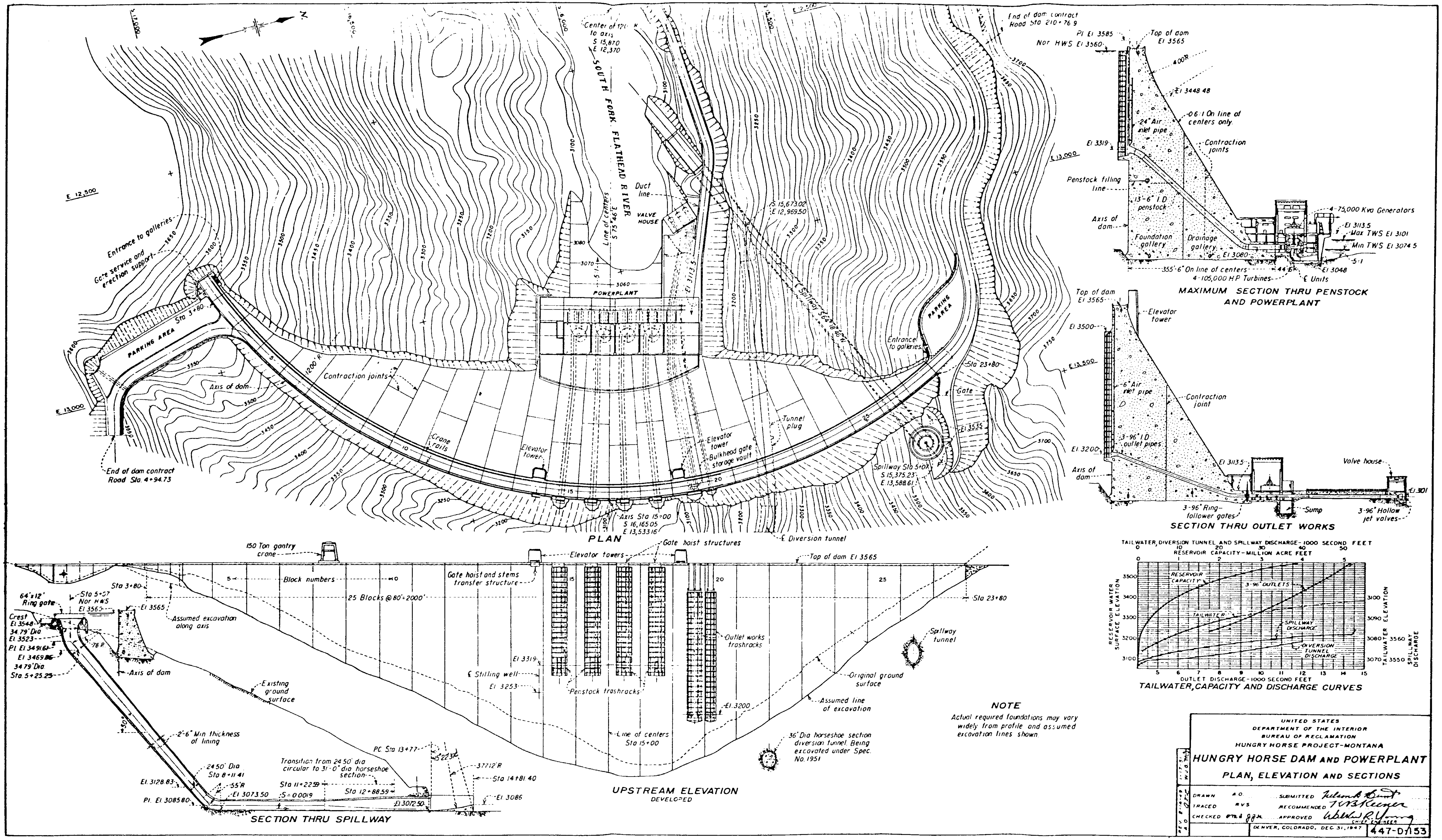

SOUTH FORK FLATHEAD RIVER
POWERPLANT
VALVE HOUSE
PARKING AREA Sta 3+80
PARKING AREA
Entrance to galleries
Gate service and erection support
Contraction joints
Axis of dam
Crane rails
Elevator tower
Elevator tower
Bulkhead gate storage vault
Tunnel plug
Duct line
Gate
End of dam contract Road Sta. 4+94.73
End of dam contract Road Sta 210+76.9
Center of 1200 R to axis S 15,870 E 12,370
Axis Sta 15+00 S 16,165.05 E 13,533.16
Spillway Sta 5+0X S 15,375.23 E 13,588.61
S 15,673.02 E 12,969.50
Sta 23+80
Diversion tunnel
PLAN
150 Ton gantry crane
Elevator towers
Gate hoist structures
Top of dam El 3565
Block numbers
25 Blocks @ 80' = 2000'
Gate hoist and stems transfer structure
Assumed excavation along axis
Existing ground surface
Stilling well
Penstock trashracks
Outlet works trashracks
Spillway tunnel
Original ground surface
Assumed line of excavation
Line of centers Sta 15+00
36" Dia horseshoe section diversion tunnel. Being excavated under Spec. No. 1951
UPSTREAM ELEVATION
DEVELOPED
64' x 12' Ring gate
Crest El 3548
34.79' Dia
2'-6" Min thickness of lining
Transition from 24.50' dia circular to 31'-0" dia horseshoe section
SECTION THRU SPILLWAY
Top of dam El 3565
Nor HWS El 3560
24" Air inlet pipe
13'-6" I.D. penstock
Foundation gallery
Drainage gallery
4-75,000 Kva Generators
4-105,000 H.P. Turbines
Max TWS El 3101
Min TWS El 3074.5
MAXIMUM SECTION THRU PENSTOCK AND POWERPLANT
Elevator tower
6" Air inlet pipe
Contraction joint
3-96" I.D. outlet pipes
Valve house
3-96" Ring-follower gates
Sump
3-96" Hollow jet valves
SECTION THRU OUTLET WORKS
TAILWATER, DIVERSION TUNNEL AND SPILLWAY DISCHARGE - 1000 SECOND FEET
RESERVOIR CAPACITY - MILLION ACRE FEET
RESERVOIR CAPACITY
3-96" OUTLETS
TAILWATER
SPILLWAY DISCHARGE
DIVERSION TUNNEL DISCHARGE
OUTLET DISCHARGE - 1000 SECOND FEET
RESERVOIR WATER SURFACE ELEVATION
TAILWATER ELEVATION
SPILLWAY DISCHARGE
TAILWATER, CAPACITY AND DISCHARGE CURVES
NOTE
Actual required foundations may vary widely from profile and assumed excavation lines shown.
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
HUNGRY HORSE PROJECT-MONTANA
HUNGRY HORSE DAM AND POWERPLANT
PLAN, ELEVATION AND SECTIONS
DENVER, COLORADO, DEC. 31, 1947
447-D-153

FIGURE 1

hydraulic head on the dam most of the time. The hydraulic height of the dam is 515 feet.

A board of consultants was engaged on the project during the design stage. The board consisted of J. L. Savage, Chairman; Dr. Charles P. Berkey; and William H. Irwin. J. L. Savage was formerly Chief Designing Engineer for the Bureau. Dr. Charles P. Berkey was a geologist and was the founder of the branch of engineering geology. William H. Irwin was a Bureau geologist, Denver office. Savage and Berkey were engaged as a consulting board during construction.

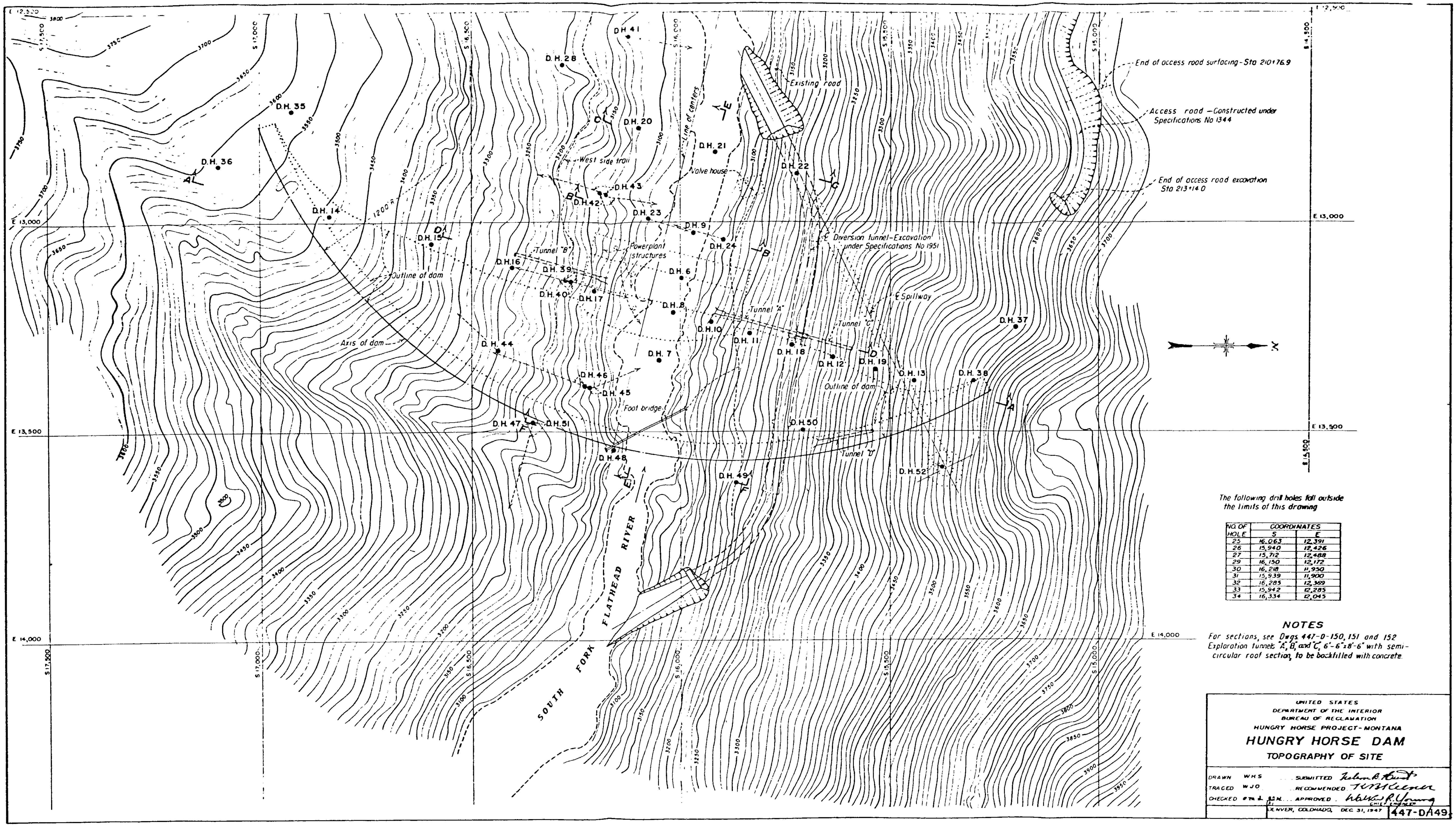
The photographs in this report were reproduced from the Final Construction Report; the numbers for the figures and pages have been changed for assimilation into this report.

II. GEOLOGY

2. General. Chapter II -- "Geology", Part II -- "Foundation Conditions and Treatment", Technical Record of Design and Construction is reproduced in appendix B. Chapter II covers both the geology developed in preconstruction geologic investigations and the geology developed during construction.

The plan of exploration is shown on figure 2, and the preconstruction geologic section for axis G-G is presented on figure 3. The section for axis G-G is drawn along section D-D of figure 2, which approximates the downstream toe of the constructed dam.

From foundation treatment and grouting standpoints, it is important that the limestone at Hungry Horse Dam is dolomitic, argillaceous and siliceous, which precludes it from being highly soluble as are purer varieties of limestone.



The following drill holes fall outside the limits of this drawing

NO. OF HOLE	COORDINATES	
	S	E
25	16,063	12,391
26	15,940	12,426
27	15,712	12,488
29	16,150	12,172
30	16,218	11,950
31	15,939	11,900
32	16,285	12,369
33	15,942	12,285
34	16,334	12,045

NOTES

For sections, see Dwgs 447-D-150, 151 and 152
Exploration tunnels "A", "B", and "C", 6'-6"x8'-6" with semi-circular roof section, to be backfilled with concrete.

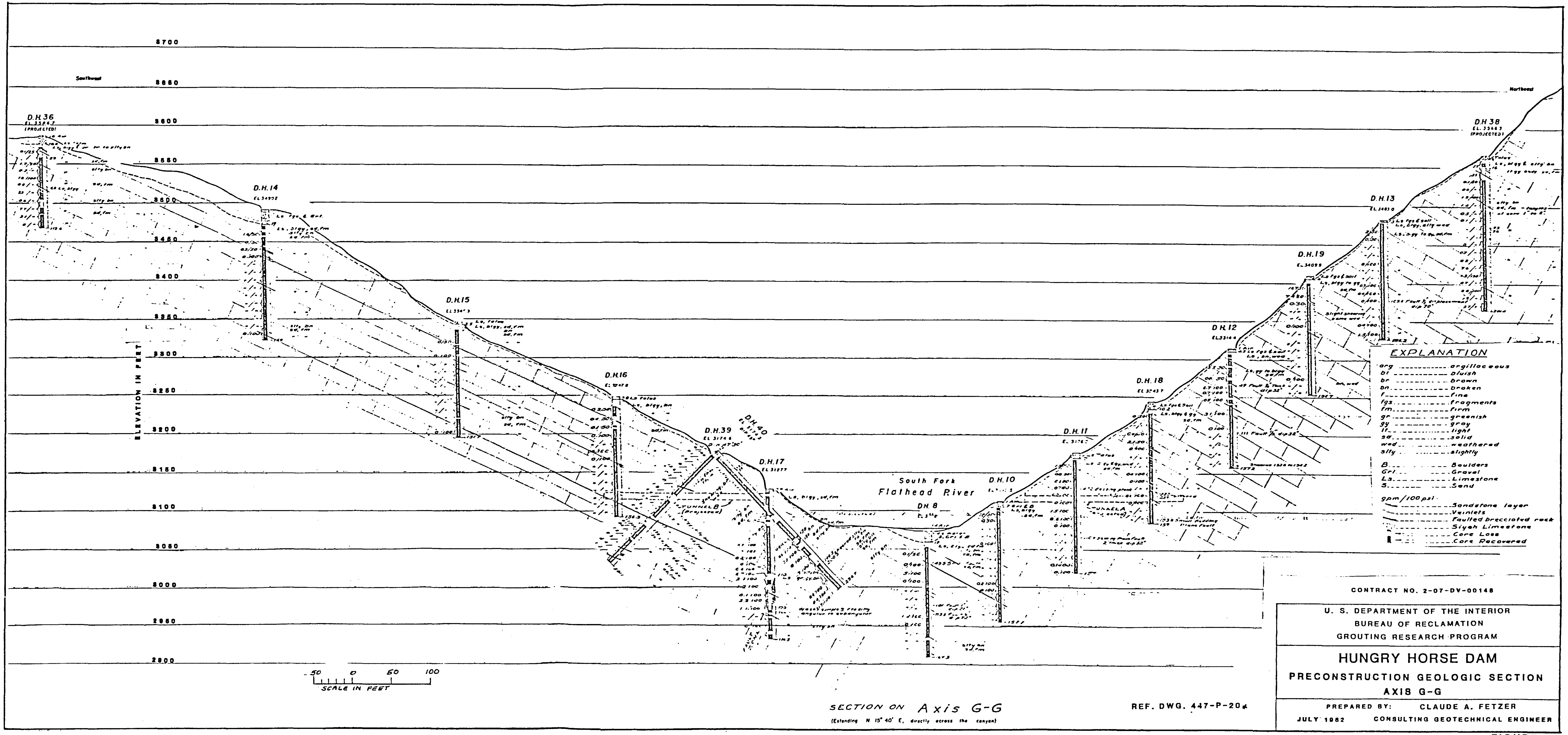
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
HUNGRY HORSE PROJECT-MONTANA

HUNGRY HORSE DAM

TOPOGRAPHY OF SITE

DRAWN W.H.S. SUBMITTED Kelson B. Hunt
TRACED W.J.O. RECOMMENDED
CHECKED APPROVED Walker R. Young CHIEF ENGINEER
DENVER, COLORADO, DEC. 31, 1947 447-D-149

FIGURE 2



3700
3650
3600
3550
3500
3450
3400
3350
3300
3250
3200
3150
3100
3050
3000
2950
2900
Southwest
Northeast
ELEVATION IN FEET
D.H.36
(PROJECTED)
D.H.14
D.H.15
D.H.16
D.H.39
D.H.40
D.H.17
TUNNEL B (Projected)
South Fork
Flathead River
D.H.8
D.H.10
D.H.11
TUNNEL A
D.H.18
D.H.12
D.H.19
D.H.13
D.H.38
(PROJECTED)
EXPLANATION
arg ---------- argillaceous
bl ---------- bluish
br ---------- brown
bn ---------- broken
f ---------- fine
fgs ---------- fragments
fm ---------- firm
gr ---------- greenish
gy ---------- gray
lt ---------- light
sd ---------- solid
wed ---------- weathered
slty ---------- slightly
B ---------- Boulders
Grl ---------- Gravel
Ls ---------- Limestone
S ---------- Sand
gpm/100 psi
---------- Sandstone layer
---------- Veinlets
---------- Faulted brecciated rock
---------- Siyeh Limestone
---------- Core Loss
---------- Core Recovered
50 0 50 100
SCALE IN FEET
SECTION ON Axis G-G
(Extending N 15° 40' E, directly across the canyon)
REF. DWG. 447-P-20
CONTRACT NO. 2-07-DV-00148
U. S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GROUTING RESEARCH PROGRAM
HUNGRY HORSE DAM
PRECONSTRUCTION GEOLOGIC SECTION
AXIS G-G
PREPARED BY: CLAUDE A. FETZER
JULY 1982 CONSULTING GEOTECHNICAL ENGINEER

FIGURE 3

III. FOUNDATION TREATMENT AND GROUTING

3. General. Chapter III -- "Grouting" and Chapter XIII -- Construction -- "Excavation and Preparation of Site", Technical Record of Design and Construction have also been reproduced in appendix B.

4. Excavation methods. The left abutment had bedding planes parallel to the slope. The contractor, as shown on figure 4, effectively used sluicing to remove the weathered rock. A closeup view of the rock in the left abutment is shown on figure 5. A view showing the completed left abutment and early stages of fault zone 6 is presented on figure 6.

The bedding planes in the right abutment were at an approximate angle of 55° to the slope. The excavation was made using 100-foot long diamond drill holes with the results as shown on figures 7 and 8. Jackhammer teams were used to remove dangerous rock from the upstream shoulder as shown on figure 9. The completed right bank excavation is shown on figure 10.

5. Treatment of fault zones. Three minor faults, Nos. 1, 2 and 5, and three major faults, Nos. 3, 4, and 6, were encountered in the foundation area at the locations shown on figures 11 and 12. Treatment of the minor faults consisted of trenching the crushed material to a depth equal to 1-1/2 to 2 times the width of the zone for the full width of the dam and backfilling with concrete. The fault zones were also cross-stitched with grout holes for the full width of the dam.

The foundation treatment for the major fault zones consisted of trenching the fault for the full width of the dam to depths

Figure 4 Foundation excavation on the left abutment. The contractor effectively used a Joshua Hendy "Hydraulic Giant" for sluicing the bedrock on this side. P447-105-1219, May 20, 1949.

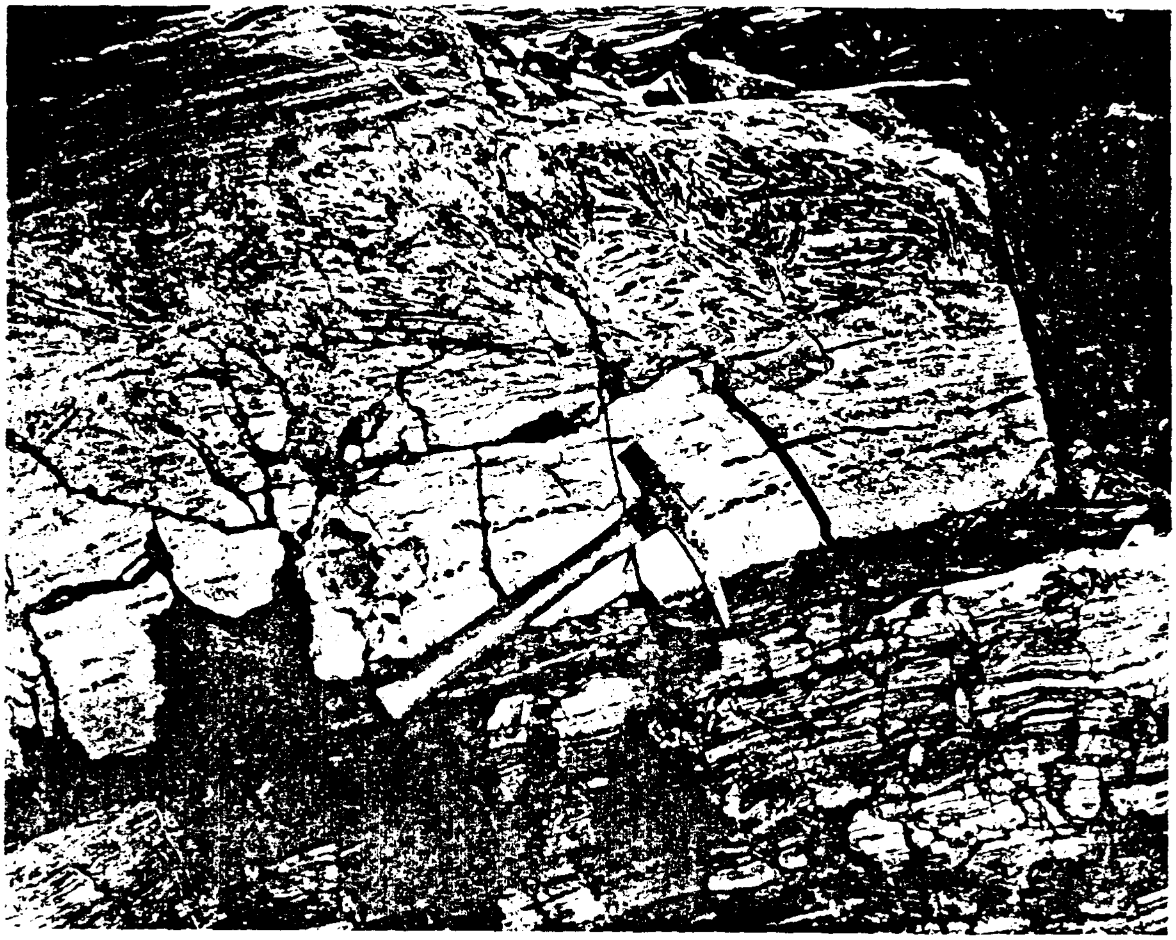


Figure 5 Detailed view in left abutment excavation showing bedding lines and partings and some of the variability of thickness of individual rock layers. Blocky nature of fractured rock is shown, but irregular fractures have been caused by disturbances in excavation. The strikingly marked internal appearance of the central massive bed is a very unusual occurrence and is believed to represent influence of some contemporaneous organic growth. P447-105-1225, May 25, 1949.

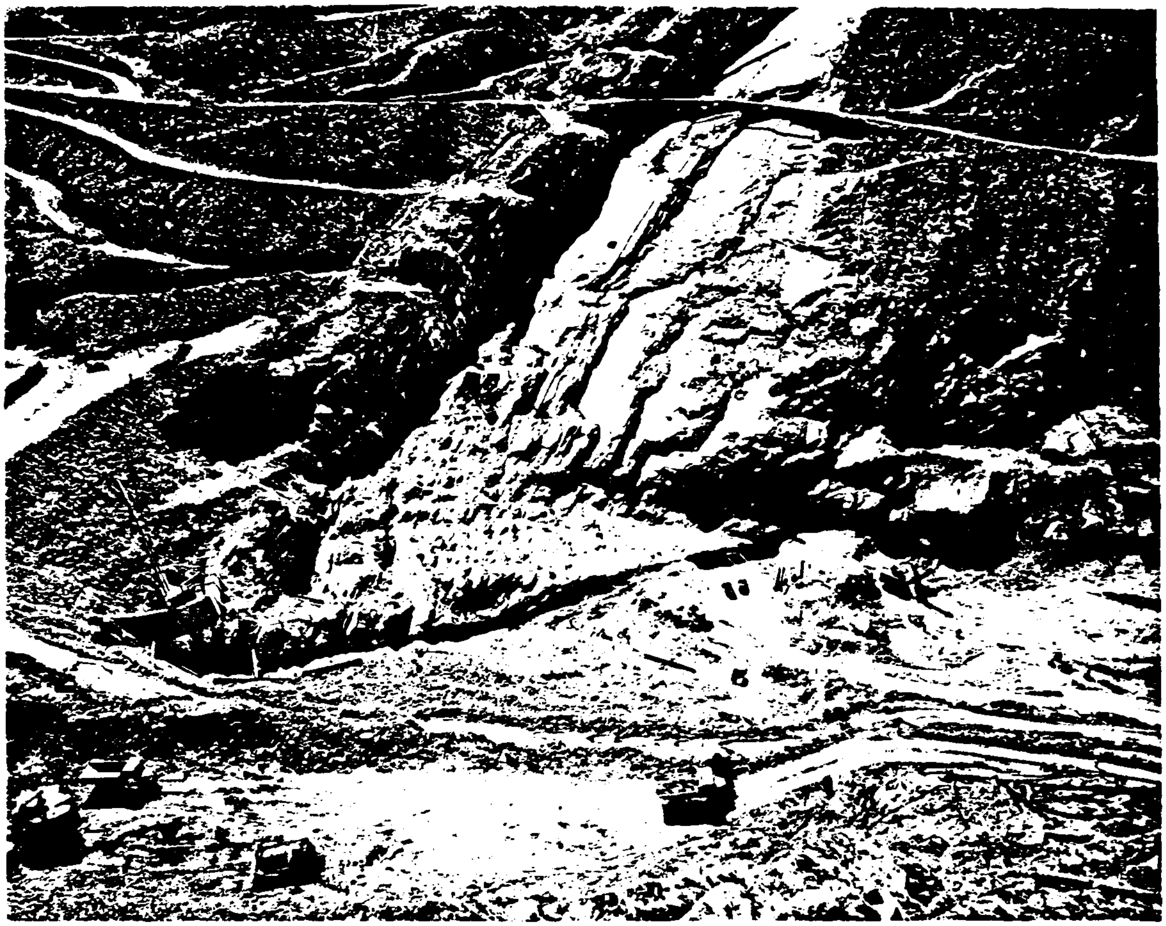


Figure 6 View showing completed left abutment foundation excavation, early stages of Fault Zone 6 excavation marked by diagonal trench in river channel, and rock work on the right abutment and for the powerhouse foundation. P447-105-1631, August 26, 1949.

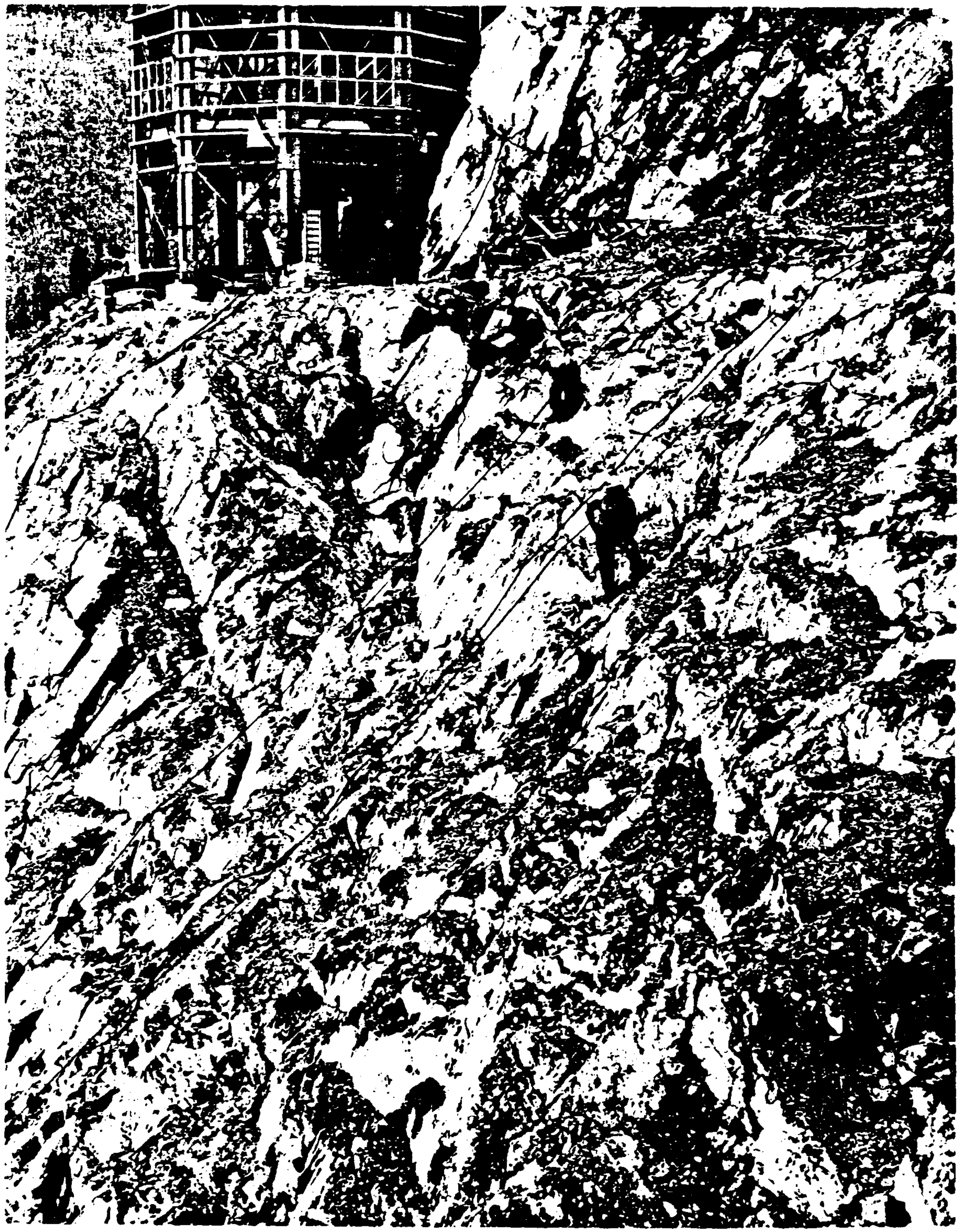


Figure 7. Right abutment foundation excavation showing results of blasting of the 100-foot long diamond drill holes in Blocks 24 and 25, between approximate elevations 3300 and 3430. P447-105-1380, July 7, 1949.

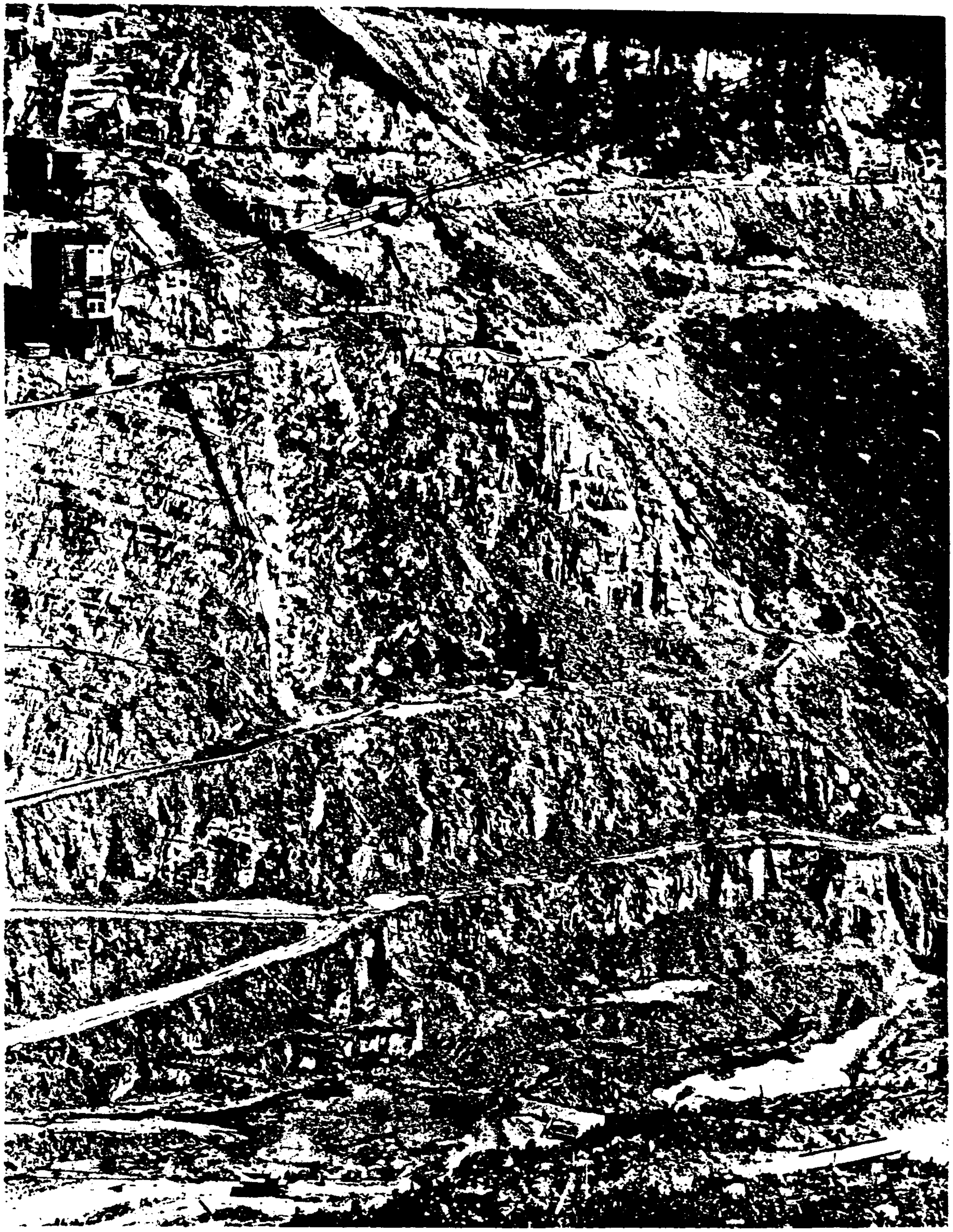
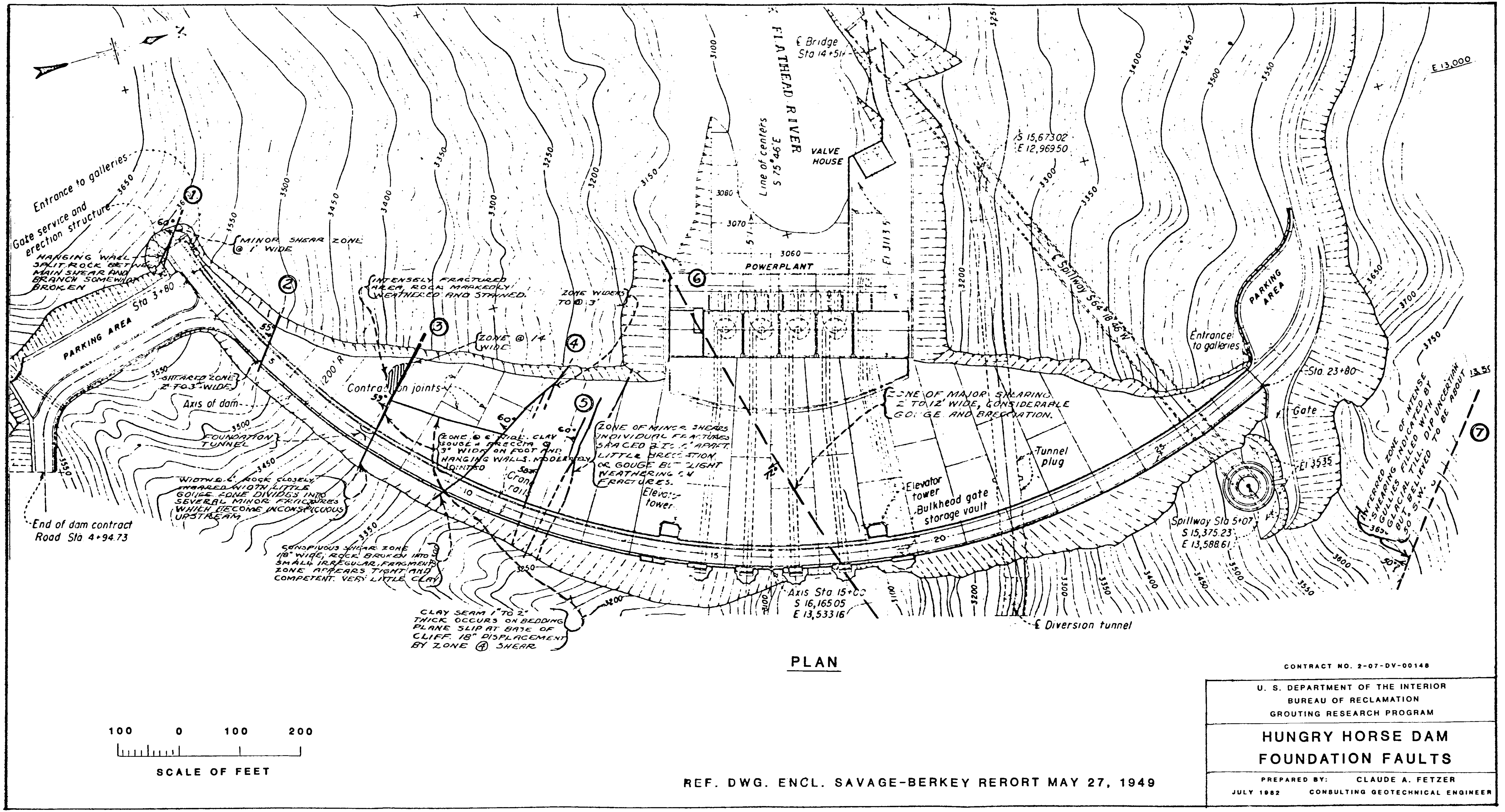


Figure 8 Dam and powerhouse excavation on right bank and in the river channel. Note results of blasting of 100-foot long diamond drill holes to the right and below concrete mixing plant. P447-105-1397, July 13, 1949.

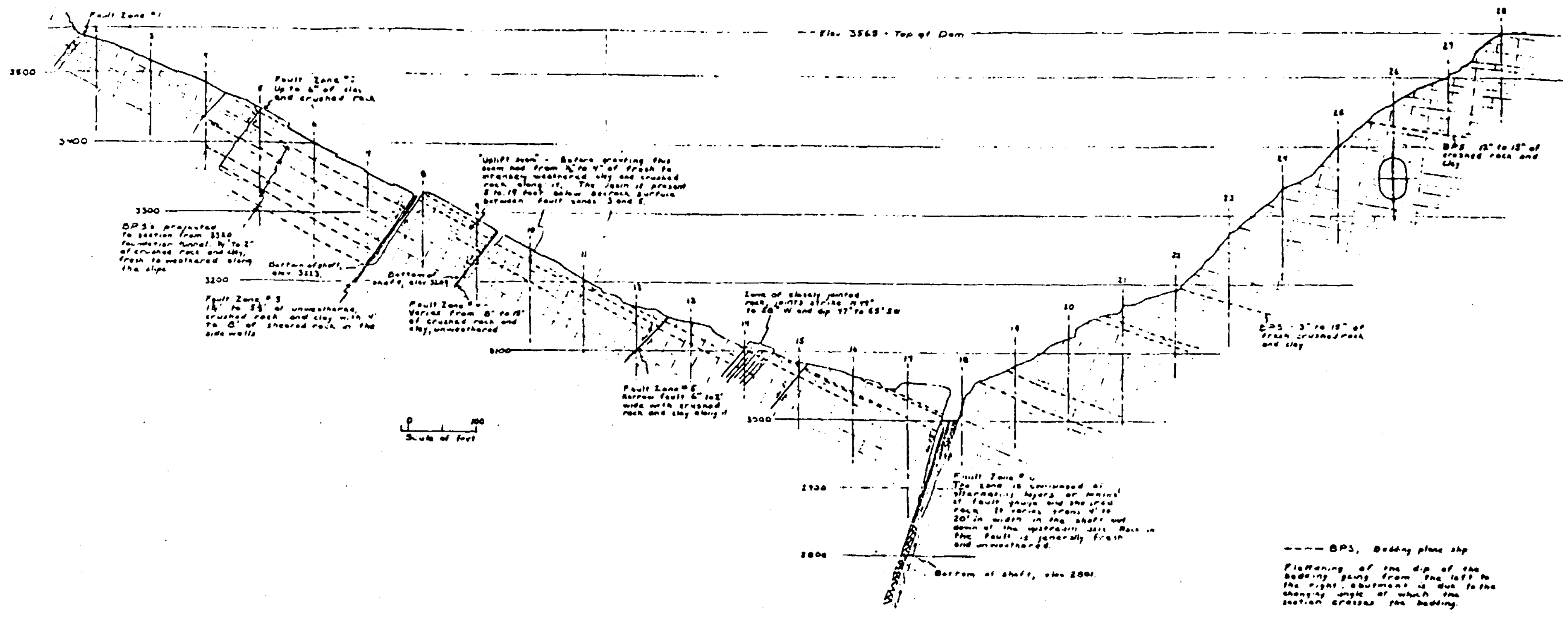
Figure 9 Jackhammer teams drilling in preparation for blasting of dangerous rock from upstream shoulder of right bank foundation excavation. P447-105-1633, August 26, 1949.

Figure 10 View of completed right bank excavation. Upstream and downstream cofferdams are under construction. Concrete placement is in its early stages in the river channel. P447-105-1806, October 13, 1949.

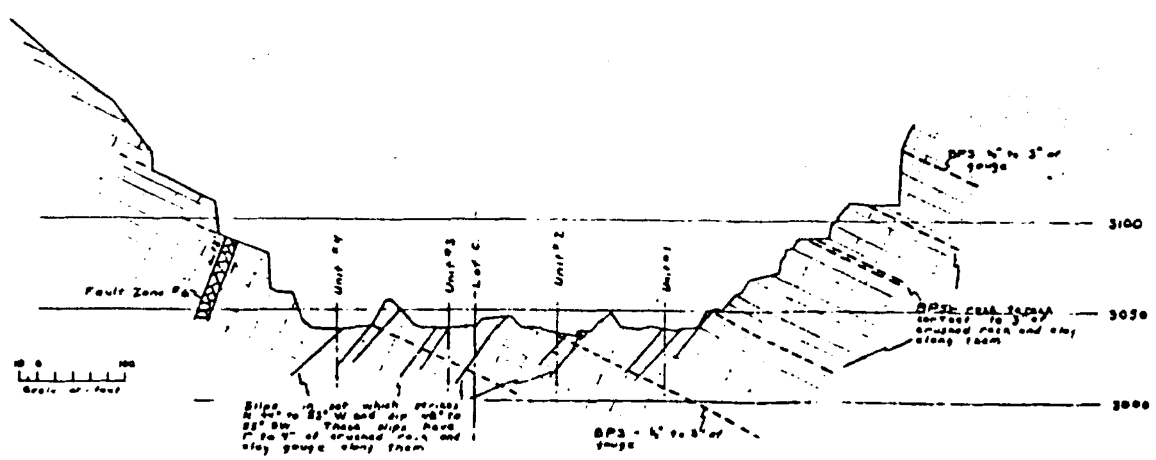


PLAN
HUNGRY HORSE DAM
FOUNDATION FAULTS
U. S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GROUTING RESEARCH PROGRAM
PREPARED BY: CLAUDE A. FETZER
JULY 1982 CONSULTING GEOTECHNICAL ENGINEER
CONTRACT NO. 2-07-DV-00148
REF. DWG. ENCL. SAVAGE-BERKEY RERORT MAY 27, 1949
100 0 100 200
SCALE OF FEET
FLATHEAD RIVER
Line of centers
VALVE HOUSE
POWERPLANT
PARKING AREA
Entrance to galleries
Gate service and erection structure
Sta 3+80
Axis of dam
FOUNDATION TUNNEL
End of dam contract Road Sta 4+94.73
Contraction joints
Elevator tower
Bulkhead gate storage vault
Tunnel plug
Sta 23+80
Gate
El 3535
Spillway Sta 5+07
S 15,375.23
E 13,588.61
S 15,673.02
E 12,969.50
E 13,000
Axis Sta 15+00
S 16,165.05
E 13,533.16
℄ Diversion tunnel
℄ Bridge Sta 14+51
MINOR SHEAR ZONE @ 1' WIDE
HANGING WALL SPLIT, ROCK BETWEEN MAIN SHEAR AND BRANCH SOMEWHAT BROKEN
INTENSELY FRACTURED AREA, ROCK MARKEDLY WEATHERED AND STAINED.
ZONE WIDENS TO @ 3'
ZONE @ 14' WIDE
SHATTERED ZONE 2' TO 3' WIDE
ZONE OF MAJOR SHEARING 2' TO 12' WIDE, CONSIDERABLE GOUGE AND BRECCIATION.
ZONE OF MINOR SHEARS INDIVIDUAL FRACTURES SPACED 3' TO 4' APART. LITTLE BRECCIATION OR GOUGE BUT SLIGHT WEATHERING ON FRACTURES.
INFERRED ZONE OF INTENSE SHEARING INDICATED BY GULLIES FILLED WITH GLACIAL TILL. DIP UNCERTAIN BUT BELIEVED TO BE ABOUT 50° S.W.
CONSPICUOUS SHEAR ZONE 18" WIDE, ROCK BROKEN INTO SMALL IRREGULAR FRAGMENTS. ZONE APPEARS TIGHT AND COMPETENT. VERY LITTLE CLAY
CLAY SEAM 1" TO 2" THICK OCCURS ON BEDDING PLANE SLIP AT BASE OF CLIFF. 18" DISPLACEMENT BY ZONE ④ SHEAR

FIGURE 11



Geologic Section Along Axis of Dam



Geologic Section on Longitudinal ℄ of Units

UNITED STATES
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HUNGRY HORSE PROJECT, MONTANA

HUNGRY HORSE DAM
GEOLOGIC SECTIONS

DRAWN Y.S.S. SUBMITTED [illegible]
TRACED Y.S.S. RECOMMENDED [illegible]
CHECKED APPROVED [illegible]

447-105-2325

FIGURE 12

equal to 1-1/2 to 2 times the width of the zone and excavating cutoff shafts at the upstream and downstream toes of the dam. The shafts were to have sufficient depth to provide a contact length between the concrete and crushed material equal to not less than 2-1/2 and not more than 4 times the maximum hydraulic head on the dam (maximum reservoir water surface minus minimum tailwater). The contact length included the contact between the base of the dam and the excavated trench. The upstream cutoff was made twice the depth of the downstream cutoff. Considerable grouting was accomplished in the cutoff shafts and along the walls of the cutoff trench. Excavation along fault zone No. 6 is shown on figures 13 and 14.

The required percolation distances for faults 3 and 4 were based on the net head at their locations as these two faults were located on the left abutment at a considerable height above tailwater. The actual depths of the shafts and the percolation distance were obtained from data in the Geologic Report on the Hungry Horse Dam site by Kenneth Soward(1) dated September 1952 and are tabulated below:

Fault No.	Net Head Feet	2.5 x NH Feet	Depth of Shafts U.S. Feet	Depth of Shafts D.S. Feet	Percolation Length Feet	Ratio $\frac{PL}{NH}$
3	230	575	130	66	550	2.39
4	331.2	828	78	46	528	1.59
6	491.6	1,229	208	95	1,095	2.23

The minimum ratio of 2.5 was specified in the Savage-Berkey(2) report of May 27, 1949; however after further examination of fault No. 6 on September 4 and 5, 1949, Consulting Engineer J. L. Savage(3)

Figure 13 Dam foundation excavation. The trench marks the early stages of excavation of Fault Zone 6 in the former river channel. P447-105-1614, August 18,1949.

Figure 14 Dam foundation excavation. Fault Zone 6, looking upstream. The concrete structure at top of photo forms a pump sump. P447-105-1664, September 2, 1949.

modified the requirements in his letter report of September 7, 1949 as follows: "If, as may be expected, the physical condition of the fault zone material improves with depth, it may be considered safe to reduce the specified percolation distance of 2-1/2 times the effective hydraulic head to perhaps 2 times this effective hydraulic head, or even less."

Hence the Bureau had considerable latitude in providing the percolation distance for each fault.

6. Lift-seam treatment. The procedures developed for the grouting of the lift seam on the left abutment were described in detail by Aubrey Walker[4] in a memorandum dated May 19, 1951 to Construction Engineer. The procedures are also described in a condensed version in appendix B.

The clay seam referred to as a lift seam had a thickness of 1-1/2 to 3 inches and was confined beneath a sloping rock block from 5 to 17 feet thick. In some areas the clay seam split into two seams about 7 feet apart. It was found over an extensive area of dam blocks 9, 10, 11 and 12 between faults Nos. 3 and 4. Procedures were developed on the project to progressively improve the removal of clay from about 50% in block to 12 to about 90% in block 8. The procedures included drilling 1-3/8-inch holes on not more than 5-foot centers and washing with high pressure jets and air over an extended period of time, i.e. for a group of 60 to 75 holes the time required was about 20 to 25 shifts. After the clay was removed the void was filled with thick grout starting from the downhill side of the treated area. The enormous amount of work and time required to remove the clay indicates the difficulty in removing clay from joints and bedding planes during grouting operations. The treatment was evidently satisfactory at this site as no settlement or seepage problems have developed in the treated area. The investigations, drilling, washing and grouting are illustrated in the photographs on figures 15 through 20.

7. Foundation grouting. The drilling and grouting equipment and the hook-ups to the grout holes are shown in the photographs on figures 21 through 26.

(a) Design. Grouting of the foundation consisted of the following:

(1) Six lines of consolidation "B" holes on 20-foot centers were located parallel to and from 10 to 100 feet downstream from the dam axis. An additional row of "B" holes on 10-foot centers were drilled in the foundation at the junction with the upstream excavated slope, roughly corresponding to the line of the axis. Other "B" holes were drilled in areas of major foundation defects such as faults, open joints and shallow, clay-filled bedding planes. The "B" holes ranged from 5 to 40 feet in depth, and were grouted using the ascending-stage method with packers, i.e., a packer was set at the top of each stage to be grouted.

(2) A single line of "C" holes on 10-foot centers was drilled and grouted through pipes placed in the concrete fillet at the upstream heel of the dam. The holes ranged in depth from 50 to 75 feet and were grouted at intermediate pressures to form an upstream supplemental curtain. These holes were grouted after a minimum of 25 feet or more of concrete had been placed. Located one foot downstream of the axis, the holes were inclined

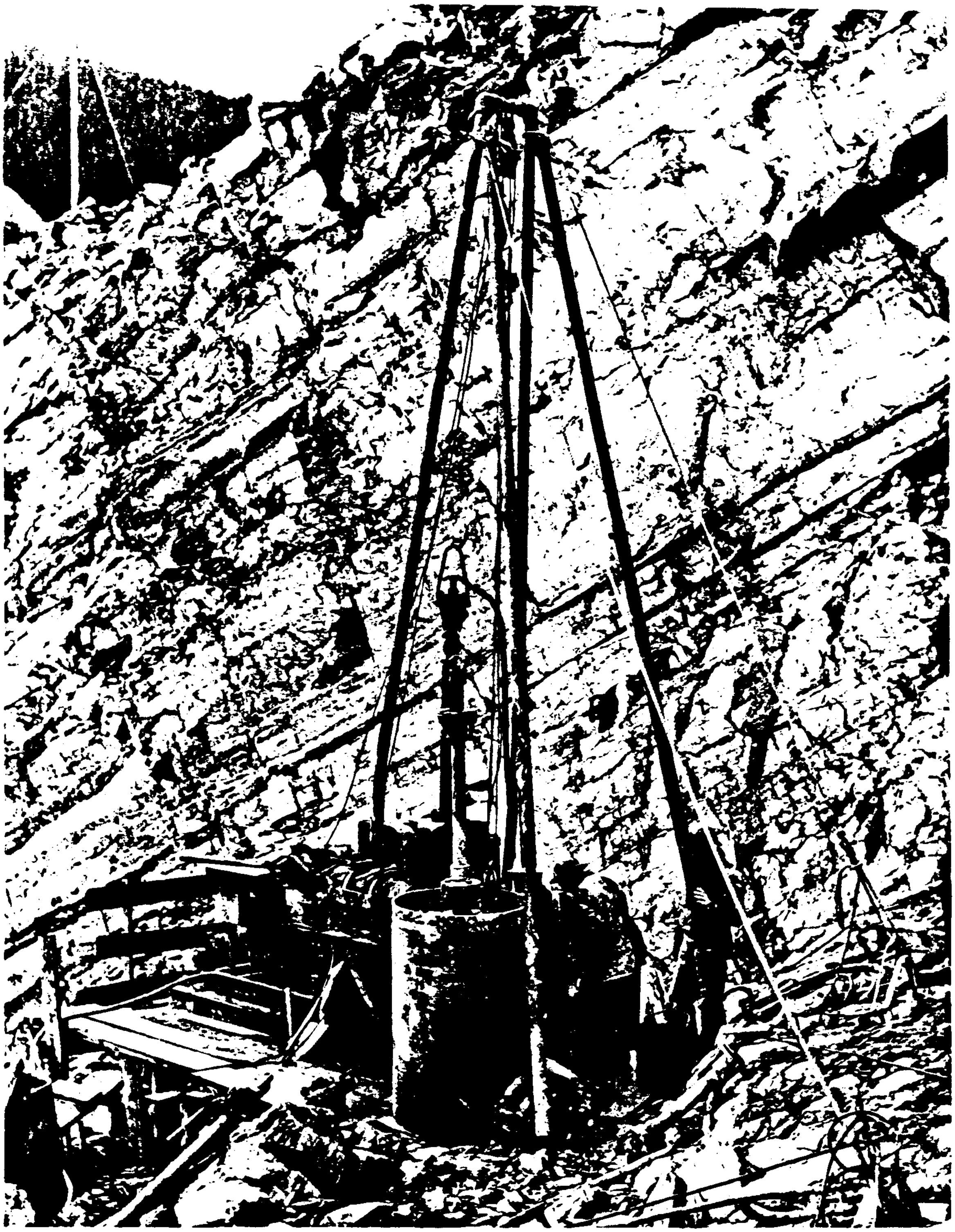


Figure 15 Calyx drill boring 36-inch diameter holes for purpose of inspection of clay seams in left abutment dam foundation rock. P447-105-1943, November 28, 1949.

Figure 16 Clay seam treatment. Nipples mark holes in the rock which were drilled for washing and grouting of clay seams. Fault Zone 4 may be seen just below where men are standing. P447-105-3083, October 24, 1950.

Figure 17 Clay seam treatment. Workmen flushing clay from seams in rock. P447-105-2956, September 16, 1950.

Figure 18 Clay seam treatment. Workmen washing clay seams in right abutment foundation rock. P447-105-2957, September 16, 1950.

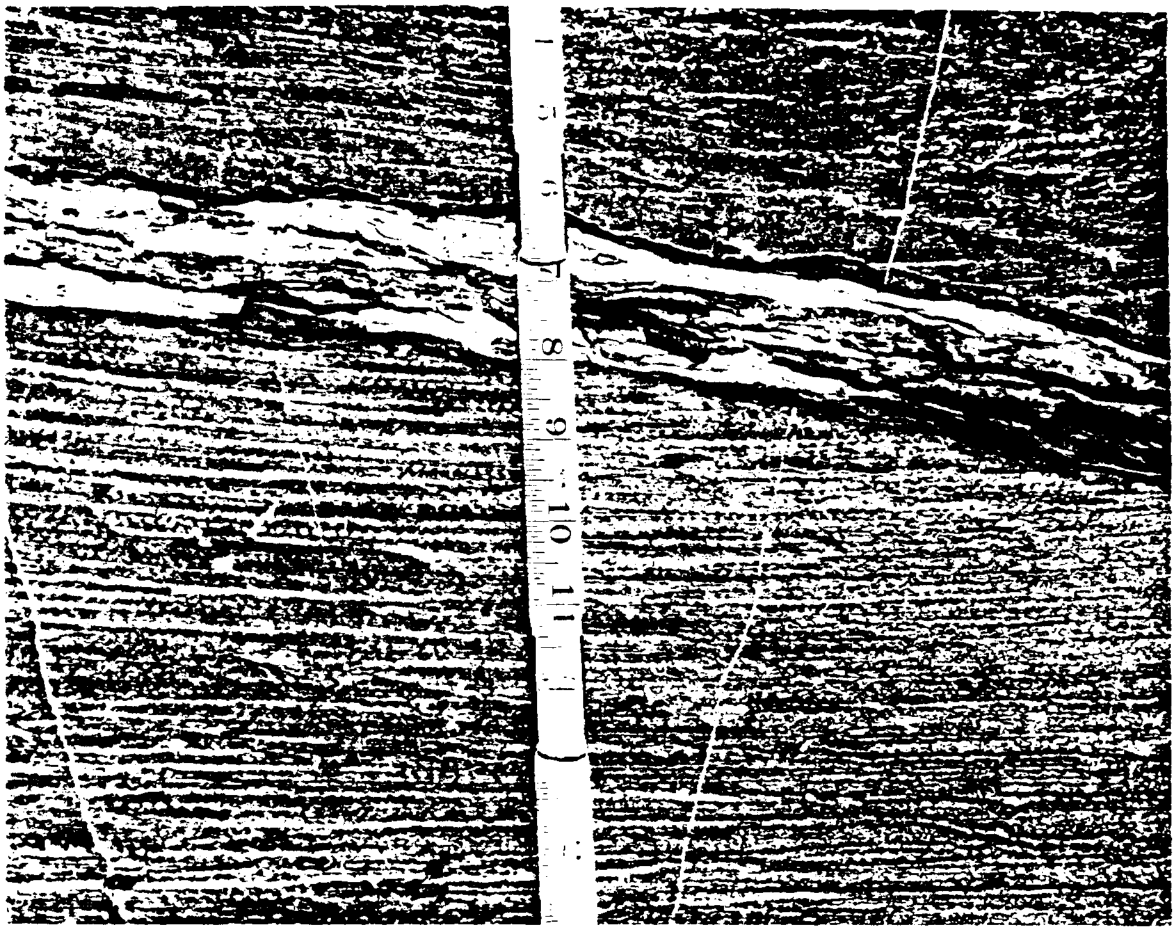


Figure 19 Clay seam treatment. Grouted seam exposed by 36-inch diameter calyx hole. Clay, interbedded with grout, was washed away by drill water. (See photo 2495). P447-105-2494, June 8, 1950.

Figure 20 Excavation of rock above clay seam along the axis of dam in Blocks 6 and 7. P447-105-3705, July 19, 1951.

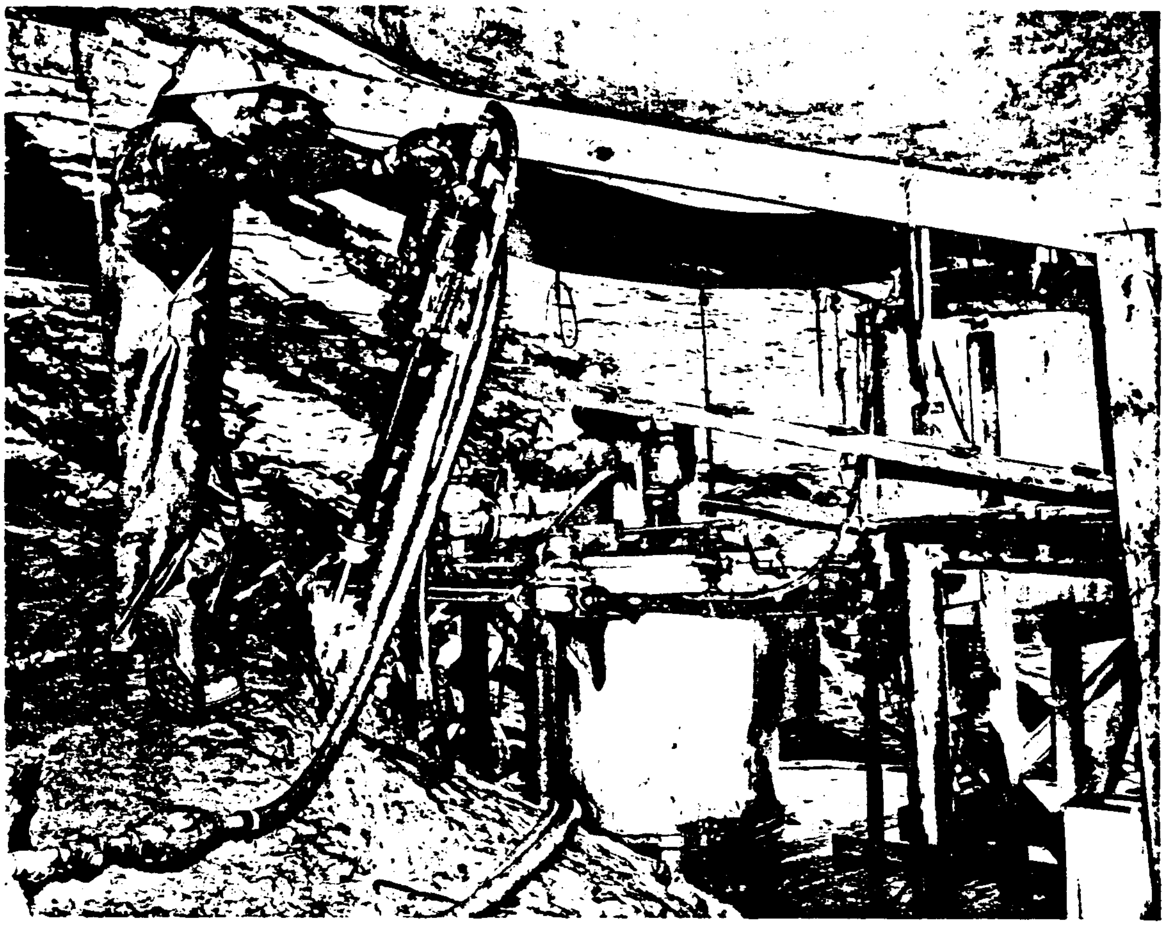


Figure 21 Diamond drilling of left abutment dam foundation rock. Grout pump, agitator, and mixer on right. P447-105-1915, November 15, 1949.

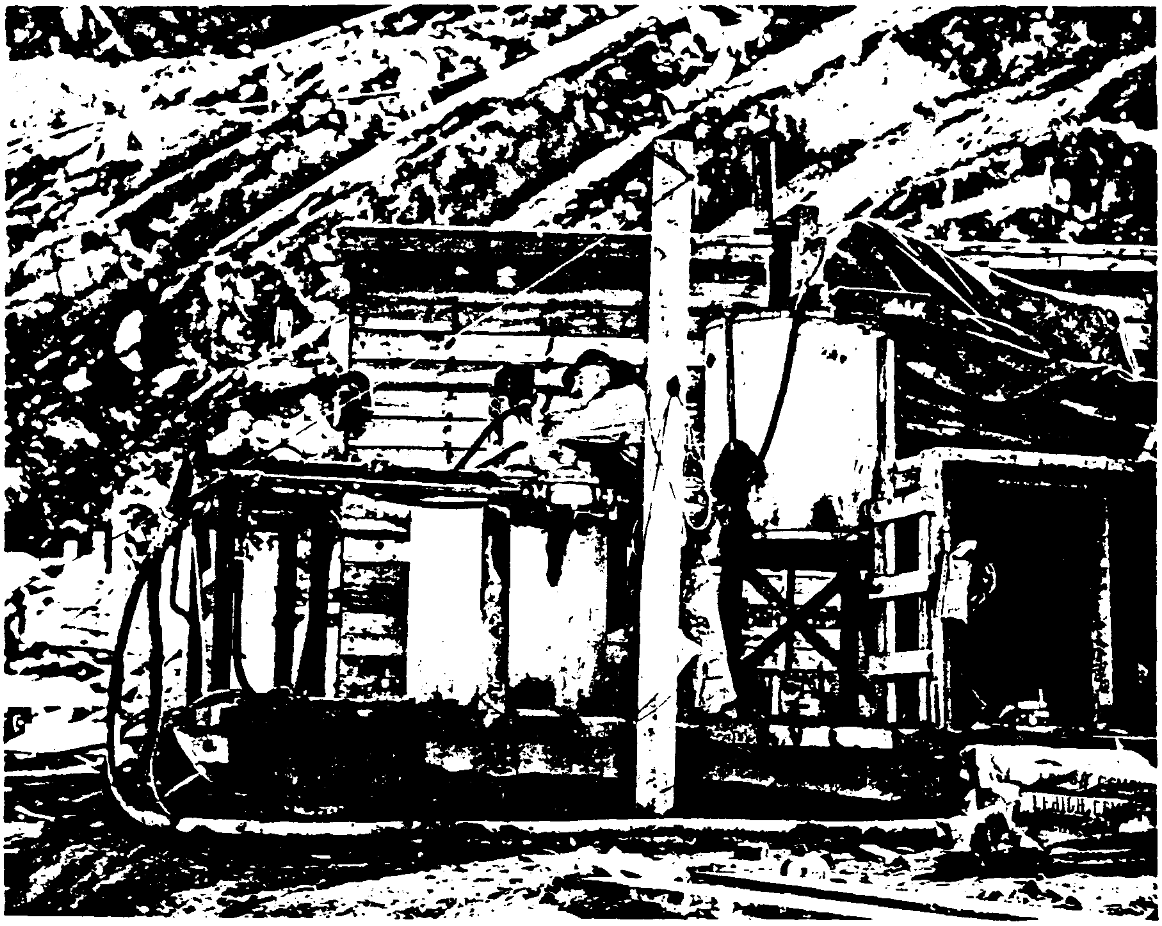


Figure 22 Pressure grouting equipment. Left to right: Duplex piston displacement pump, grout agitator, water meter, operator, and grout mixer on bench. P447-105-1761, September 28, 1949.

Figure 23 Typical hookup for packer grouting in foundation rock. P447-105-1688, September 7, 1949.

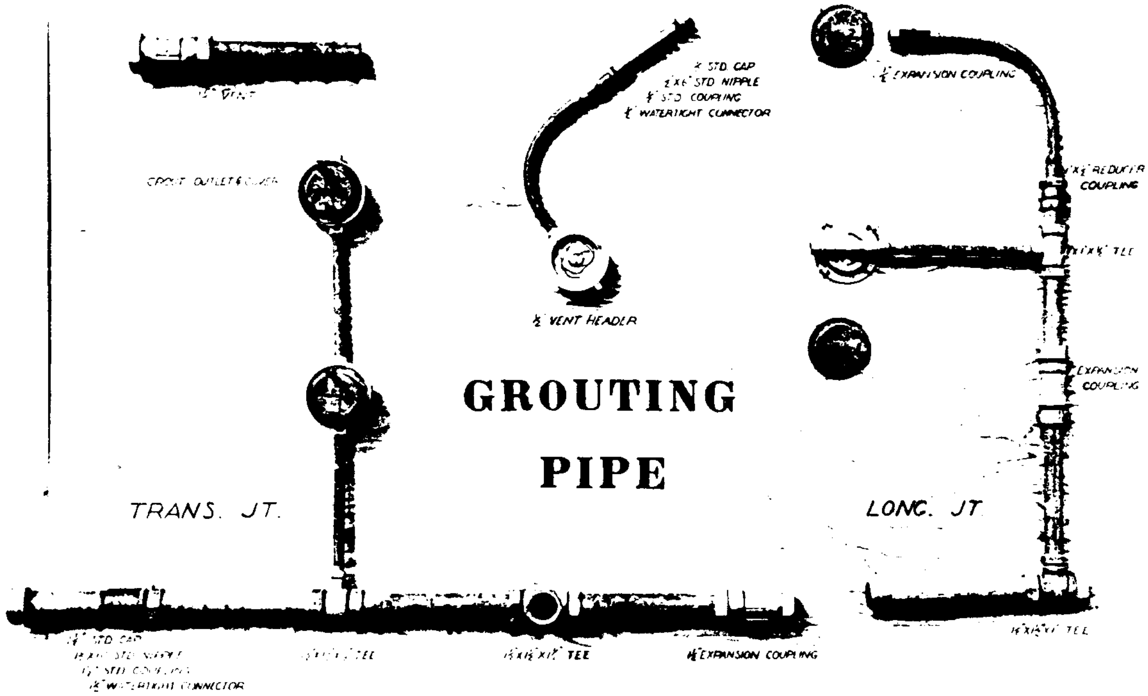


Figure 24 Grouting pipe display board showing typical hookups. P447-105-1612, August 18, 1949.

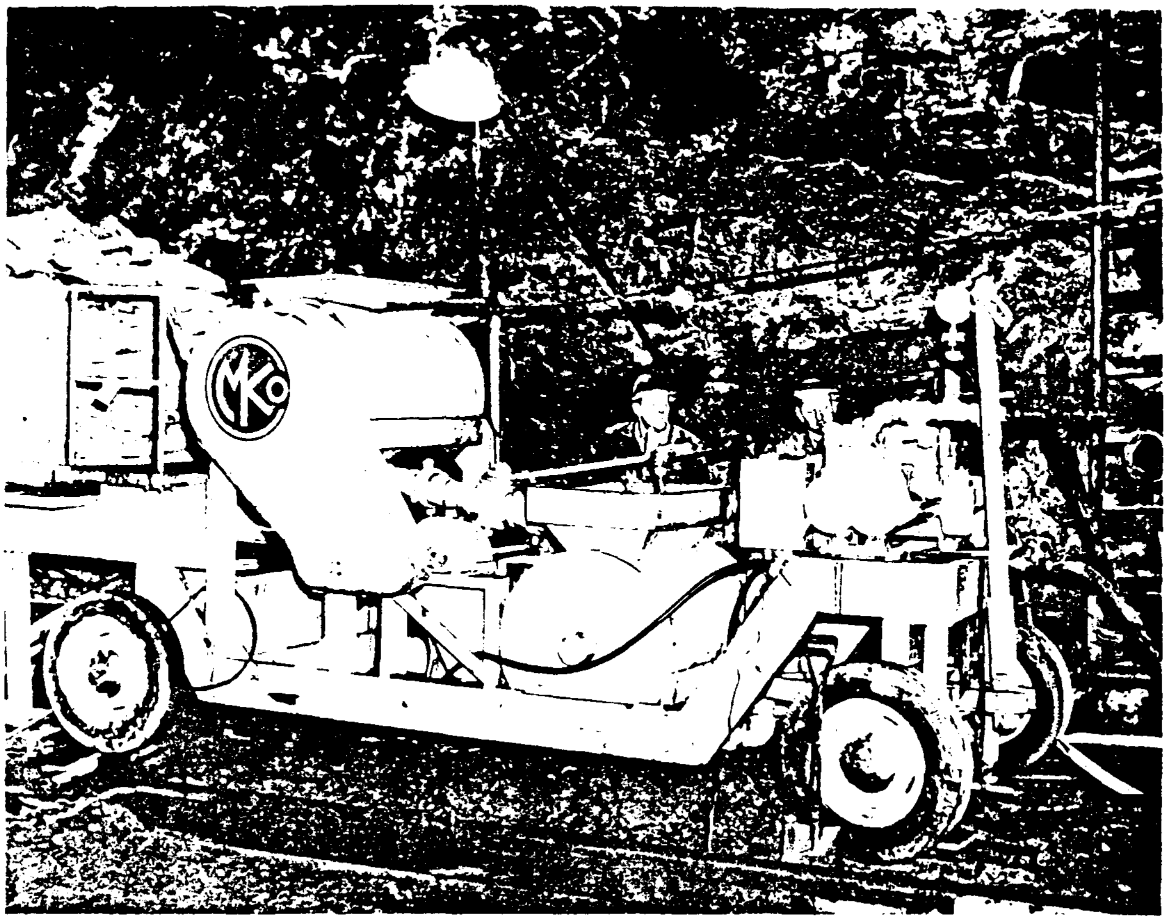


Figure 25 Trailer-mounted grouting equipment. Mixer on left, agitator in center, and duplex piston displacement pump on right. P447-105-1001, February 23, 1949.

Figure 26 Manifold hookup for pressure grouting of invert section of inlet structure to the diversion tunnel. P447-105-1002, February 23, 1949.

12 degrees from the vertical in a downstream direction. The holes were grouted by the ascending-stage method using packers.

(3) The primary cutoff curtain consisted of a single line of "A" holes drilled through pipes in the drainage gallery. The holes were inclined upstream at 12 degrees so that the base of the grout curtain was located on the vertical projection of the heel of the dam. The depth of "A" holes varied from 110 to 310 feet and the average depth of the holes was 157 feet. Grouting was by the split-spacing method from an initial spacing of 160 feet (primary holes) to a final spacing of 10 feet. The plan was to drill the 5-foot-spaced holes only if deemed necessary, on the basis of grout take in the holes on 10-foot spacing. The ascending-stage grouting method was used with packers. The profile of the "A" holes is shown on figures 27, 28 and 29.

(b) Specifications. The specifications were complete in regard to the general performance of the work. However, the specifications did not include requirements for: (1) the expected range of water-cement ratios; (2) water-pressure tests for each zone and the test time; (3) minimum size and capacity of equipment and pumps; (4) minimum size of the grout pipe within the packer; (5) final backfilling of completed holes; (6) storage of a sufficient quantity of cement at or near the grout plant to insure that grouting operations would not be held up by lack of cement; and (7) a placing unit (agitator sump and pump) at an elevation approximately that of the hole collar where the elevation difference between primary grout plant and the hole collar may result in gravity pressure higher than the maximum allowable pressure.

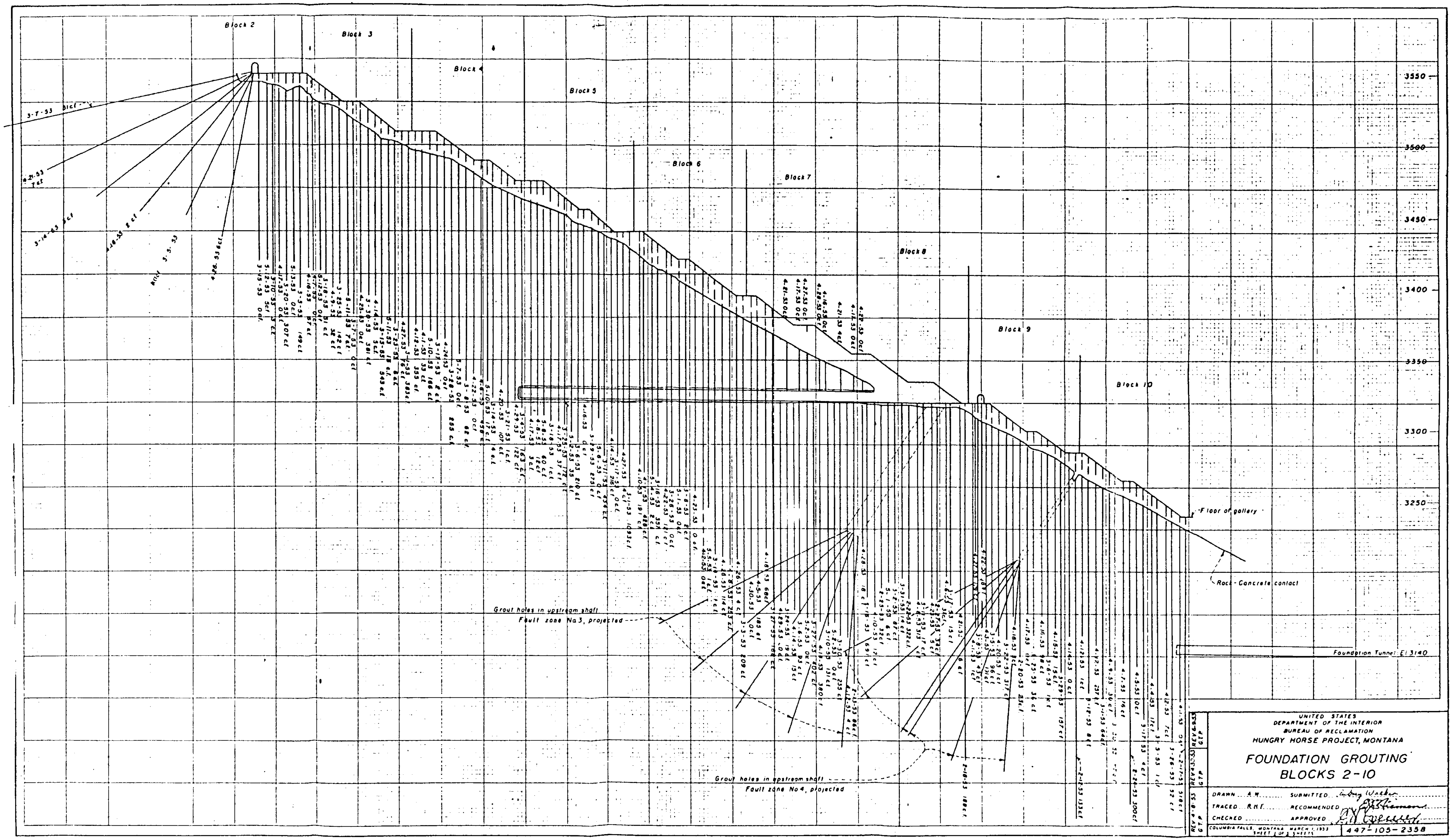
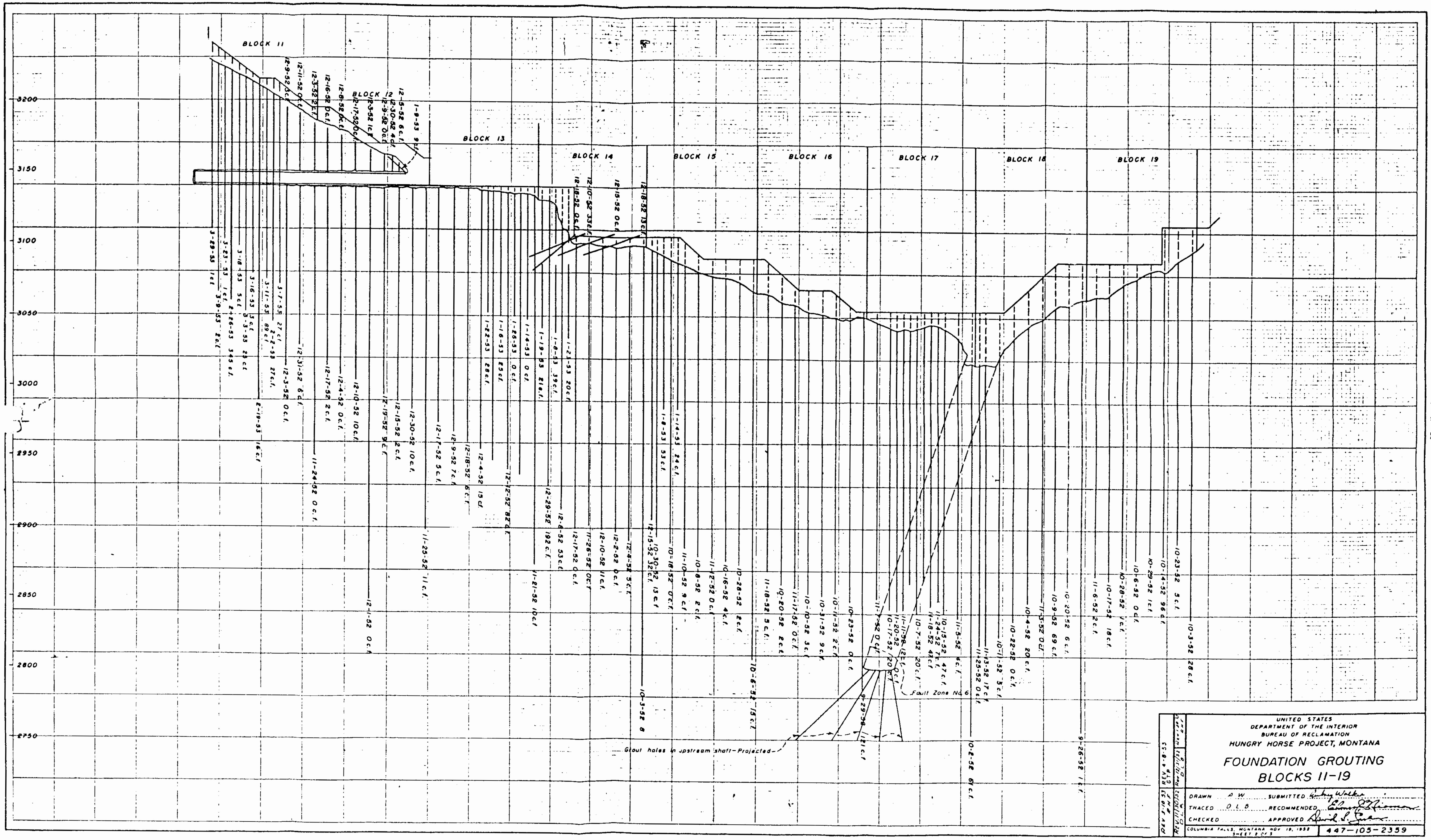

Block 2
Block 3
Block 4
Block 5
Block 6
Block 7
Block 8
Block 9
Block 10
3550
3500
3450
3400
3350
3300
3250
Floor of gallery
Rock - Concrete contact
Grout holes in upstream shaft
Fault zone No. 3, projected
Grout holes in upstream shaft
Fault zone No. 4, projected
Foundation Tunnel El. 3140
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
HUNGRY HORSE PROJECT, MONTANA
FOUNDATION GROUTING
BLOCKS 2-10
DRAWN
SUBMITTED
TRACED
RECOMMENDED
CHECKED
APPROVED
COLUMBIA FALLS, MONTANA
447-105-2358

FIGURE 27


BLOCK 11
BLOCK 12
BLOCK 13
BLOCK 14
BLOCK 15
BLOCK 16
BLOCK 17
BLOCK 18
BLOCK 19
3200
3150
3100
3050
3000
2950
2900
2850
2800
2750
Fault Zone No. 6
Grout holes in upstream shaft-Projected
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
HUNGRY HORSE PROJECT, MONTANA
FOUNDATION GROUTING
BLOCKS 11-19
DRAWN
TRACED
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
447-105-2359

45

FIGURE 28

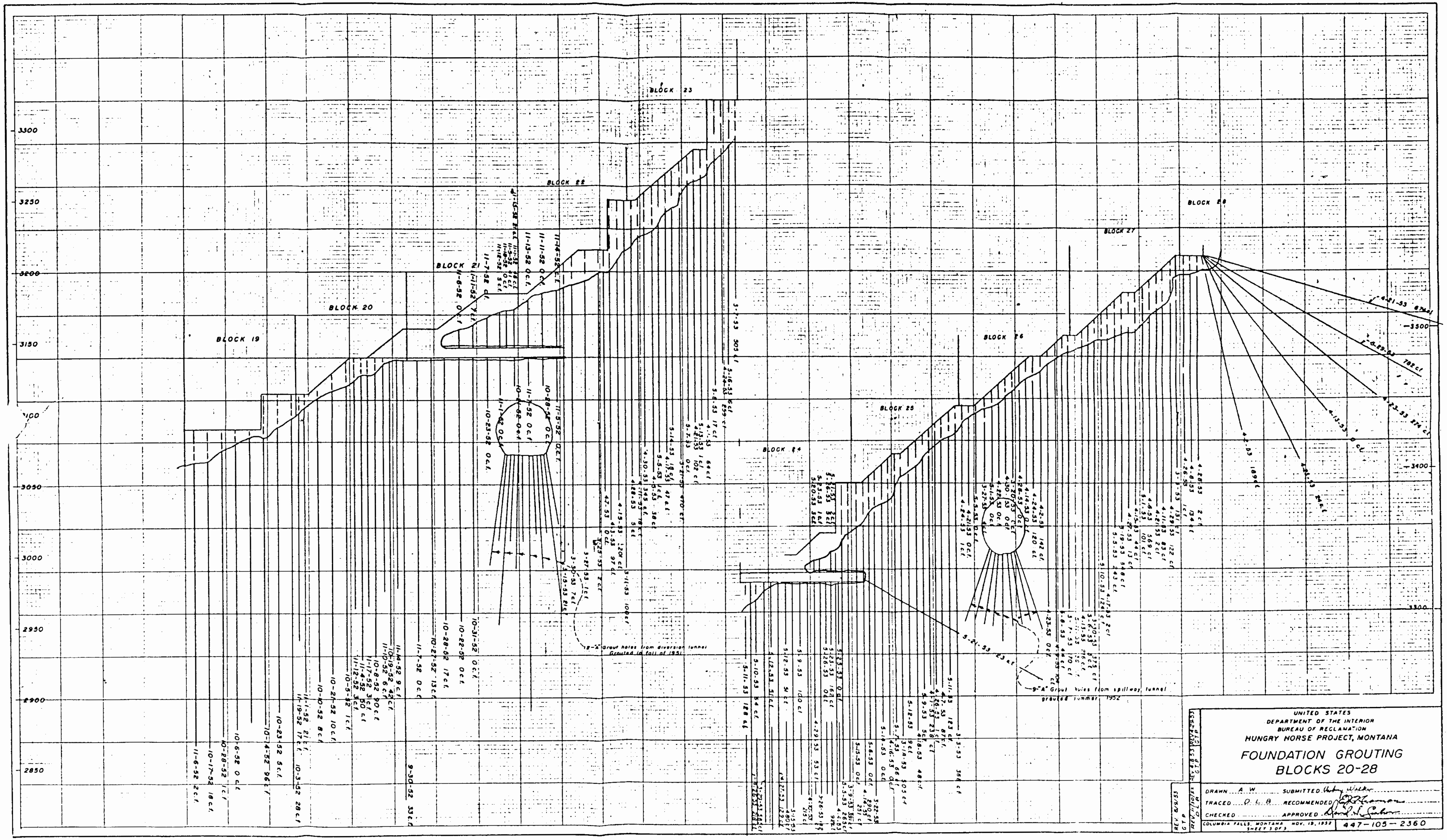

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
HUNGRY HORSE PROJECT, MONTANA
FOUNDATION GROUTING
BLOCKS 20-28
COLUMBIA FALLS, MONTANA
447-105-2360
BLOCK 19
BLOCK 20
BLOCK 21
BLOCK 22
BLOCK 23
BLOCK 24
BLOCK 25
BLOCK 26
BLOCK 27
BLOCK 28

FIGURE 29

(c) Water-pressure tests. The specifications required testing up to the required grouting pressure. The pressures used and the results were not included in the summaries of the daily drilling and grouting records.

(d) Grout mixes. The grout mixes varied, but as described in the Grouting Report on the Hungry Horse Dam - 1954 were as follows:

(1) In grouting the lift seam, only thick mixes were used, ranging from 2:1 to 0.75 water-cement ratio, by volume with the greater amount injected being 0.8:1 grout which readily travelled from hole to hole through the open seam.

(2) In the grouting of the other shallow holes, the grout mixtures ranged from 5:1 to 1:1 water-cement ratio.

(3) By experimenting with grout mixtures in the deeper holes, it was found that grout having a water-cement ratio of 8:1 was the thinnest that was practical to pump. The tight, fine seamed "A" holes were grouted with 8:1 grout and the more open holes with thicker grout.

(e) Grout-injection pressures. The pressures used for the various types of grouting were as follows:

Type Grouting	Average Depth of Holes	Injection Pressures
"B" Holes (Low Pressure)		
General Consolidation	30'	5 - 150 psi
Right Abutment		75 - 150 psi
Left Abutment		5 - 150 psi
Fault Zones	25' - 150'	not more than 50 psi
Lift Seams	7' - 20'	5 - 40 psi
"C" Holes (Intermediate Pressure)	64' - 74'	150 psi without packer 200 psi with packer at 25' 250 psi with packer at 50'
Note: All "C" grouted after concrete placed to height of 25'		
"A" Holes (High Pressure)		

Block 2	150 psi + 1-1/4 psi per ft. of depth to packer
Block 3	175 psi + 1-1/4 psi per ft. of depth to packer
Blocks 4 & 28	200 psi + 1-1/4 psi per ft. of depth to packer
Blocks 5 & 27	225 psi + 1-1/4 psi per ft. of depth to packer
Blocks 6 & 26	250 psi + 1-1/4 psi per ft. of depth to packer
Blocks 7 & 25	300 psi + 1-1/4 psi per ft. of depth to packer
Block 8	350 psi + 2-1/2 psi per ft. of depth to packer
Blocks 9 & 24	400 psi + 2-1/2 psi per ft. of depth to packer
Blocks 10 & 11	450 psi + 2-1/2 psi per ft. of depth to packer
Blocks 22 & 23	450 psi + 2-1/2 psi per ft. of depth to packer
Blocks 12 thru 21	- 500 psi for any part of hole

<u>Note</u>: (1) All concrete had been placed before starting "A" holes.
(2) Pressures noted above are gage pressures at collar.
(3) All data on pressures are from Grouting Report on The Hungry Horse Dam - 1954.[5]

(f) <u>Spacing of holes and closure</u>. Grouting of the "A" holes was by the split-spacing closure method. According to the specifications, "It is contemplated that the "A" holes will be drilled at approximately 5 feet, or greater spacing." This was the plan followed and data on the unit takes are presented in the following tabulation:

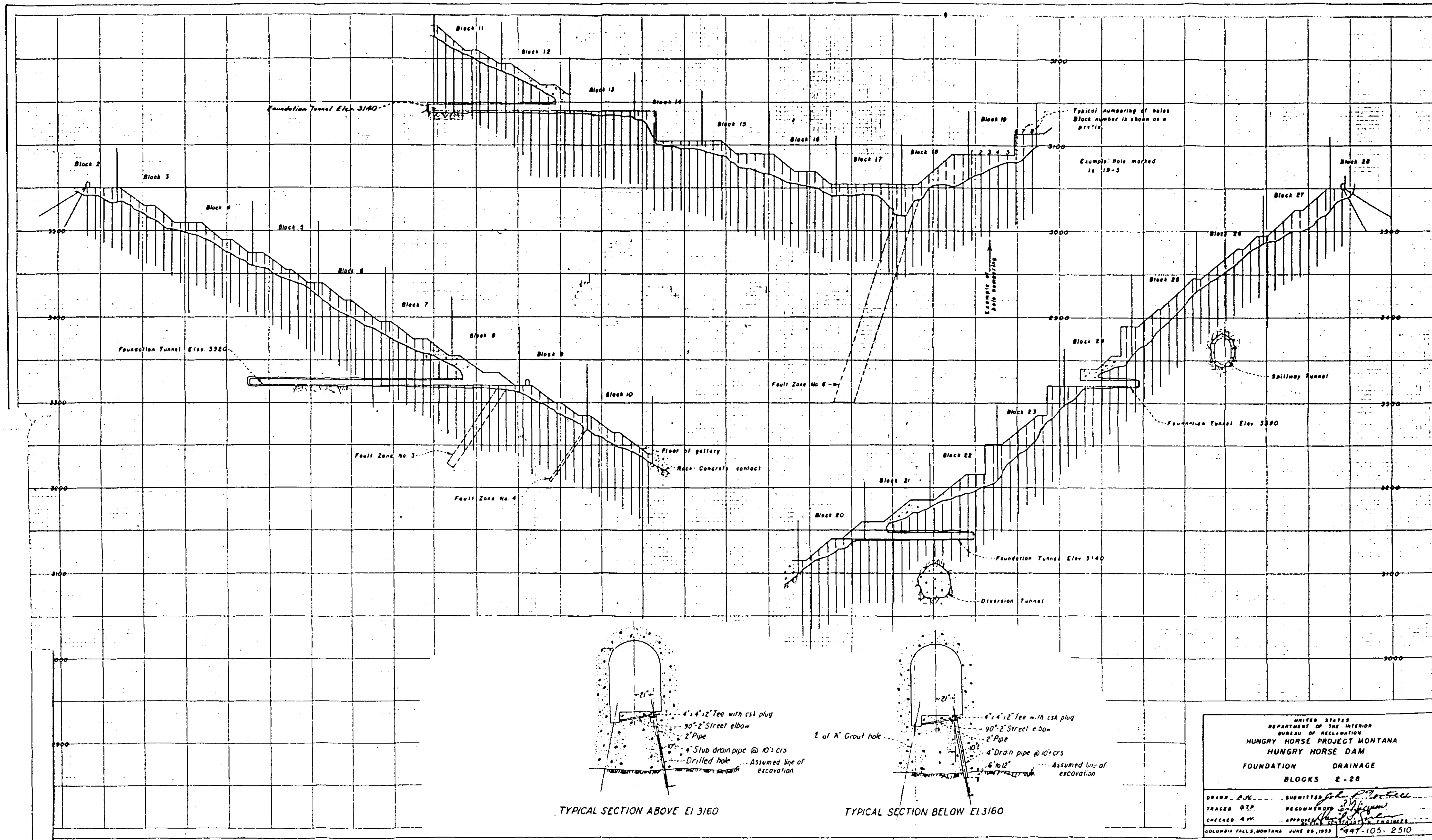
Spacing	No. Holes	Sacks Per Lin. Foot	Sacks Per Hole
80' Centers	26	0.91	192
40' Centers	27	0.79	139
20' Centers	53	0.63	103
10' Centers	106	0.47	76
5' Centers	149	0.33	45

(g) Refusal criteria. The criteria for refusal given 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 the grout mixture in 20 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, and in 5 minutes if pressures in excess of 200 pounds per square inch are being used."

(h) Final backfill of holes. The specifications do not have a requirement for sounding completed holes or for filling any part of the hole not filled with solid grout. The records do not indicate what methods were used, if any.

8. Foundation drainage. The profile of foundation drainage holes is shown on figure 30. Based on the limited data available from the explorations, the drainage holes penetrated through the zone of high-water takes and into the zone of low-water takes.



Block 2
Block 3
Block 4
Block 5
Block 6
Block 7
Block 8
Block 9
Block 10
Block 11
Block 12
Block 13
Block 14
Block 15
Block 16
Block 17
Block 18
Block 19
Block 20
Block 21
Block 22
Block 23
Block 24
Block 25
Block 26
Block 27
Block 28
Foundation Tunnel Elev. 3140
Foundation Tunnel Elev. 3320
Foundation Tunnel Elev. 3380
Foundation Tunnel Elev. 3140
Typical numbering of holes. Block number is shown on profile.
Example: Hole marked is 19-3
Example of hole numbering
Fault Zone No. 3
Fault Zone No. 4
Fault Zone No. 6
Floor of gallery
Rock-Concrete contact
Spillway Tunnel
Diversion Tunnel
3200
3100
3000
3400
3300
2900
3500
2800
2700
4"x4"x2" Tee with csk plug
90°-2" Street elbow
2" Pipe
4" Slub drain pipe @ 10'± crs
Drilled hole
Assumed line of excavation
℄ of "A" Grout hole
4"x4"x2" Tee with csk plug
90°-2" Street elbow
2" Pipe
4" Drain pipe @ 10'± crs
6" to 12"
Assumed line of excavation
TYPICAL SECTION ABOVE El 3160
TYPICAL SECTION BELOW El 3160
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
HUNGRY HORSE PROJECT MONTANA
HUNGRY HORSE DAM
FOUNDATION DRAINAGE
BLOCKS 2-28
DRAWN A.K.
TRACED G.T.P.
CHECKED A.W.
SUBMITTED
RECOMMENDED
APPROVED
COLUMBIA FALLS, MONTANA JUNE 25, 1953
447-105-2510

FIGURE 30

IV. ANALYSIS

9. Preconstruction geologic investigations. Explorations at the dam site consisted of 49 borings and one tunnel on the left abutment and three tunnels on the right abutment. Core recovery in the borings averaged about 85 percent. In some borings, the recovery exceeded 95 percent; however, borings located in fault zones, such as DH-43, had recovery of less than 20 percent. The core and the tunnels were logged in sufficient detail and included adequate information on rock quality. The geologist logging the core, Kenneth S. Soward, used a classification system in 1946-47 which is very close to the RQD, (Rock Quality Determination) system so widely recommended and publicized 20 years later by the University of Illinois, Deere, et al. Soward classified the core by soundness and length of pieces as excellent, very good, good, fair, poor to broken and sheared. The water drilling loss information and data from the percolation tests were also well recorded on the logs.

The drilling was laid out and performed primarily for an axis (G-G) of a straight-gravity dam 420 feet high. As the exploratory program progressed, it was found that the rock was capable of supporting a higher arch-gravity dam. The design was changed to an arch-gravity dam with the axis of the arch dam crossing the river channel 350 feet upstream from the G-G axis. As a result, most of the explorations were accomplished along the downstream toe of the dam used in the final design (see figure 2). Hence, the explorations along the arch-dam axis and beneath the arch dam were extremely sparse, particularly for such a large structure.

The preconstruction geology reports do not mention the need for a grout curtain or make any recommendations concerning a grout curtain. The borings along the final dam axis were too few and too shallow to be of much value in the design of a grout curtain. Nevertheless, the geology report should have indicated how the basic geologic conditions as developed from the borings for the G-G axis would have influenced the design of the grout curtain.

After the type of dam and location were changed, it is considered that additional borings should have been provided to give better coverage for the designed structure. In many cases when slight changes are made in dam siting, there is insufficient time to re-explore the site. Nevertheless, the geologic conditions under the shifted structure may be considerably different than at the original site. The need for thorough explorations at the final site is even greater now than in the late 1940's as the faults and clay seam discovered during construction would now be the basis for very large claims by the contractor not only for the extra work but also for the ripple effect throughout the remainder of the construction.

The available borings within or close to the lift seam area (DH-15, DH-16 and DH-44) were reviewed to determine if the lift seam could be detected. In DH-44 all water was lost in a 5-inch crevice at an angled depth of 68.3 feet. This crevice was at the same approximate location as the lift seam in block 11. DH-16 was located close to fault No. 4 and had high-core losses to about the level of the lift seam. DH-15 had no recorded defects that would indicate the presence of a clay seam at the appropriate elevation.

A continuous clay seam cannot be identified from this limited data; however, other preconstruction information indicated the possibility of continuous clay seams. Harshman[7 and 8] found clay seams in the Siyeh limestone bedrock in the downstream road cuts and in tunnel A in the right abutment; however, he believed the clay was due to weathering and would not be present in appreciable amounts at depths below 15 to 20 feet. Clay seams were found in tunnel B on the left abutment by Soward[9] in 1946.

Only fault zone No. 6 was found before construction. Five additional faults were found during construction. The full scope of treatment for fault zone No. 6 was not developed until the dam was under construction. From this and other standpoints it must be concluded that the preconstruction geologic investigations were inadequate.

10. Design of grout curtain. The depth of the deep curtain is considered to be adequate. The logs and pressure test results from 49 exploratory borings at the site disclosed that the limestone foundation was sound and that surface weathering and solutioning effects were generally confined to the upper 40 to 50 feet of rock, which was removed during foundation excavation. Although minor water losses occurred to depths of 200-210 feet below the surface (DH-24 and DH-39), there was no indication that these losses represented a widespread horizon or persistent zone of solutioning at depth. Therefore, assuming that solutioning was primarily structurally controlled, the drilling and grouting of the holes by the closure method to a final spacing of 5 feet (and closer if required) should have insured intersection of, and effective sealing of most foundation defects.

The use of vertical grout holes with a 12-degree inclination upstream is considered satisfactory as it provided a chance to intersect the sloping bedding planes and the perpendicular jointing between the bedding planes.

11. Grouting methods and procedures. (a) Specifications. It is considered that more detailed specifications would have provided the contractor a better basis for bidding.

(b) Water-pressure tests. It is considered that the results of the water-pressure tests could have been used as a guide in selecting the initial grout mixes.

(c) Grout mixes. Most of the grout used for backfilling the lift seam had a water-cement ratio of about 0.8:1. The grouting report states this mixture "readily travelled from hole to hole through the open seam." As the thick grout readily travelled through the 1- to 3-inch-thick seam, the question is raised as to why extremely thin mixes are needed elsewhere. The thick grout used in the lift seam had a minimum of shrinkage and bleed water was insignificant as holes drilled behind the grouting did not seep.

The grouting report states, "By experimenting with grout mixtures in the deeper holes, it was found that grout having a water-cement ratio of 8:1 was the thinnest that was practicable to pump." It is considered that thinner mixes were not needed, but why thinner mixes were not practicable to pump is not understood.

(d) Grout-injection pressures. Data presented in the grouting report indicates that the pressures used in grouting the deep curtain "A" holes were excessive. The following quote is from the grouting report:

> "Uplift of the dam and foundation occurred with the grouting of a hole in block 5, with the packer at a depth of 140 feet. Cement pumped into the hole was 1036 sacks and the pressure was 400 p.s.i. with a water-cement ratio of 5:1 to 3:1. Pressure was lowered to 200 p.s.i. and the grout thickened to a 1:1 water-cement ratio when uplift was indicated. <u>The uplift from this hole was fairly general over the left abutment</u>. The uplift was zero in Block 14, gradually increasing to approximately 0.49 inches in Block 3." (underlining added)

The report indicates that measures adopted to prevent further uplift during later grouting were a reduction of grouting pressures, on a sliding scale and a thickening of the grout mixture being pumped. The report describes the sliding scale method and goes on to state the following.

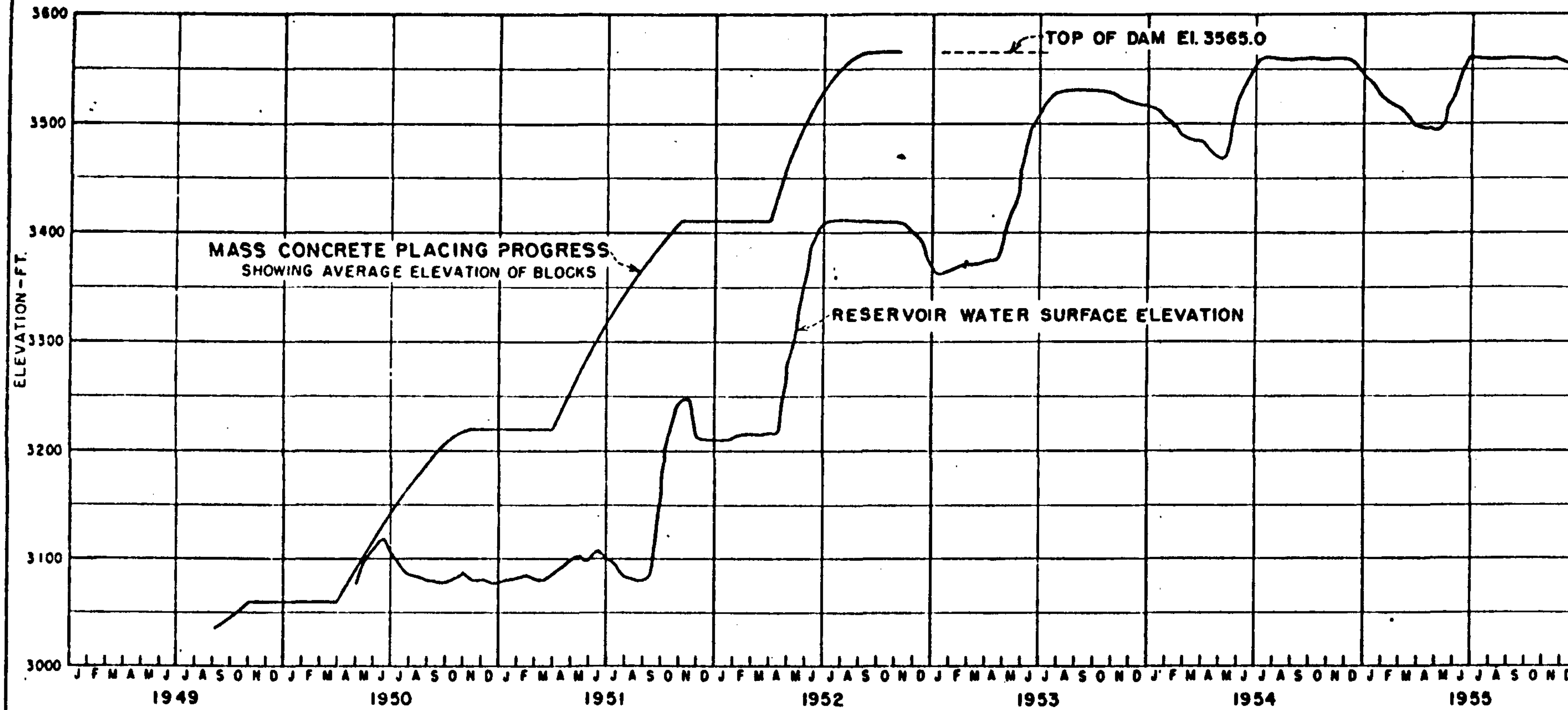
> "This method appeared to be very satisfactory, as <u>the accumulated uplift in the next two and one half months was an additional 0.31 inch</u> and which, it is thought, occurred during the grouting of several holes in which carelessness or inexperience of inspectors were responsible, in that the precautions were not closely followed." (underlining added)

Movements continue to occur to near the end of the grouting program as indicated by the following quote from the Foundation Drilling and Grouting Report for April 1953: "Uplift of dam and foundation of the left abutment was slight with the total to date being 1/8 inch in block 14 to a maximum of 13/16 inch in block 3."

It should be noted that the reductions in grouting pressures and thickening of the grout mixtures were to be used only after movement was detected, i.e., after the damage had been done.

The hole being grouted on February 25, 1953 was hole No. 169. The gallery elevation was 3,440 feet. Reservoir level was 3,370 feet as shown on figure 31. The grouting record for hole No. 169 is shown on figure 32. Computations showing the computed effective uplift at the packer and the computed effective vertical load at the packer level are presented on figure 33. The groundwater table was assumed to be at the reservoir level. These computations indicate that the safety factor against uplift was 0.43. For a safety factor of 1.0, the collar grout pressure would have had to be reduced to 142.1 psi. The bedding planes are parallel to the slope on the left abutment and applying excessive uplift pressures to an open seam under these conditions is apt to cause lifting of the rock and the dam. As shown on figure 34, lifting of the dam also occurred on the other abutment where the bedding was dipping into the abutment.

The lifting of such a wide portion of the foundation and dam on the left abutment probably disturbed and damaged the grouting that had been completed in advance of the lifting. The left abutment foundation rock was not monolithic; it was broken into at least seven sections by faults, the trends of which were approximately normal to the axis. Prior to "A" hole grouting, the faults had been treated by construction of positive cutoffs both upstream and downstream as well as by special grouting. It is unlikely that the treated fault zones escaped damage in the lifting.



HUNGRY HORSE DAM

CONSTRUCTION PROGRESS AND RESERVOIR OPERATION

447-D-2042

FIGURE 31

BUREAU OF RECLAMATION — Hungry Horse PROJECT — February 1953 (MONTH, YEAR)

RECORD OF DRILLING AND GROUTING OPERATIONS

SHEET 4 OF 5

DRAWING 447-105-2358, 447-105-2359 — LINE "A" (A,B,C, ETC) — FEATURE Dam (DAM, SPILLWAY, TUNNEL, ETC)

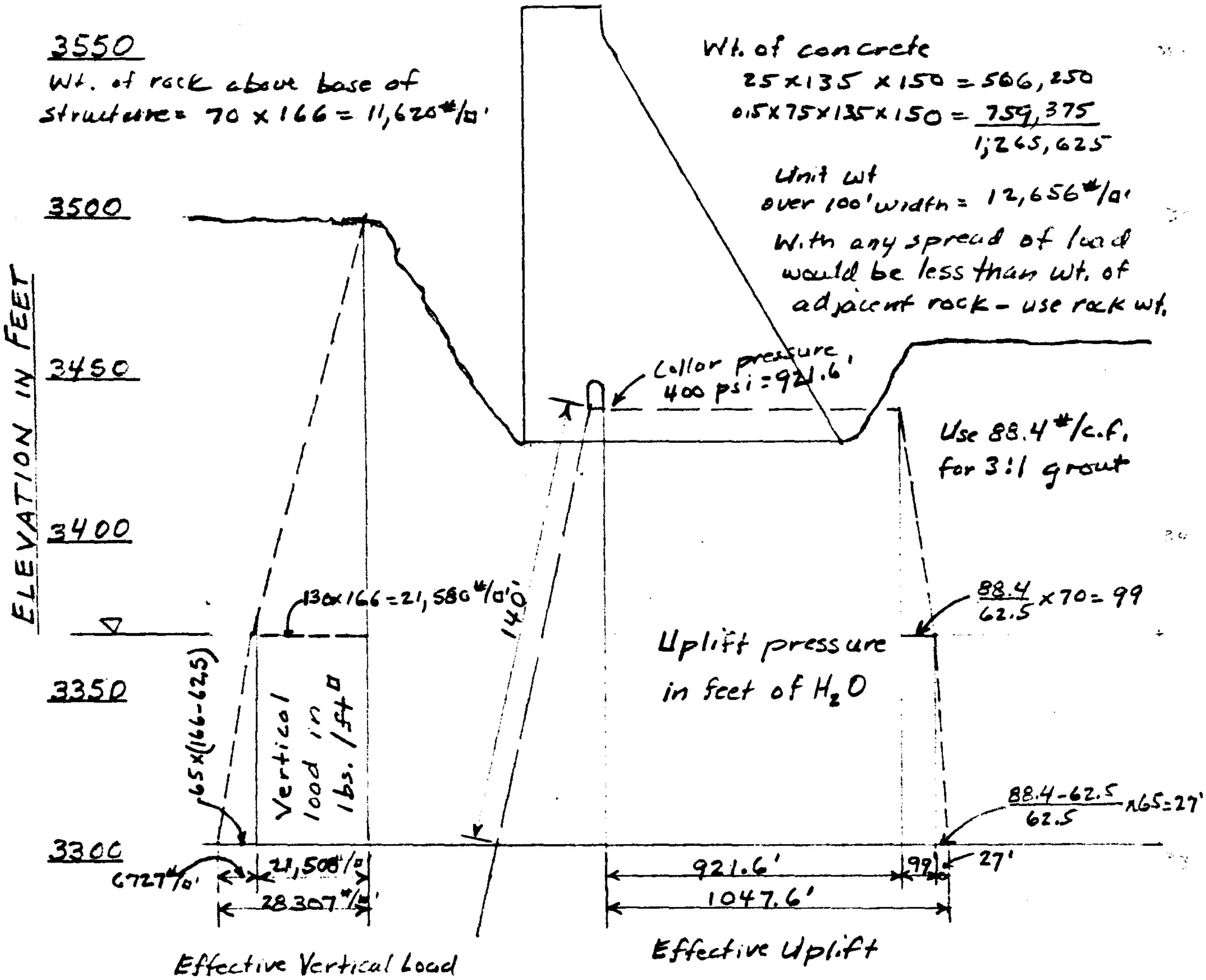
DRILLING						GROUTING							
No.	Depth of Hole			Date Drilled	Remarks	Date Grouted	Cement Placed	Pressure		W/C		Max Sks/Hr	Remarks
			Start Depth					Max	Min	Max	Min		
4	16	153		2-25-53	Block 11	2-26-53	0	500	500	–	–	–	Packer at 150'
						"	0	500	500	–	–	–	" " 130'
						"	0	500	500	–	–	–	" " 110'
						"	6	500	500	5.0	5.0	6	" " 90'
						"	0	500	500	–	–	–	" " 70'
						"	0	500	500	–	–	–	" " 50'
						"	337	500	500	5.0	2.0	24	" " 30'
						2-27-53	2	450	450	5.0	5.0	2	Nipple
12	3	136		2-21-53	Block 8	2-27-53	0	500	500	–	–	–	Packer at 120'
						"	0	500	500	–	–	–	" " 100'
						"	16	500	500	5.0	4.0		" " 50'-leaking
						"	4	450	450	4.0	4.0		" " 20' "
						"	14	350	350	3.0	3.0	10	Nipple
16	10	169		2-22-53	Block 5	2-24-53	0	400	400	–	–	–	Packer at 160'
						2-25-53	1036	400	200	5.0	1.0	55	" " 140'
						2-26-53	21	250	250	4.0	3.0		" " 120'
						"	0	200	200	–	–	–	" " 100'
						"	0	200	200	–	–	–	" " 80'
						"	0	200	200	–	–	–	" " 60'
						"	0	200	200	–	–	–	" " 40'
							0	200	200	–	–	–	" " 20'
						3-1-53	36	200	100	1.0	1.0	36	Nipple
76	10	135		2-23-53	Block 3	2-26-53	1	350	350	6.0	6.0	1	Packer at 130'
						2-27-53	165	350	325	6.0	3.0	12	" " 110'
						"	0	275	275	–	–	–	" " 90'
						"	0	250	250	4.0	4.0	–	" " 70'
						"	0	225	225	–	–	–	" " 50'
						"	0	225	225	–	–	–	" " 30'
						3-1-53	217	[illegible]	75	4.0	1.0	55	Nipple

THIS CONSOLIDATED REPORT TOGETHER WITH PLANS AND PROFILE SHOWING LOCATION OF HOLES TO BE FURNISHED THE DENVER OFFICE MONTHLY — 7-1349

FIGURE 32

HUNGRY HORSE DAM

CALCULATIONS FOR EFFECTIVE VERTICAL LOAD AND EFFECTIVE UPLIFT AT BLOCK 5



$$\text{Uplift} = \frac{1047.6 \times 62.5}{144} = 454.7 \text{ p.s.i.}$$

$$\text{Vertical Load} = \frac{28307}{144} = 196.6 \text{ p.s.i.}$$

$$\text{Safety factor against uplift} = \frac{196.6}{454.7} = 0.43$$

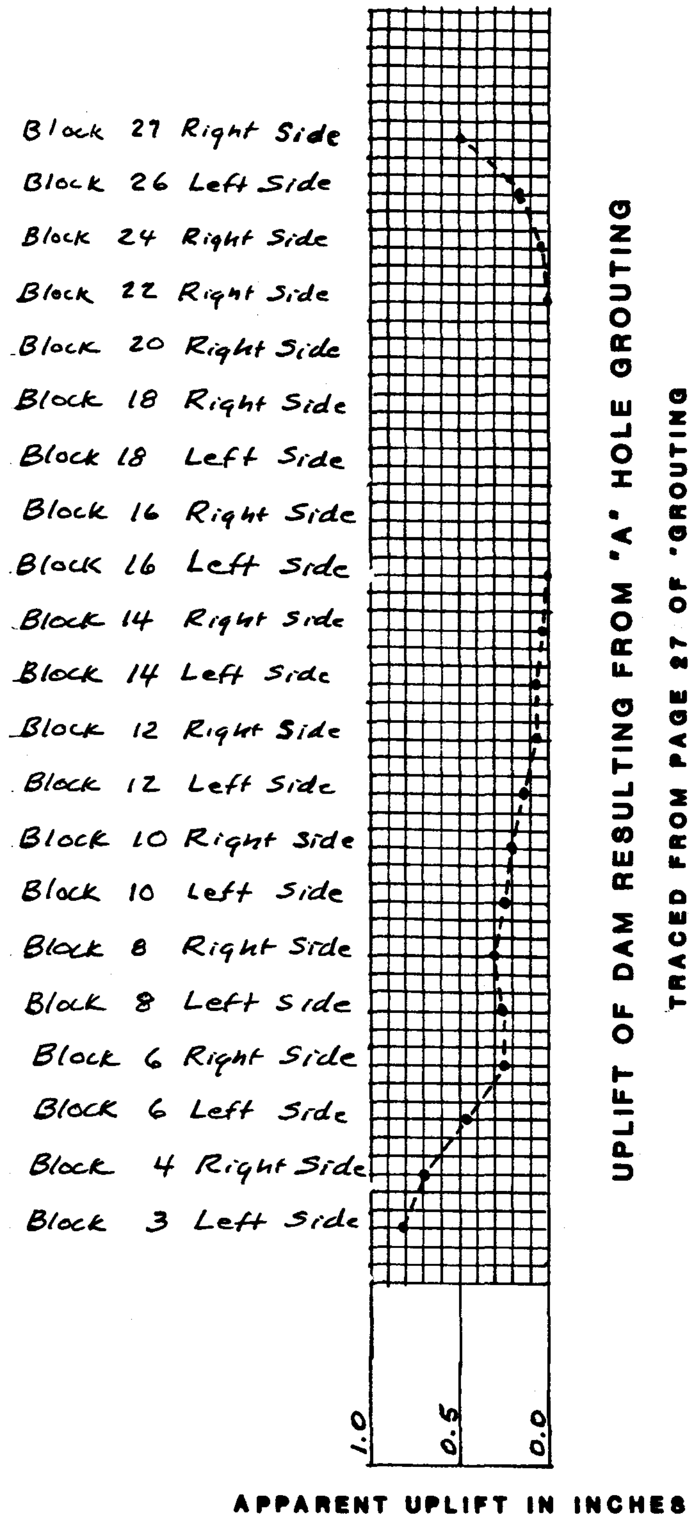
Collar pressure for safety factor of 1. The collar pressure plus pressure due to grout in hole must not exceed unit vertical load.

$$\therefore \text{ for S.F.} = 1 \text{ Collar pressure} = 196.6 - \frac{126}{2.31} = 196.6 - 54.5 = 142.1 \text{ psi}$$

PREPARED BY CLAUDE A. FETZER JULY 1982

FOR BUREAU OF RECLAMATION CONTRACT NO. 2-07-DV-00148

FIGURE 33



UPLIFT OF DAM RESULTING FROM "A" HOLE GROUTING

TRACED FROM PAGE 27 OF "GROUTING REPORT ON THE HUNGRY HORSE DAM"

CONTRACT NO. 2-07-DV-00146

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

HUNGRY HORSE DAM

UPLIFT OF DAM FROM GROUTING

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

FIGURE 34

As shown on figure 27, a large number of split-spaced "A" holes were drilled in the lifted area after February 25, 1953, the date the uplift hole was grouted. The split holes were probably required to repair the damaged foundation. They may also account for some of the overrun in drilling quantities. With the widespread jacking of both abutments, it is not possible to determine how much grout was used to grout original voids in the rock, how much grout was used in filling voids created by lifting the rock, and how much was used to repair cracked grout in previously grouted voids.

Based on computations of effective weight versus effective uplift, it is considered that most pressures used in the "B" and "C" holes exceeded safe uplift pressures. In particular, the "C" holes grouted without packer with 150 psi when the adjacent concrete had been placed to a height of only 25 feet would have a computed safety factor below 1.0. The sections of the "C" holes grouted with a packer at a depth of 25 feet with a pressure of 200 psi and at a depth of 50 feet with a pressure of 250 psi would also have a computed factor of safety considerably below 1.0.

Special care must be used in grouting rock with continuous bedding planes and especially with thin-mix grout. The thin-mix grout can spread over a large area quickly; and with its slow set time, the thin-mix grout can exert full uplift pressure over a large area for an extended period of time.

(e) Spacing of holes and closure. There was a progressive decrease in unit take from the primary through secondary, tertiary, quaternary and quinary "A" holes. This indicates that the curtain was tightening with closure. However, split-spacing was discontinued when the spacing between holes along the line was at 5-foot centers. When grouting was discontinued, the unit take on 5th order holes was 0.33 sacks per foot of hole and with an average take of 45 sacks per hole. However, it is not known how much of the grout was filling existing voids in the rock or how much was going into voids created by lifting the rock with excessive pressures. The records indicate that lifting of the structure (and rock) was still occurring near the end of the grouting program when some of the closure holes were being grouted.

The criteria for discontinuing split-spacing are not presented in the grouting records. On some jobs, higher order holes are drilled and grouted in any zone in which the take in the same zone in either adjacent hole exceeds a specific amount of cement per linear foot of the zone grouted; the specific amount may range from 0.1 to 0.2 sack per foot of grouted zone. On other jobs if the pattern closure holes accept more than 10 to 20 sacks or if there is any question as to the adequacy of the grouting, i.e. holes lost, interconnection etc., additional holes are drilled and grouted until the curtain is considered tight. With reference to figures 27, 28 and 29, there are many places where split spacing would be justified under normal procedures. The following are but a few of examples that could be cited:

(1) A split to 5 feet should have been provided between hole 169 in block 5 which took 1,093 sacks (uplift occurring) and the adjacent hole in block 6 that took 191 sacks.

(2) In block 23, a 2-1/2-foot split should have been drilled between the two holes which took 100 and 201 sacks.

(3) In block 24, 2-1/2-foot splits should have been drilled between each of the holes which took 265 and 361 sacks; 129 and 480 sacks; 218 and 585 sacks.

(4) In block 27, 2-1/2-foot splits should have been drilled between the holes taking 101 and 569 sacks, 243 and 348 sacks and 124 and 375 sacks.

If lifting of the rock is occurring, additional split spacing as per the examples above will not result in reaching normal closure criteria. Each split-spaced hole will continue to take grout as the excessive uplift pressure creates its own void.

(f) Refusal criteria. The criteria used for refusal is apparently Bureau standards. It is considered that requiring the contractor to stay on a hole up to 20 minutes to inject 1 cubic foot of grout mixture is excessive. It is considered that refusal could be adequately defined as less than 1 cubic foot of mixture in 5 minutes at the injection pressure, which is the same as the criterion often used for not grouting a zone if the water take is less than 1 cubic foot in 5 minutes at the injection pressure. Use of a more relaxed refusal criteria would be an incentive for the contractor not to "kill the hole" by slugging.

(g) Final backfilling of holes. To assure that all completed holes were filled with solid grout, each hole should have been sounded with a heavy rod and any voids found filled with thick grout using a tremie pipe. These procedures were particularly needed for holes using thin mixes until refusal occurred.

12. Unexpected geologic conditions encountered during grouting. It is not discernible how many of the unknown foundation problems were discovered during the excavation and how many were discovered during drilling and grouting. Five faults and the clay seam (lift seam) on the left abutment were discovered prior to the commencement of any grouting. The difficulty and effort required to wash out the clay in the lift seam was discovered during the drilling and grouting. The grouting report indicates that some clay seams or partings were found in the "C" holes and attempts were made to wash out the clay between the single line of holes. It is assumed that these clay seams were discovered during grouting as there were very few exploratory holes along the axis prior to construction.

13. Grout takes as related to geology. The grout takes in the washed-out clay seam were as expected. There were appreciable grout takes in the shafts at the ends of the fault zones 3, 4 and 6; and there were appreciable grout takes in the holes drilled along and across the faults beneath the dam. As faulting involves disturbance of the rock alongside appreciable grout takes should be expected in these areas.

Other than the faulted zones most of the weathered and open rock was removed in excavating for the dam. The sparsity of information along the dam axis makes it difficult to postulate what the grout takes should have been. Furthermore, the grout takes were undoubtedly materially increased by lifting of the rock.

14. Evaluation of grouting and drainage. The primary objectives of the grouting and drainage are to reduce the amount of foundation seepage to an acceptable volume and to reduce the uplift pressures on the structure and in the foundation rock to safe limits. The "B" hole treatment for the lift seam on the left abutment can more probably be classified as a special treatment used in lieu of removal of the rock above the seam. The performance of the dam to date indicates that the treatment was adequate to prevent settlement or sliding of the five blocks in the treated area.

(a) Seepage. Plots of the flows from the foundation drain holes are presented on figure 35. The data indicate that the overall volume of drain flows is relatively low for a dam of this height. However, the plots indicate large fluctuations in the flows from blocks 20, 21, 22 and 23 when the reservoir level changes back and forth from about El. 3,500 to El. 3,560. For instance, the flow from block 23 increased in 1980 from a flow of 170 gph at a reservoir level of El. 3,490 to 650 gph at a reservoir level of El. 3,560. On a hydraulic head increase of 26 percent, the drain flow increased 382 percent. This increased flow probably includes seepage through the upstream construction joints and may not be related to foundation seepage. A letter[6] dated December 31, 1959 from the Regional Director to Assistant Commissioner and Chief Engineer, subject: "Water Leakage into the block 24 elevator and shaft and inspection gallery at elevation 3,550 -- Hungry Horse Dam etc." states in part: "Construction joints in blocks 3, 6, 8, 21 and 23 leak at the floor level when

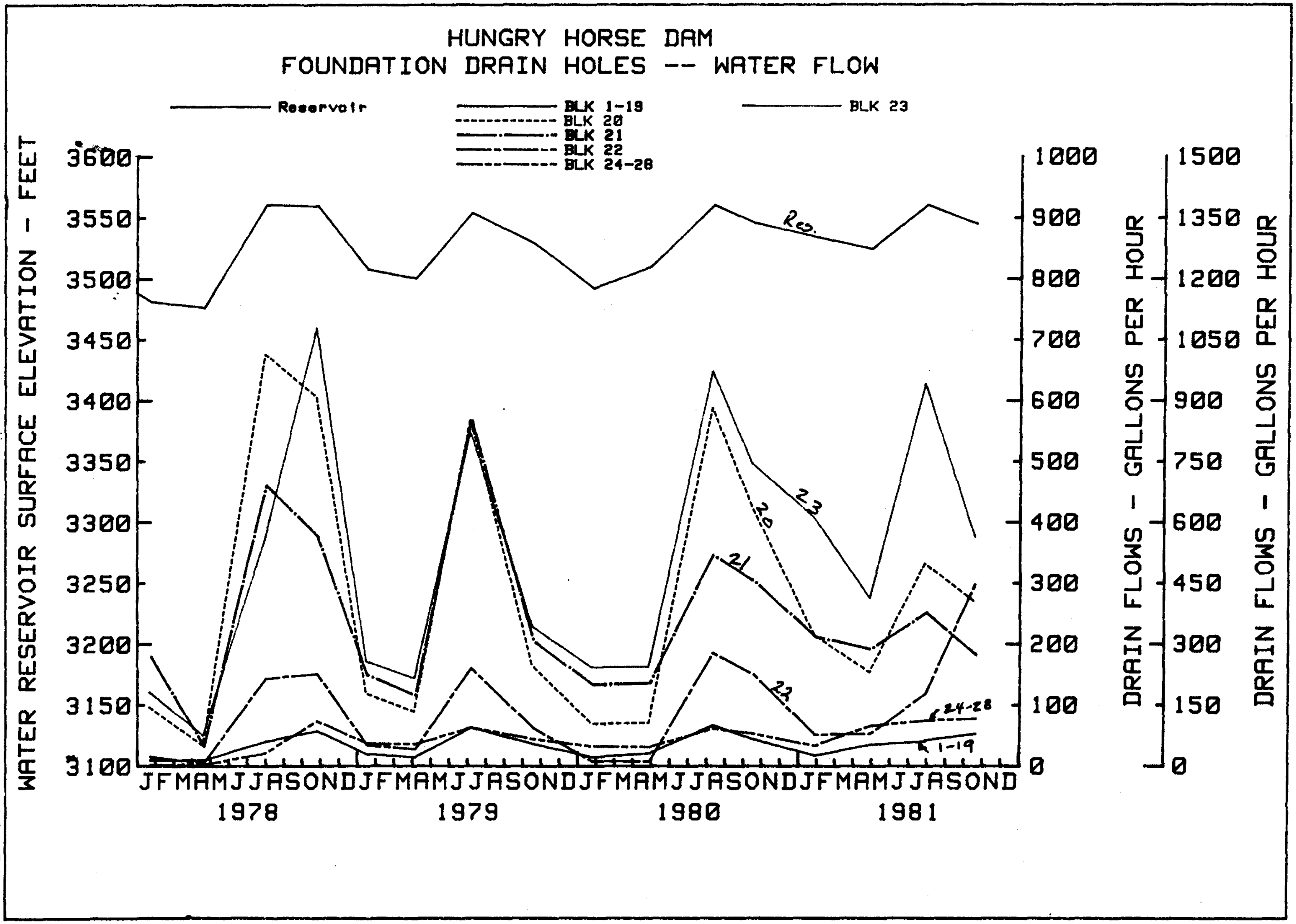


FIGURE 35

ENGINEERING & RESEARCH CENTER - STRUCTURAL BEHAVIOR SECTION the reservoir water surfaces exceed elevation 3550. Elevation 3555 construction joint leaks in blocks 4, 11, 13, 19 and 22 during high water."

Drawing 447-105-1086, "Contraction Joint Grouting, Progress Chart" indicates that all the longitudinal joints were grouted prior to the end of May 1952 and that the transverse joints below El. 3,400 were grouted by the end of May 1952. The transverse joints above El. 3,400 were grouted during the period of January-April 1953. Grouting of the "A" holes was started on September 25, 1952 and was completed on May 26, 1953. Lifting of the ends of the dam by excessive grout pressure would tend to increase the length along the axis near the base and tend to shorten or compress the length along the axis at the crest. As the dam has a curvature, it would have more flexibility than a straight-axis dam. Nevertheless, it is possible that lifting of the ends of the dam could have caused grouted construction joints to be broken, particularly near the crest of the dam where the dam blocks would be pushing against each other.

(b) Uplift. The plan and sections of the uplift pressure pipes are shown on figure 36. The data from the uplift pipes for the years 1965 through 1981 are plotted on figures 37 through 43, and the data for October 20, 1981 are plotted on sections through the dam for lines 1 through 7 on figures 44 through 50. These plots indicate a very high drop in uplift pressure from the most upstream pipe (A) to the pipes immediately at and downstream of the foundation galleries. Uplift pressures beneath the downstream half of the dam are very low.

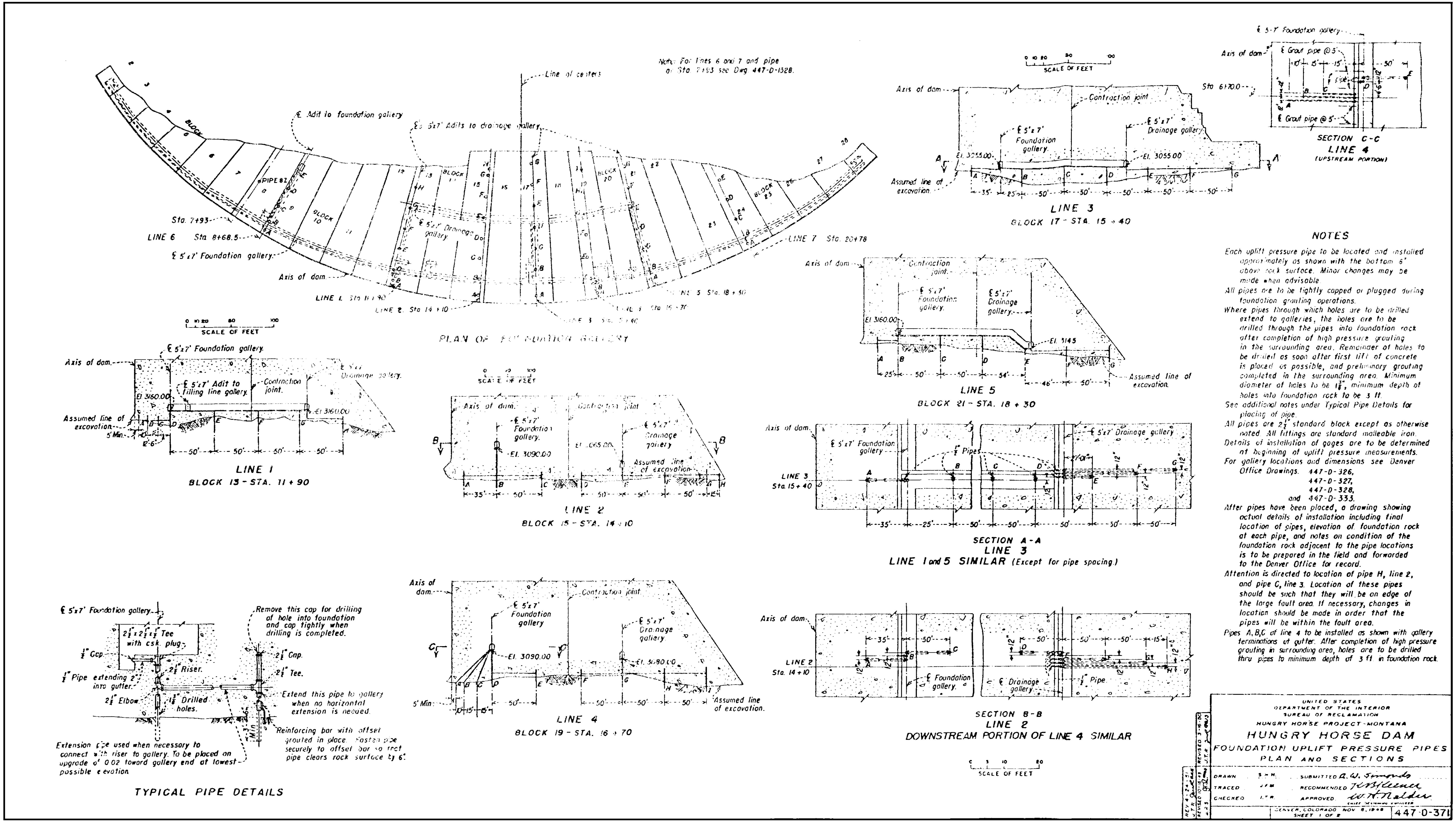
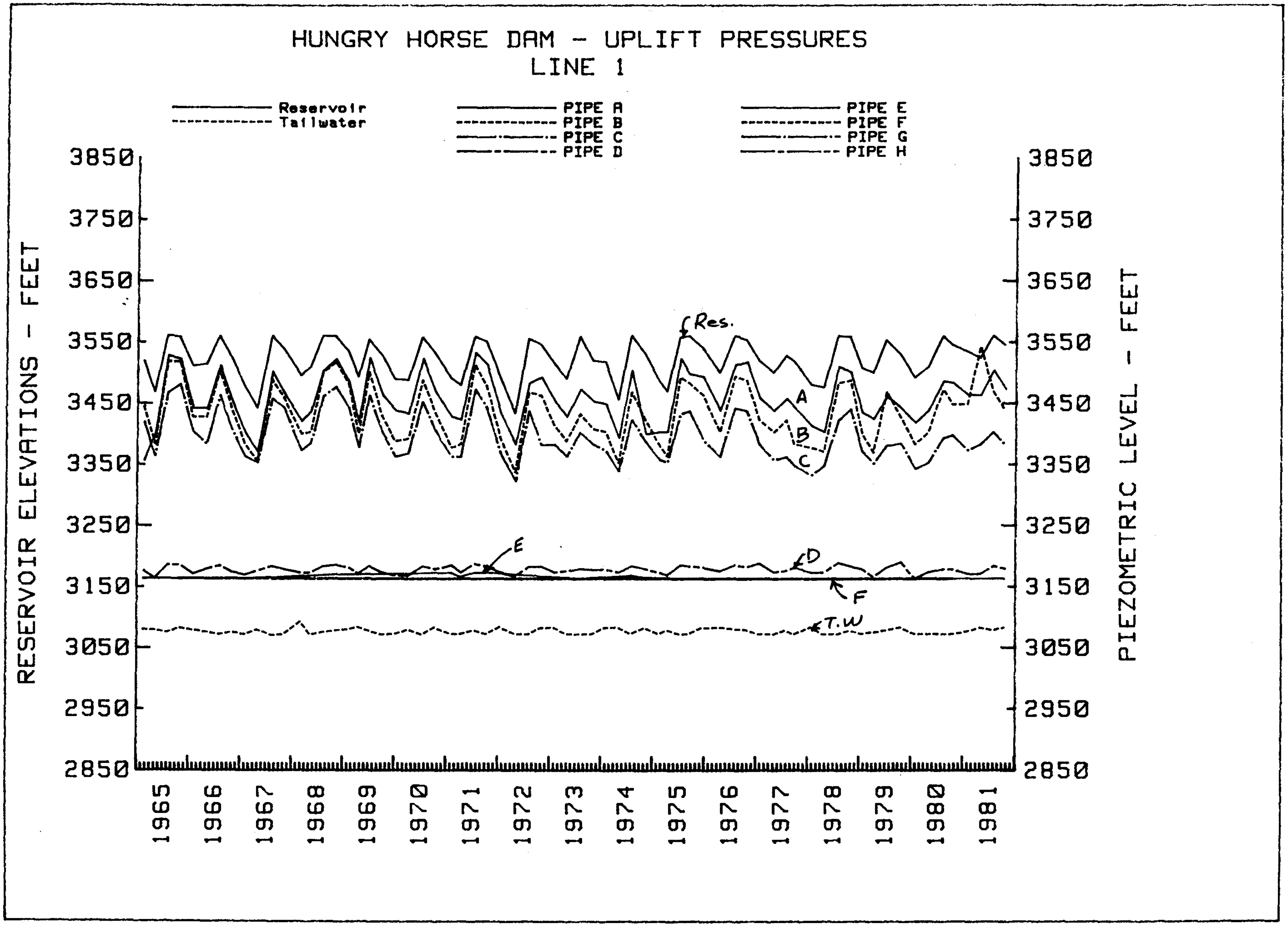

PLAN OF FOUNDATION GALLERY
LINE 1
BLOCK 13 - STA. 11 + 90
LINE 2
BLOCK 15 - STA. 14 + 10
LINE 3
BLOCK 17 - STA. 15 + 40
LINE 4
BLOCK 19 - STA. 16 + 70
LINE 5
BLOCK 21 - STA. 18 + 30
SECTION C-C
LINE 4
(UPSTREAM PORTION)
SECTION A-A
LINE 3
LINE 1 and 5 SIMILAR (Except for pipe spacing)
SECTION B-B
LINE 2
DOWNSTREAM PORTION OF LINE 4 SIMILAR
TYPICAL PIPE DETAILS
SCALE OF FEET
NOTES
Each uplift pressure pipe to be located and installed approximately as shown with the bottom 6" above rock surface. Minor changes may be made when advisable.
All pipes are to be tightly capped or plugged during foundation grouting operations.
Where pipes through which holes are to be drilled extend to galleries, the holes are to be drilled through the pipes into foundation rock after completion of high pressure grouting in the surrounding area. Remainder of holes to be drilled as soon after first lift of concrete is placed as possible, and preliminary grouting completed in the surrounding area. Minimum diameter of holes to be 1¼", minimum depth of holes into foundation rock to be 3 ft.
See additional notes under Typical Pipe Details for placing of pipe.
All pipes are 2½" standard black except as otherwise noted. All fittings are standard malleable iron.
Details of installation of gages are to be determined at beginning of uplift pressure measurements.
For gallery locations and dimensions see Denver Office Drawings 447-D-326, 447-D-327, 447-D-328, and 447-D-333.
After pipes have been placed, a drawing showing actual details of installation including final location of pipes, elevation of foundation rock at each pipe, and notes on condition of the foundation rock adjacent to the pipe locations is to be prepared in the field and forwarded to the Denver Office for record.
Attention is directed to location of pipe H, line 2, and pipe C, line 3. Location of these pipes should be such that they will be on edge of the large fault area. If necessary, changes in location should be made in order that the pipes will be within the fault area.
Pipes A, B, C of line 4 to be installed as shown with gallery terminations at gutter. After completion of high pressure grouting in surrounding area, holes are to be drilled thru pipes to minimum depth of 3 ft in foundation rock.
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
HUNGRY HORSE PROJECT-MONTANA
HUNGRY HORSE DAM
FOUNDATION UPLIFT PRESSURE PIPES
PLAN AND SECTIONS
DENVER, COLORADO NOV. 8, 1948
447-D-371

FIGURE 36



ENGINEERING & RESEARCH CENTER - STRUCTURAL BEHAVIOR SECTION

FIGURE 37

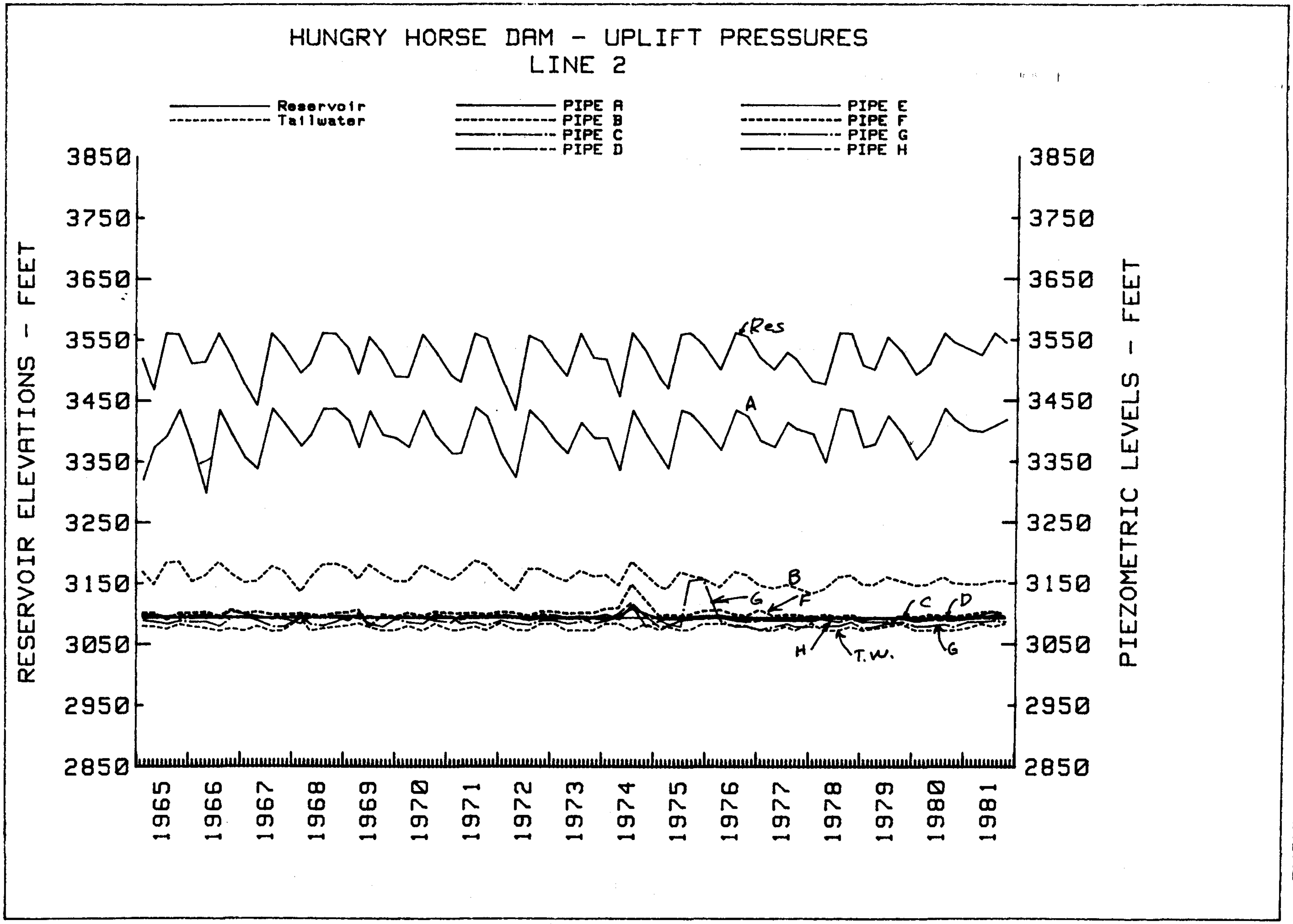

HUNGRY HORSE DAM - UPLIFT PRESSURES
LINE 2
Reservoir
Tailwater
PIPE A
PIPE B
PIPE C
PIPE D
PIPE E
PIPE F
PIPE G
PIPE H
RESERVOIR ELEVATIONS - FEET
PIEZOMETRIC LEVELS - FEET
3850
3750
3650
3550
3450
3350
3250
3150
3050
2950
2850
1965
1966
1967
1968
1969
1970
1971
1972
1973
1974
1975
1976
1977
1978
1979
1980
1981
Res
A
B
G
F
C
D
H
T.W.
G
ENGINEERING & RESEARCH CENTER - STRUCTURAL BEHAVIOR SECTION
Header file /HHUH2

FIGURE 3.8

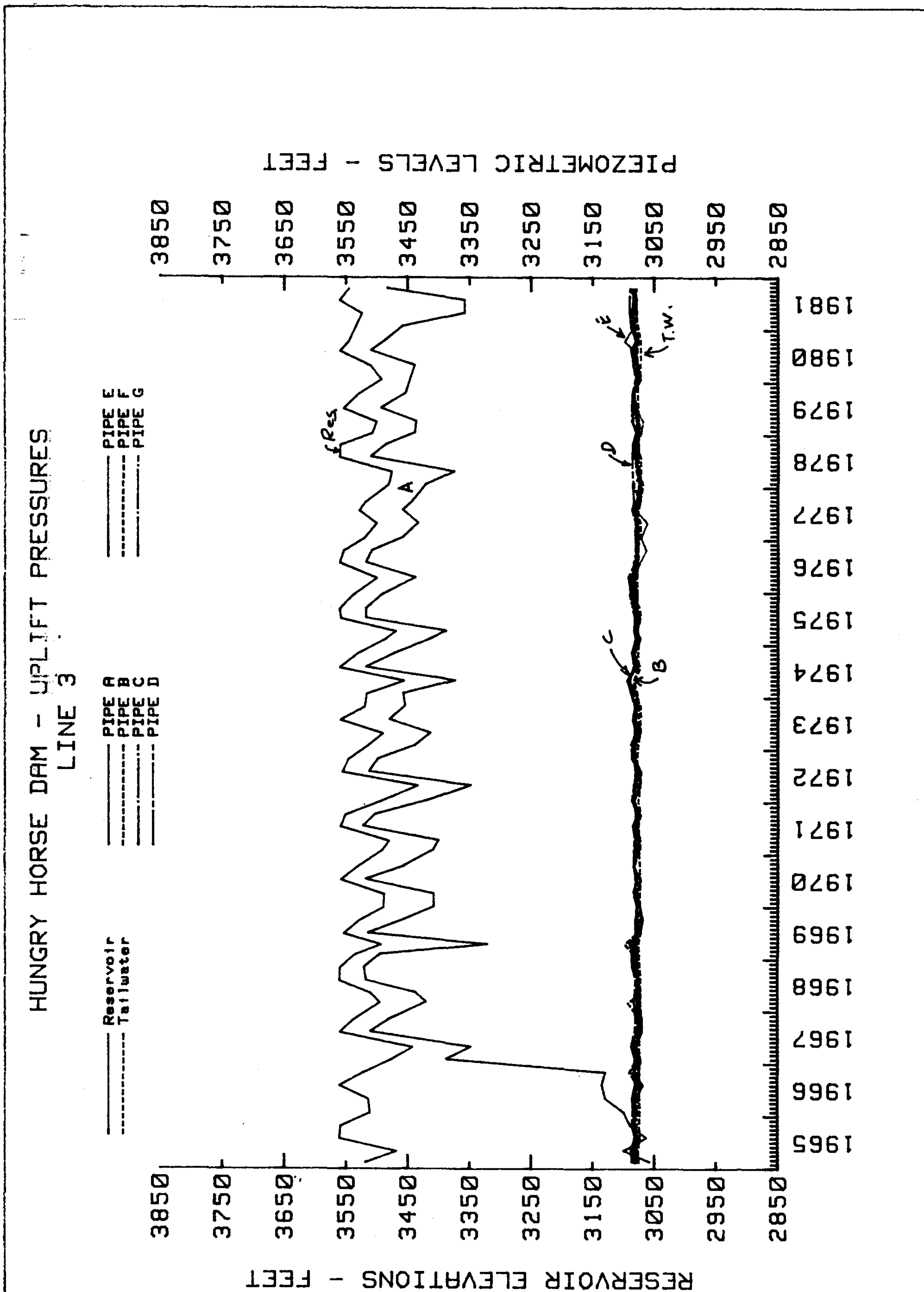
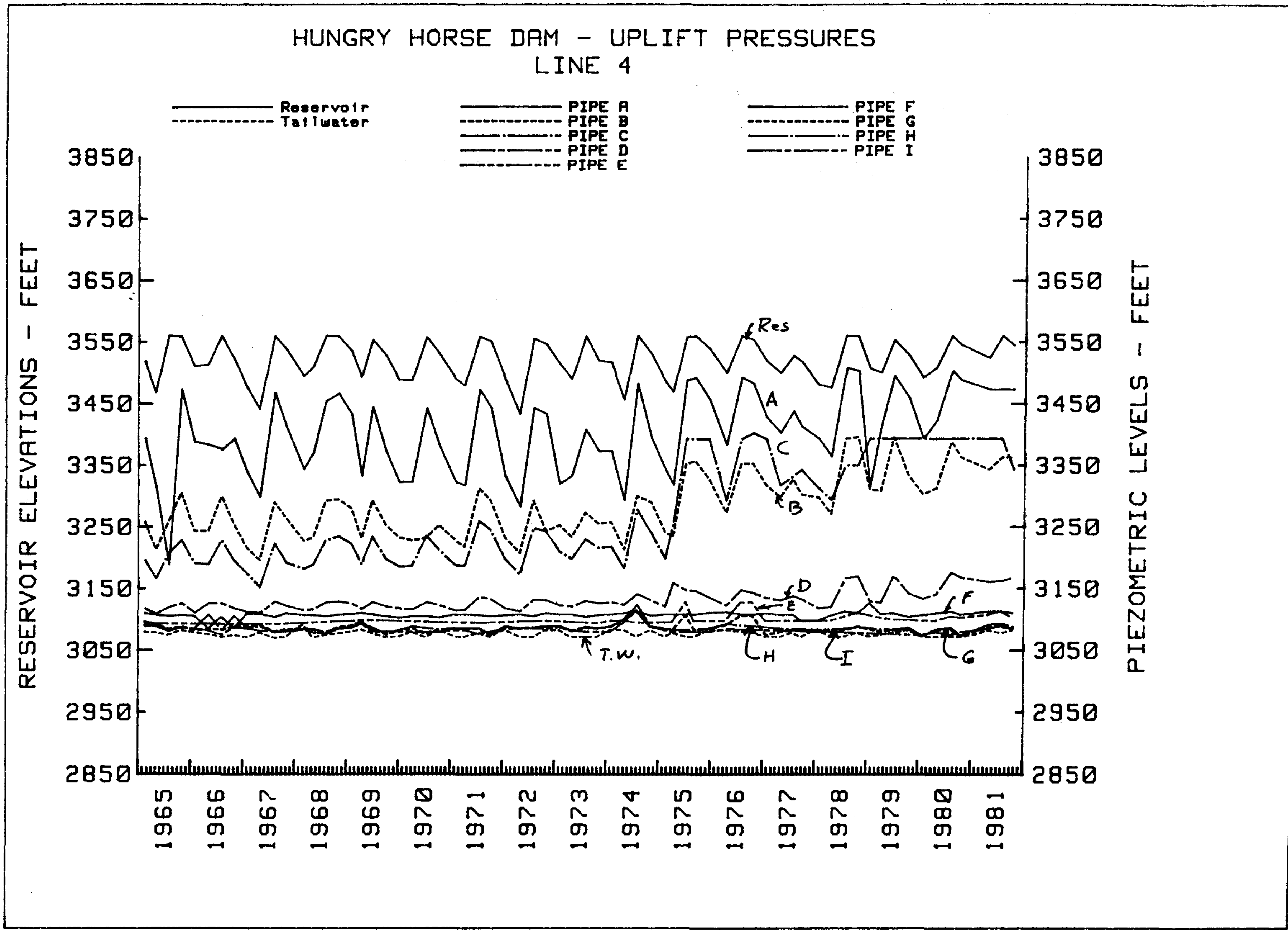

HUNGRY HORSE DAM - UPLIFT PRESSURES
LINE 3
Reservoir
Tailwater
PIPE A
PIPE B
PIPE C
PIPE D
PIPE E
PIPE F
PIPE G
PIEZOMETRIC LEVELS - FEET
RESERVOIR ELEVATIONS - FEET
3850
3750
3650
3550
3450
3350
3250
3150
3050
2950
2850
1965
1966
1967
1968
1969
1970
1971
1972
1973
1974
1975
1976
1977
1978
1979
1980
1981
Res
A
B
C
D
E
T.W.
ENGINEERING & RESEARCH CENTER - STRUCTURAL BEHAVIOR SECTION

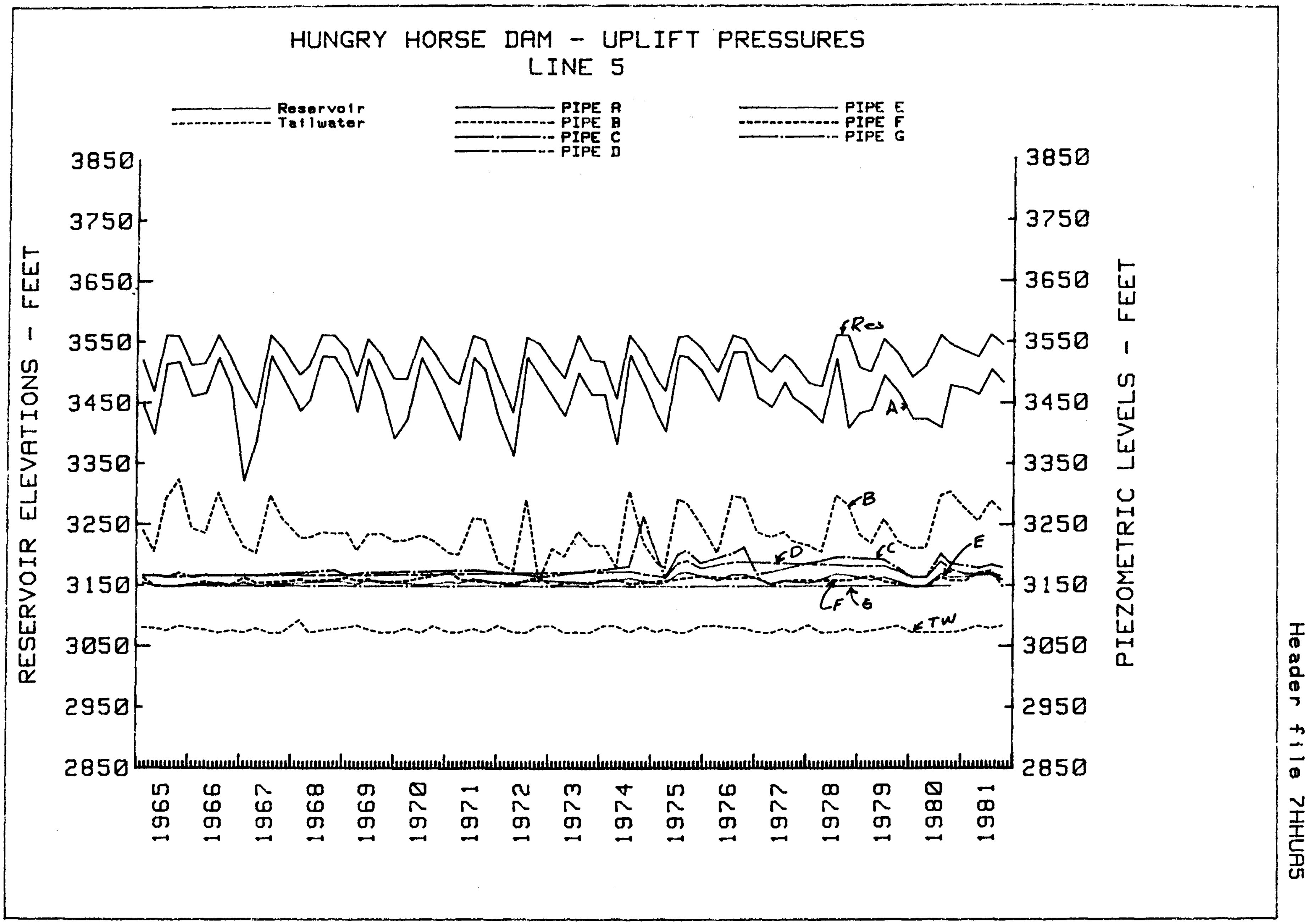
FIGURE 39



ENGINEERING & RESEARCH CENTER - STRUCTURAL BEHAVIOR SECTION

Header file 7HHUA4

FIGURE 40

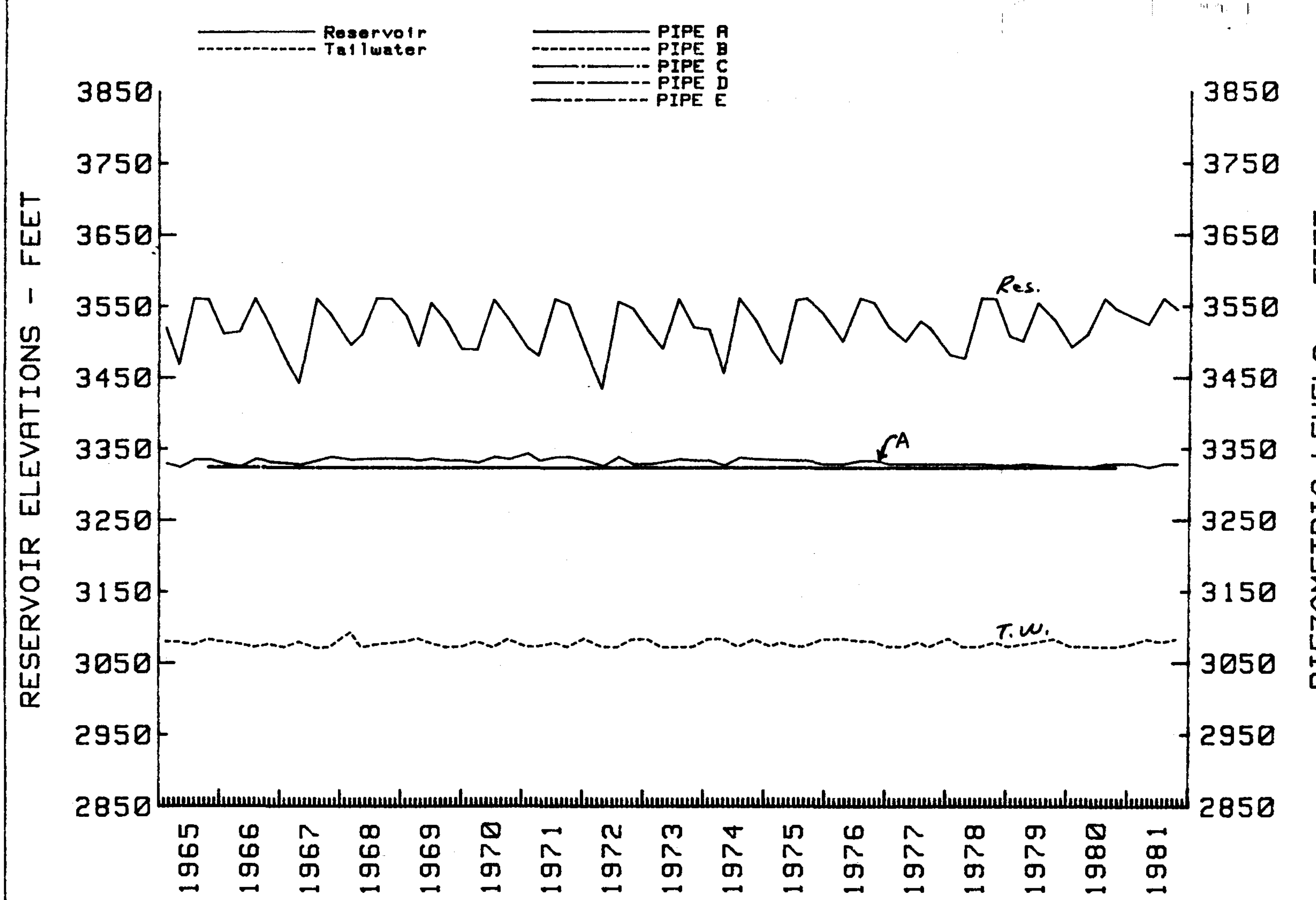


79

FIGURE 41

ENGINEERING & RESEARCH CENTER - STRUCTURAL BEHAVIOR SECTION

HUNGRY HORSE DAM - UPLIFT PRESSURES
LINE 6



ENGINEERING & RESEARCH CENTER - STRUCTURAL BEHAVIOR SECTION

FIGURE 42

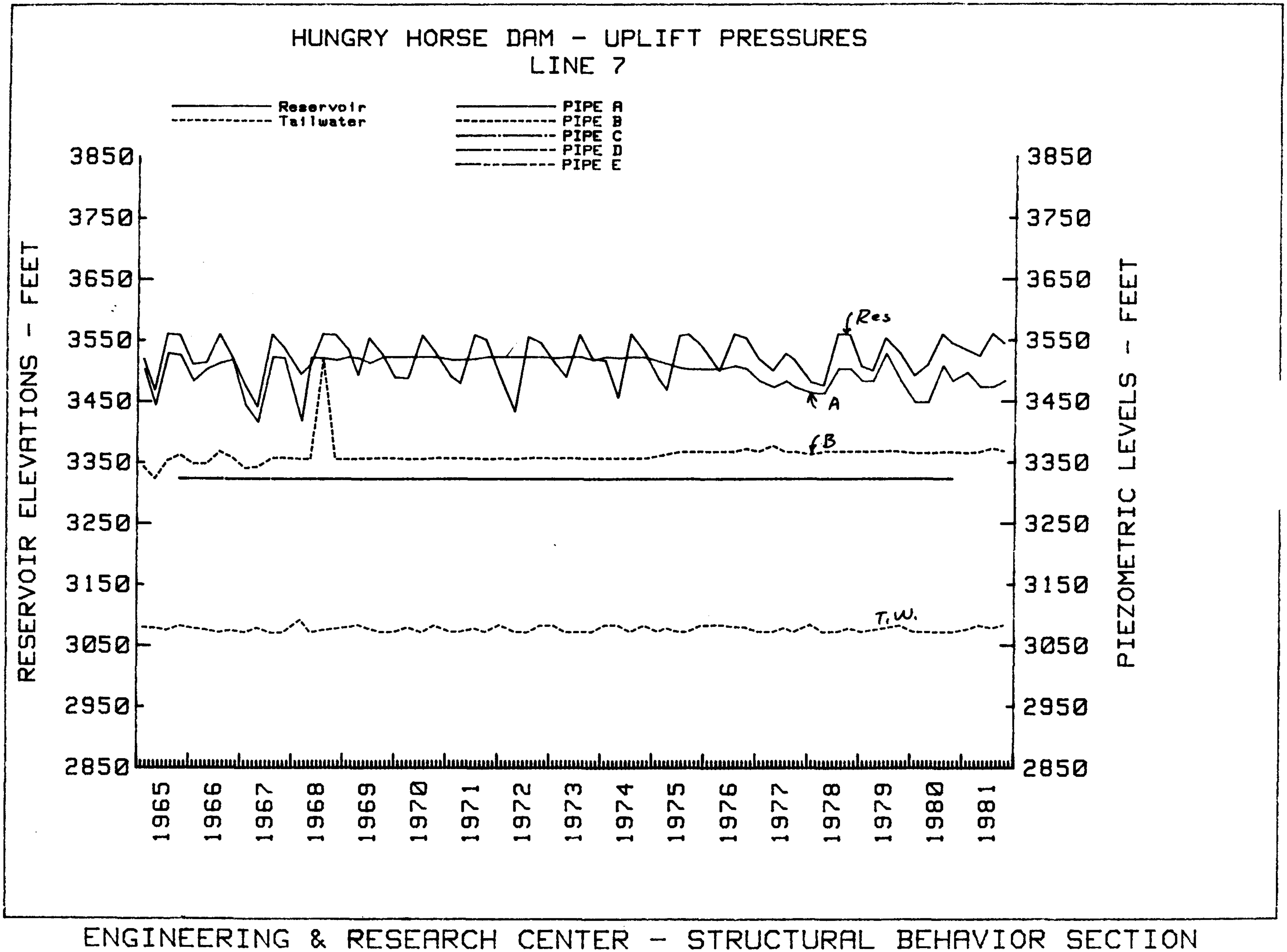
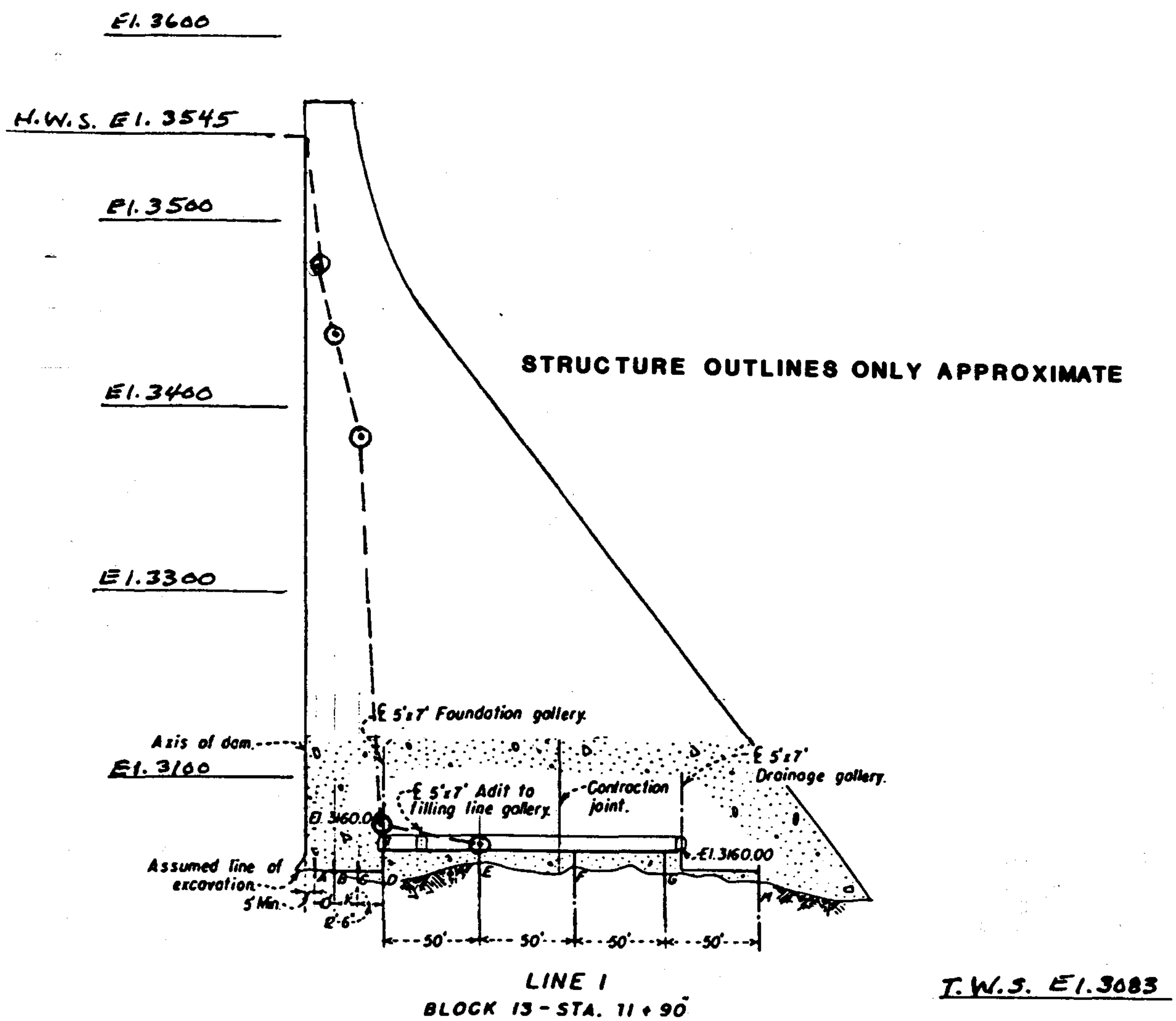

HUNGRY HORSE DAM - UPLIFT PRESSURES
LINE 7
Reservoir
Tailwater
PIPE A
PIPE B
PIPE C
PIPE D
PIPE E
RESERVOIR ELEVATIONS - FEET
PIEZOMETRIC LEVELS - FEET
3850
3750
3650
3550
3450
3350
3250
3150
3050
2950
2850
1965
1966
1967
1968
1969
1970
1971
1972
1973
1974
1975
1976
1977
1978
1979
1980
1981
Res
A
B
T.W.
ENGINEERING & RESEARCH CENTER - STRUCTURAL BEHAVIOR SECTION

FIGURE 43



REF. DWG. 447-D-371

CONTRACT NO. 2-07-DV-00148

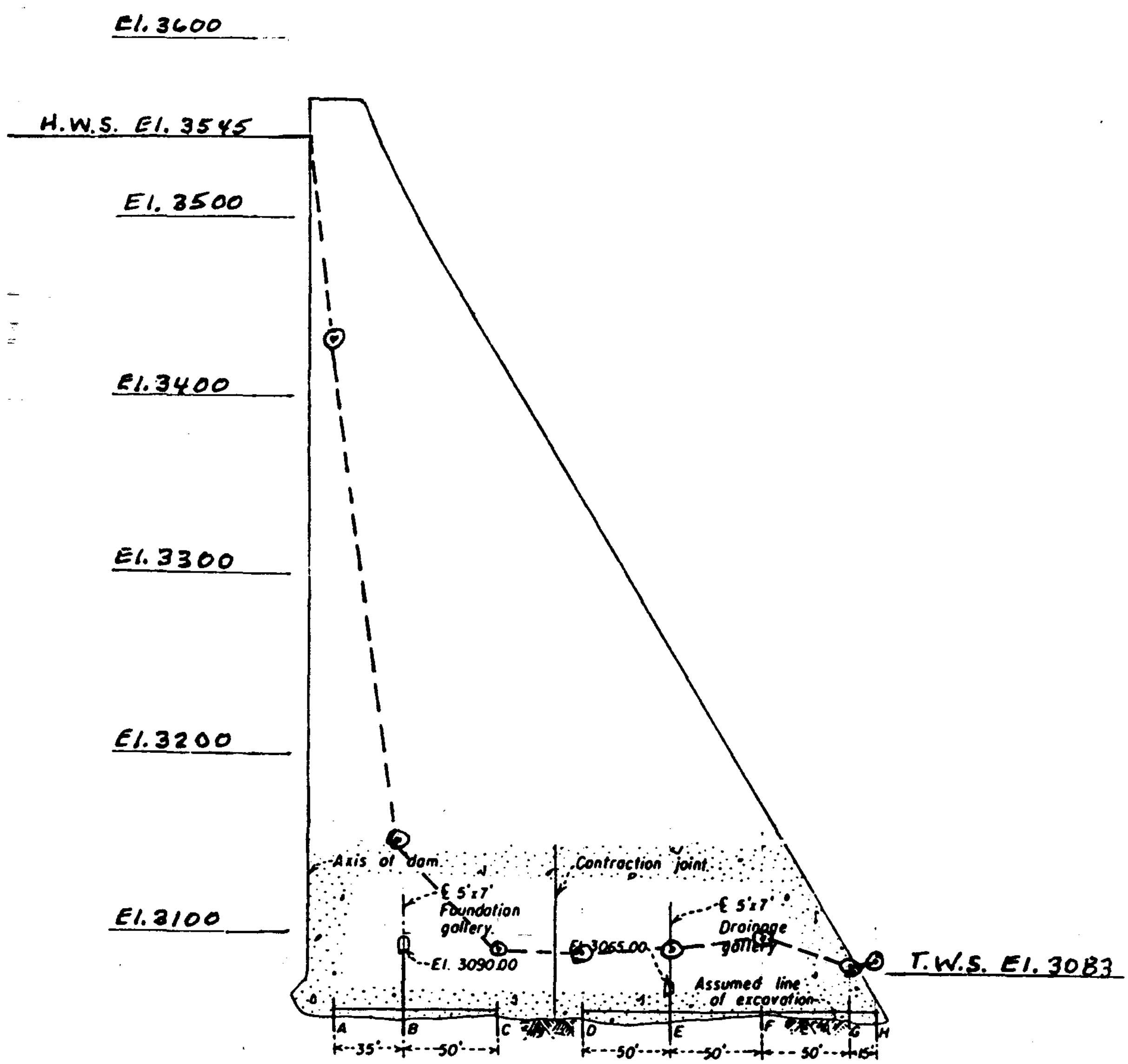
U. S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GROUTING RESEARCH PROGRAM

HUNGRY HORSE DAM

UPLIFT PRESSURES
LINE 1 PROFILE 10-20-81

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

FIGURE 44



LINE 2
BLOCK 15 - STA. 14+10

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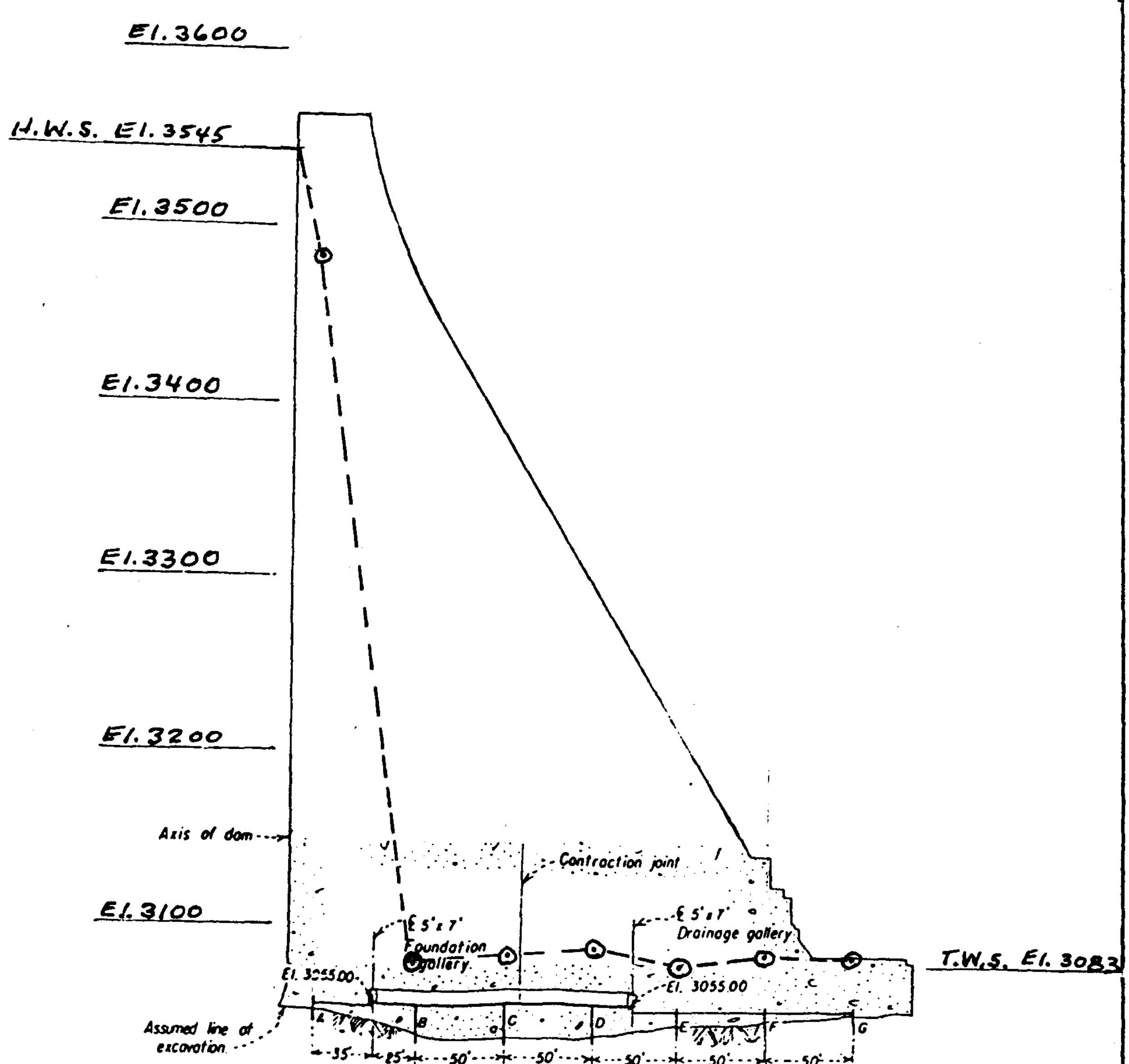
GROUTING RESEARCH PROGRAM

HUNGRY HORSE DAM

UPLIFT PRESSURES
LINE2 PROFILE 10-20-81

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

FIGURE 45



LINE 3

BLOCK 17 - STA. 15 + 40

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HUNGRY HORSE DAM

UPLIFT PRESSURES

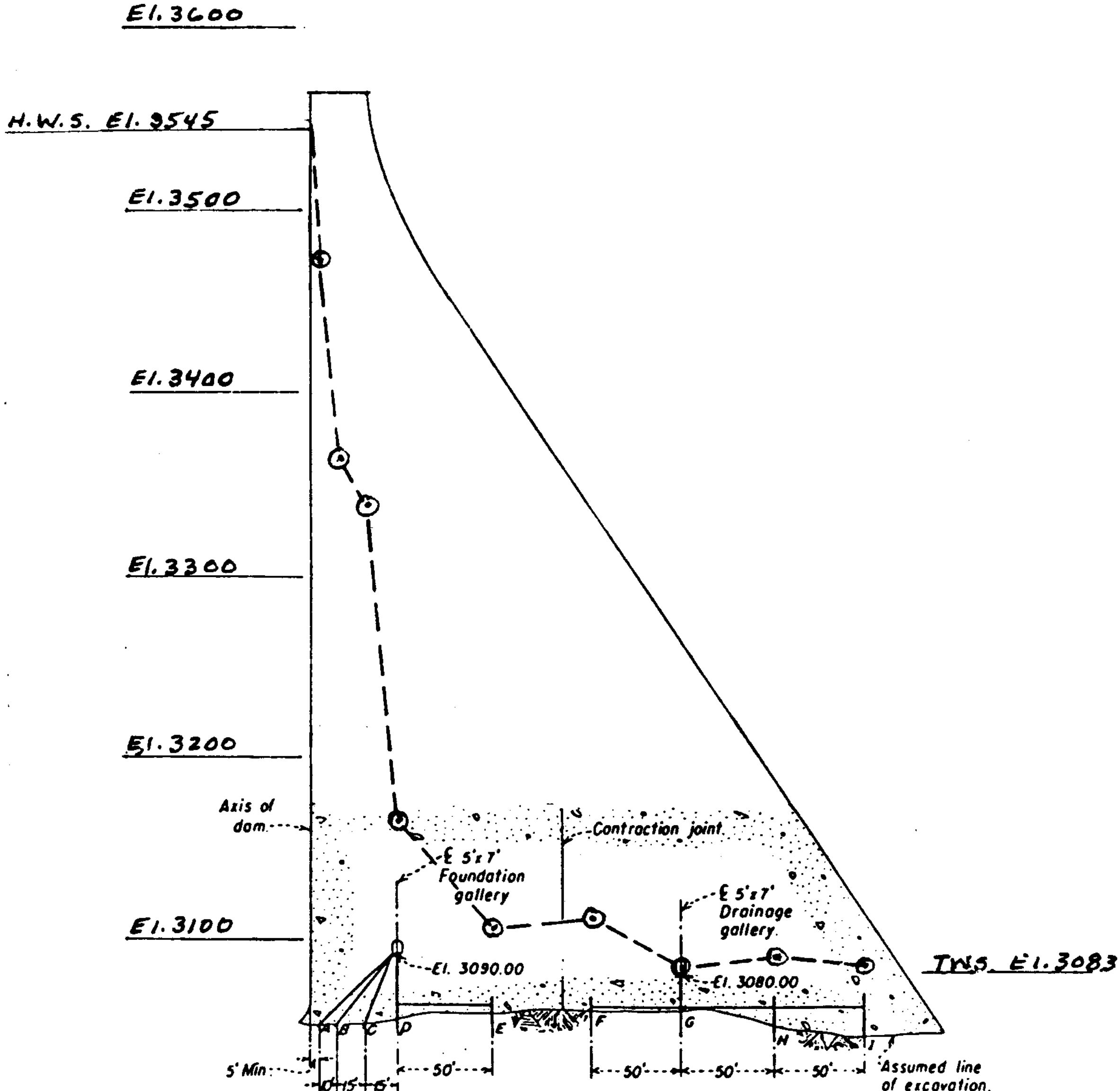
LINE 3 PROFILE 10-20-81

CONTRACT NO. 2-07-DV-00148

PREPARED BY: CLAUDE A. FETZER

JULY 1982 CONSULTING GEOTECHNICAL ENGINEER

FIGURE 46



LINE 4
BLOCK 19 - STA. 16 + 70

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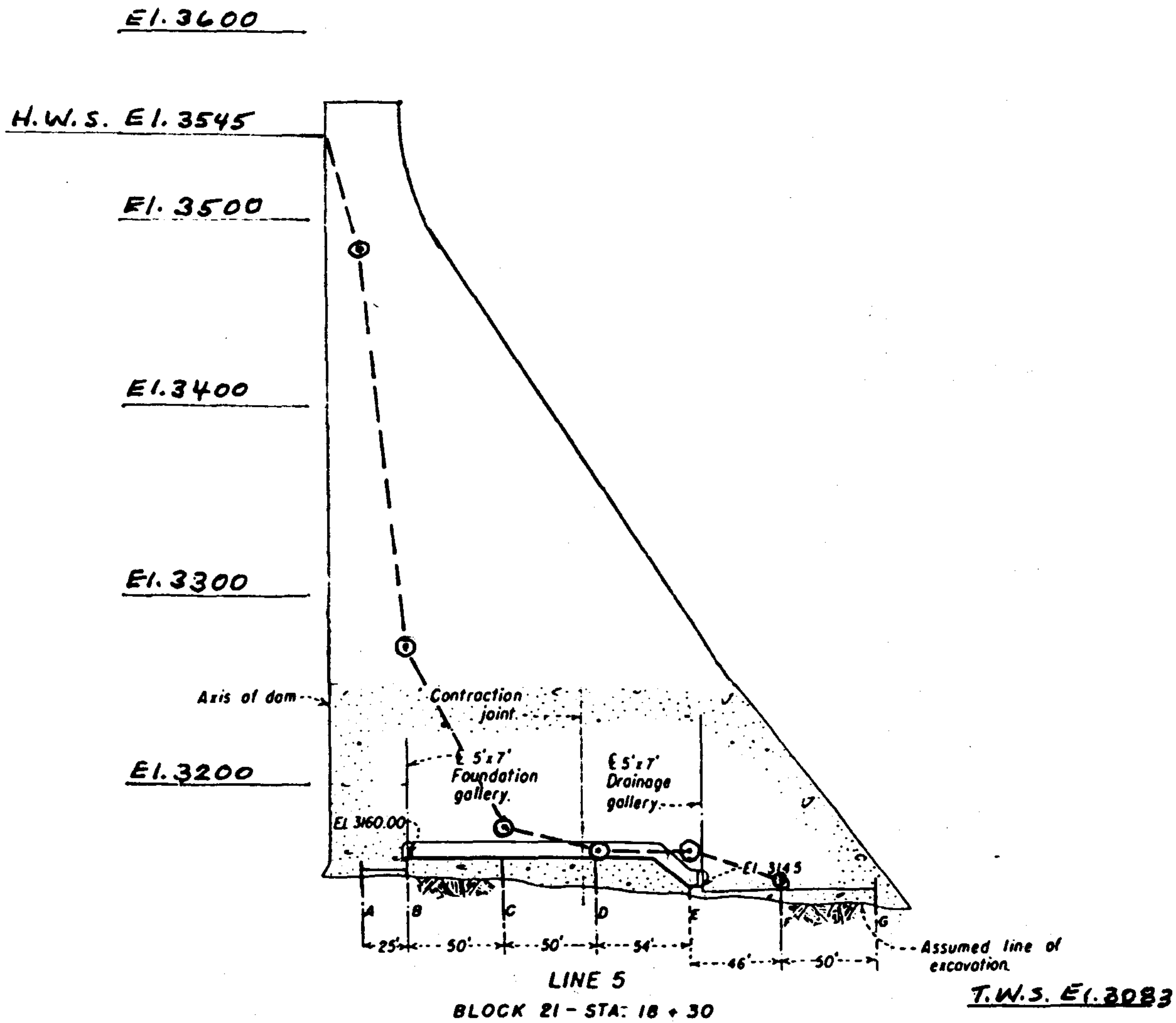
HUNGRY HORSE DAM

UPLIFT PRESSURES
LINE4 PROFILE 10-20-81

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

FIGURE 47



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GROUTING RESEARCH PROGRAM

HUNGRY HORSE DAM

UPLIFT PRESSURES
LINE 5 PROFILE 10-20-81

PREPARED BY: CLAUDE A. FETZER
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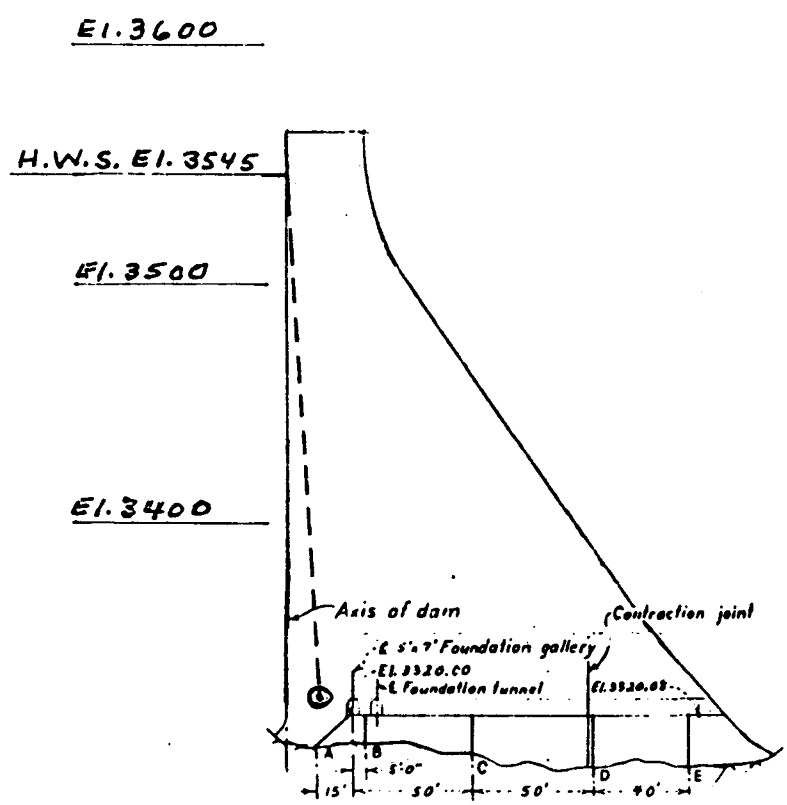
FIGURE 48



LINE 6
BLOCK 9 - STA. 8+68.5

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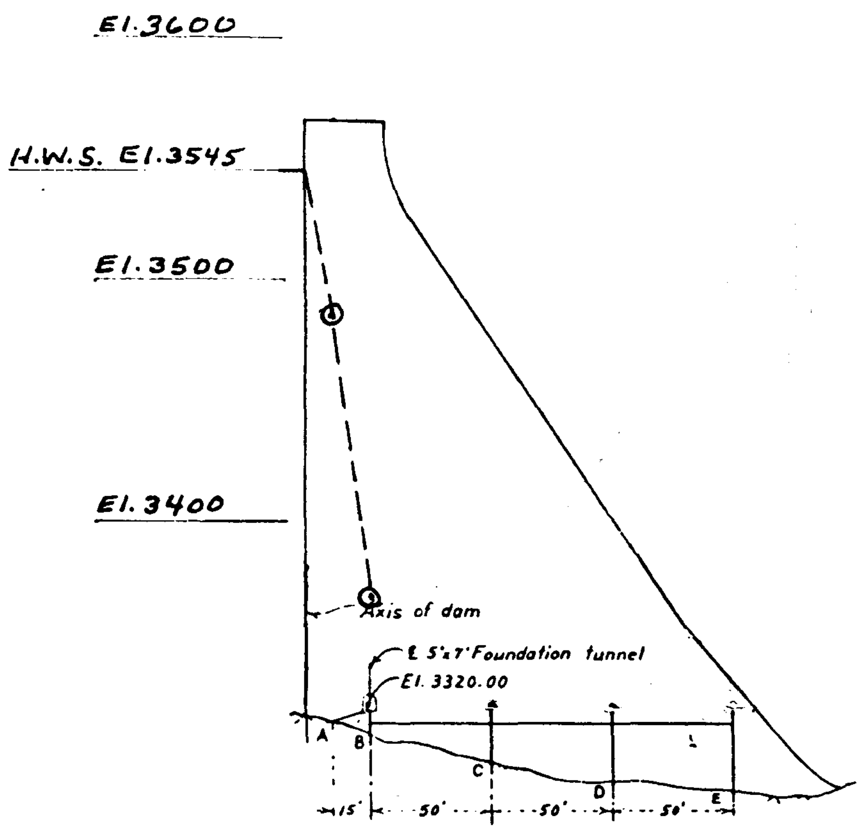
U. S. DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GROUTING RESEARCH PROGRAM

HUNGRY HORSE DAM

UPLIFT PRESSURES
LINE 6 PROFILE 10-20-81

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

FIGURE 49



LINE 7
BLOCK 24 - STA. 20+78

CONTRACT NO. 2-07-DV-00148

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

HUNGRY HORSE DAM

UPLIFT PRESSURES
LINE 7 PROFILE 10-20-81

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

FIGURE 50

As shown on figure 31, the reservoir elevation from 1 July 1952 until about 1 November 1952 was at approximate El. 3,410, thus creating a hydraulic head on the dam of about 265 feet. The drain holes were not drilled until the period of December 5, 1952 to June 11, 1953. Hence, the dam sustained approximately 50 percent of maximum head without the benefit of either the deep grout curtain or the foundation drainage curtain. The records do not indicate any outbreak of seepage downstream during this period. The current records for the uplift pipes start January 12, 1953 which was after the "A"-hole grouting had started.

(c) Conclusion. The data indicate that the grouting and drainage have reduced the foundation seepage flows to minimal levels and have adequately controlled the uplift pressures at the base of the structure. However, there are no instruments to measure the uplift pressures in the foundation rock at depths greater than 3 feet. Consideration should be given to installing sealed piezometers from the downstream drainage gallery into major joints or bedding planes in the foundation rock.

V. RECOMMENDATIONS

15. Hungry Horse Dam.

(1) Sealed piezometers be installed at depth into major joints or bedding planes in the rock.

(2) Core holes be drilled under Phase II through the lift seam to determine if the grout washed out or deteriorated.

16. Other large concrete dams.

(1) Preconstruction geologic investigations include extensive borings at the final site and of sufficient depth to embrace the grout curtain.

(2) Preconstruction geologic report include analysis of the need for a grout curtain and recommendations based on the geologic conditions regarding the depth and spacing of holes, the angle of holes, anticipated grout takes, and depth, spacing and angle of drainage holes.

(3) The specifications include requirements for: the expected range of water-cement ratios; water pressure tests for each zone with a time limit for the tests; minimum capacity for grout-mixing tubs, agitator sump and grout pump (two 10-cu.ft. mixing tubs or one 20-cu.ft. agitator sump and a 60 gpm pump should provide adequate capacity); a minimum size grouting tube through the packer which will permit placement at a suitable rate (reducers should not be permitted in grout tubes as this restricts flow and causes plugging); provisions for sounding and final back-filling of completed grout holes and payment therefor; storage of adequate quantities of cement at or near the grout plant to insure that holes can be completed without materials delay; and the use

of a placing unit (agitator sump and pump) at an elevation approximately that of hole collar where the elevation difference between the primary grouting plant and the hole collar could result in a gravity pressure greater than the maximum allowable pressure.

(4) The starting grout mixes be based on the takes in the water-pressure tests and that the mix be thickened in accordance with an evaluation of the takes and back pressures. Where there is concern about losing the hole, gradually reduce the water-cement ratio in 1/4 or 1/2 units. As used in the grouting of the lift seam to reduce shrinkage and bleeding, use the thickest grout possible.

(5) Grout injection pressures and water pressure tests be based on a balance of effective pressures at the packer. For abutments the safe pressures may be limited by the shear strength of horizontal or dipping weak seams in the rock and rock bolting may be required prior to "B"-hole grouting.

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

(7) Results of drilling and grouting be continuously evaluated by on-site geologist and engineer to make adjustments for unexpected geologic conditions and that core drilling be used when questions arise on flow of grout, large takes, etc.

(8) Additional foundation drainage lines be installed downstream of the axis for dams with wide bases.

(9) Uplift pipes be installed in weak seams or open joints in the rock at depth below the base.

APPENDIX

APPENDIX A

REFERENCES

1. Soward, Kenneth, Geologic Report on the Hungry Horse Dam Site, Vol. I Text, Maps & Sections, Vol. II Logs of drill holes 1949-1952, September 1952.

2. Savage, J. L. and Berkey, Charles P., letter report dated May 27, 1949 to L. N. McClellan, Chief Engineer, subject: "Inspection and Report on Foundation Excavations at Hungry Horse Damsite".

3. Savage, J. L., "Report on Foundation Excavations at Hungry Horse Damsite", dated September 7, 1949.

4. Walker, Aubrey, memorandum to Construction Engineer dated May 19, 1951, subject: "Washing and grouting of lift seam, left abutment, Hungry Horse Dam".

5. Grouting Report on the Hungry Horse Dam, 1954.

6. Letter dated December 31, 1959, from the Regional Director to Assistant Commissioner and Chief Engineer, subject, "Water Leakage into the block 24 elevator and shaft and inspection gallery", Hungry Horse Dam.

7. Harshman, E. N., letter dated April 27, 1946, to Chief Engineer, subject: "Geologic Inspection of Hungry Horse Dam Site".

8. Harshman, E. N., field trip report dated August 30, 1946, to Chief Engineer, subject: "Geologic inspection of Hungry Horse Damsite".

9. Soward, Kenneth, letter dated November 26, 1946, to Paul A. Jones, Project Engineer, subject "Exploration tunnel B on left abutment, Hungry Horse Project, Montana".

APPENDIX B

PART II--FOUNDATION CONDITIONS AND TREATMENT

CHAPTER II--GEOLOGY

10. Historical Geology. - The high mountains and ridges of the Hungry Horse area are made up of sedimentary rocks known as the Belt Series of the Proterozoic Era. These beds are some of the oldest known in the United States and, in general, dip to the northeast at angles of 15° to 60°. The valleys in this area are covered by younger and softer formations comprised of Tertiary lake beds, Pleistocene glacial deposits, and Recent alluvium.

Three formations of the Belt Series are present in the vicinity of the dam site. The Grinnell argillite is the oldest formation and consists of thinly laminated, massive, dull reddish and greenish-gray argillite with a few gray quartzite beds. The estimated thickness of this formation is 2,000 feet. Above the Grinnell argillite lies the Siyeh formation composed of three members with an estimated thickness of 4,000 to 6,000 feet. This formation is classified as a siliceous, argillaceous, dolomitic limestone. The Hungry Horse Dam is located in this formation (sec. 12). The Missoula formation makes up the upper part of the Belt Series with estimated thicknesses ranging up to 25,000 feet. This stratum is composed of thin-bedded varicolored argillities and massive red- to light-colored quartzites.

A large time-gap separates the Belt Series from rocks of Tertiary Age which are to be found in the upper part of the reservoir area. This area is covered with an unknown thickness of partly indurated sediments which are dominantly lacustrine with occasional small carbonaceous zones. The valleys of the North, Middle, and South Forks of the Flathead River have been covered with two and possibly three sheets of glacial till of Quaternary Age. .

11. Structural Geology. - Several major faults exist in the Hungry Horse area, as shown on figure 4. During the Tertiary period, mountain building forces uplifted and folded the rocks in northwestern Montana, then broke them along steeply dipping thrust faults. These stresses built up until the deformed and stonger Belt Series of rocks were thrust eastward over the younger and weaker Mesozoic rocks of the Great Plains region. The structural valleys and uplifted mountain blocks which trend north-northwest were developed by these thrust faults. These faults dip northeast at an angle of 60° to 80°, and the displacement varies from 10,000 to 30,000 feet.

The Hungry Horse project lies in the mountain block controlled by the Swan and Flathead faults. Many small faults parallel the main faults and others are transverse. Some adjustments are still taking place in this highly faulted area, and earthquake shocks have been recorded in various sections of Montana during the past 20 years. A seismograph has been established near the dam site.

12. Dam Site Geology. - (a) *Bedrock.*— The foundation rock for the dam and appurtenant structures is the Siyeh limestone formation. The average strike of these beds is N 38° W, and the average dip is 30° NE (upstream and into the right abutment). These beds are about 4,000 feet thick in this area, and the dam foundation is situated near the middle of the formation.

This limestone is not pure and contains varying amounts of siliceous, argillaceous and dolomitic materials which have increased its hardness somewhat and greatly lessened its solubility. The beds of these materials are regular and range in thickness from a few inches to several feet. Some calcareous beds are present in the foundation area.

(b) *Joints.*-- Regional stresses and deformations folded and fractured the foundation producing three well-developed, clean-cut joint systems. The average dip and strike of the joints in the three major systems are as follows:

(1) Strike N 45° W, dip 53° SW

(2) Strike N 63° E, dip 80° SE

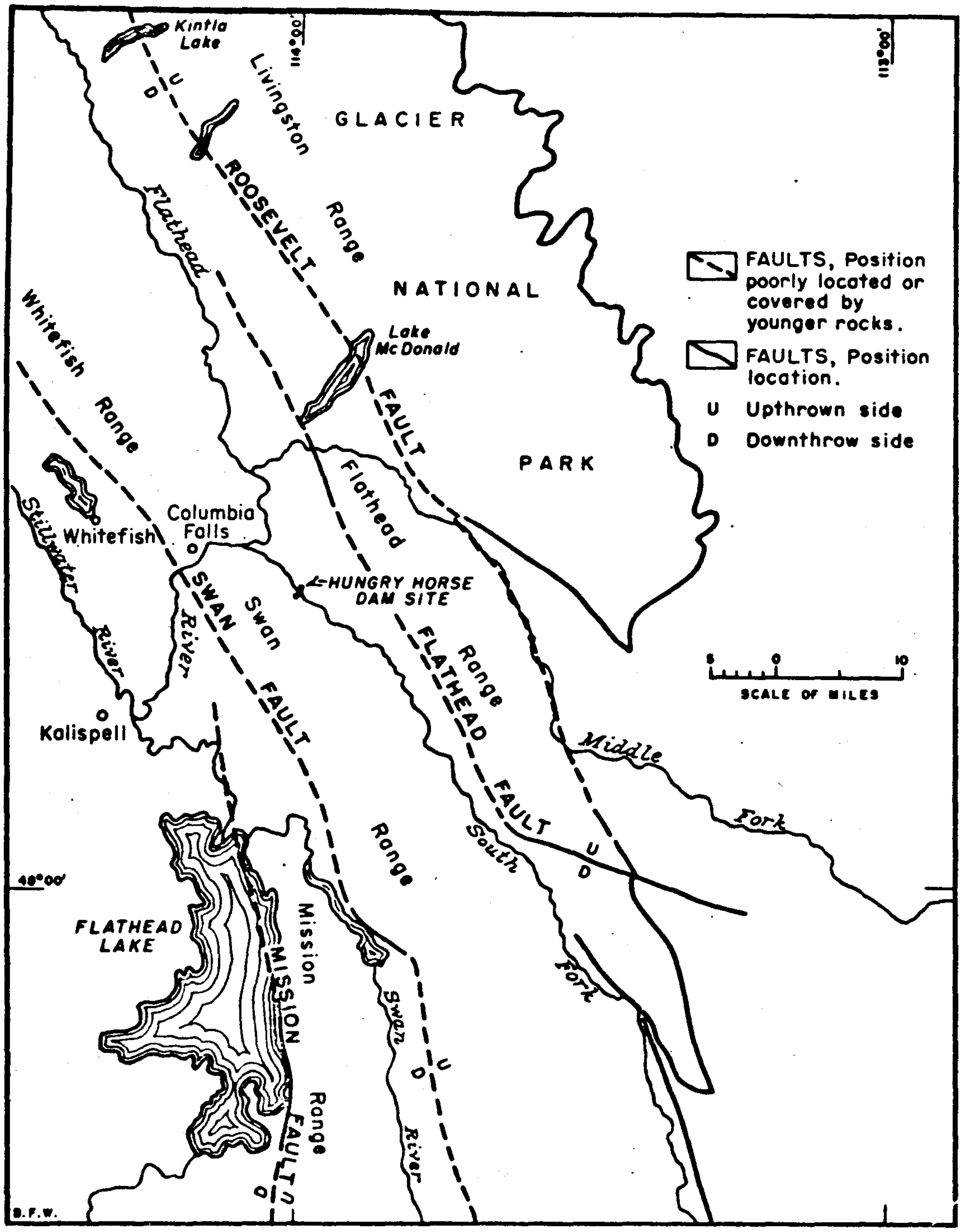


Figure 4.--Sketch showing the location of the major faults in the vicinity of Hungry Horse Dam site.

(3) Strike N 38° E, dip 80° NW

Systems (1) and (2) are the strongest and most continuous at the dam site. They are somewhat open, the openings often being filled with clay. Weathering, which decreases with depth, has taken place along many of the joints. Very minor amounts of clay and weathering, as evidenced by iron staining along the joints, were recognized in drill cores taken at depths of 100 to 150 feet below the ground surface. The joints of system (3) are less continuous than those of systems (1) and (2), are not as open, and have been affected by weathering to a lesser degree.

Some slippage along bedding planes occurred as a direct result of the regional folding. These bedding plane faults were minor features in the foundation and abutment rock with the exception of one zone containing gouge and clay from 1/16 to 4 inches in thickness.

A major fault zone was located in the original diamond drilling exploration of the dam site, and stripping for the foundation uncovered six minor fault zones.

13. Overburden. - The overburden ranged from zero in the outcrop areas to 26 feet in the stream channel. Soil, glacial till, and talus blocks concealed the bedrock on the left abutment except for a very few rock outcrops. A dense stand of pine and other coniferous trees grew on the hillside. The right abutment was only partially covered by trees and undergrowth, and bedrock exposures were numerous with many outcrops near the edge of the river. Sand, gravel, and boulders occurred in the stream channel. The upper walls of the canyon in the vicinity of the dam have been glaciated, but no signs of glaciation were noted below elevation 3300.

14. Reservoir. - Hungry Horse Reservoir covers about 22,500 acres at its maximum elevation, extending up the South Fork a distance of 34 miles. The reservoir is only 0.4 to 0.8 mile wide near the dam and widens to a maximum width of 2 miles at a distance of about 4 miles above the dam.

The river carries no silt for 9 months of the year, but during high water a small silt load is carried. Silting of the reservoir does not present a problem. The lower part of the reservoir is covered with the upper part of the Belt Series known as the Missoula Group. Tertiary beds cover much of the floor of the reservoir. Glacial deposits are present in many places in the reservoir area and in the tributaries of the South Fork.

(a) *Leakage from the Reservoir.*— Geological Survey Water Supply Paper 866-B, by Charles E. Erdmann, indicated that a buried preglacial channel might exist in what is known as the Abbott Creek area located 2-1/4 miles northeast of the dam. Topographically, this valley appears to be a former course of the South Fork of the Flathead River or of Hungry Horse Creek. There was also some evidence that a branch channel called the Lion Hill Gorge intersected the reservoir about 2 miles above the dam site. (See fig. 5).

Five washbore and diamond drill holes totaling 1,317 feet were bored and two test pits 46 and 50 feet deep were dug in 1945. These investigations seemed to substantiate the belief that the channels existed. The area north of these holes and the Lion Hill Gorge area were then investigated with a seismograph. Rock ridges were located across the gorges and subsequent drilling located bedrock only 15 to 20 feet below maximum reservoir water surface elevation 3560, which confirmed the seismic findings. Water tests showed the overburden material to be quite impermeable. From this information it was concluded that, if a channel runs down the valley, it would be very narrow and leakage from storage in the reservoir at maximum water surface would be very minor or nonexistent.

15. Choice of Site. - Several sites were considered for the dam along a 2-mile stretch of the river ranging from mile 3.5 to mile 5.5 measured upstream from the mouth of the South Fork River. The Geological Survey made various investigations from 1921 to 1936, and contour maps of the river were made to elevation 3540.

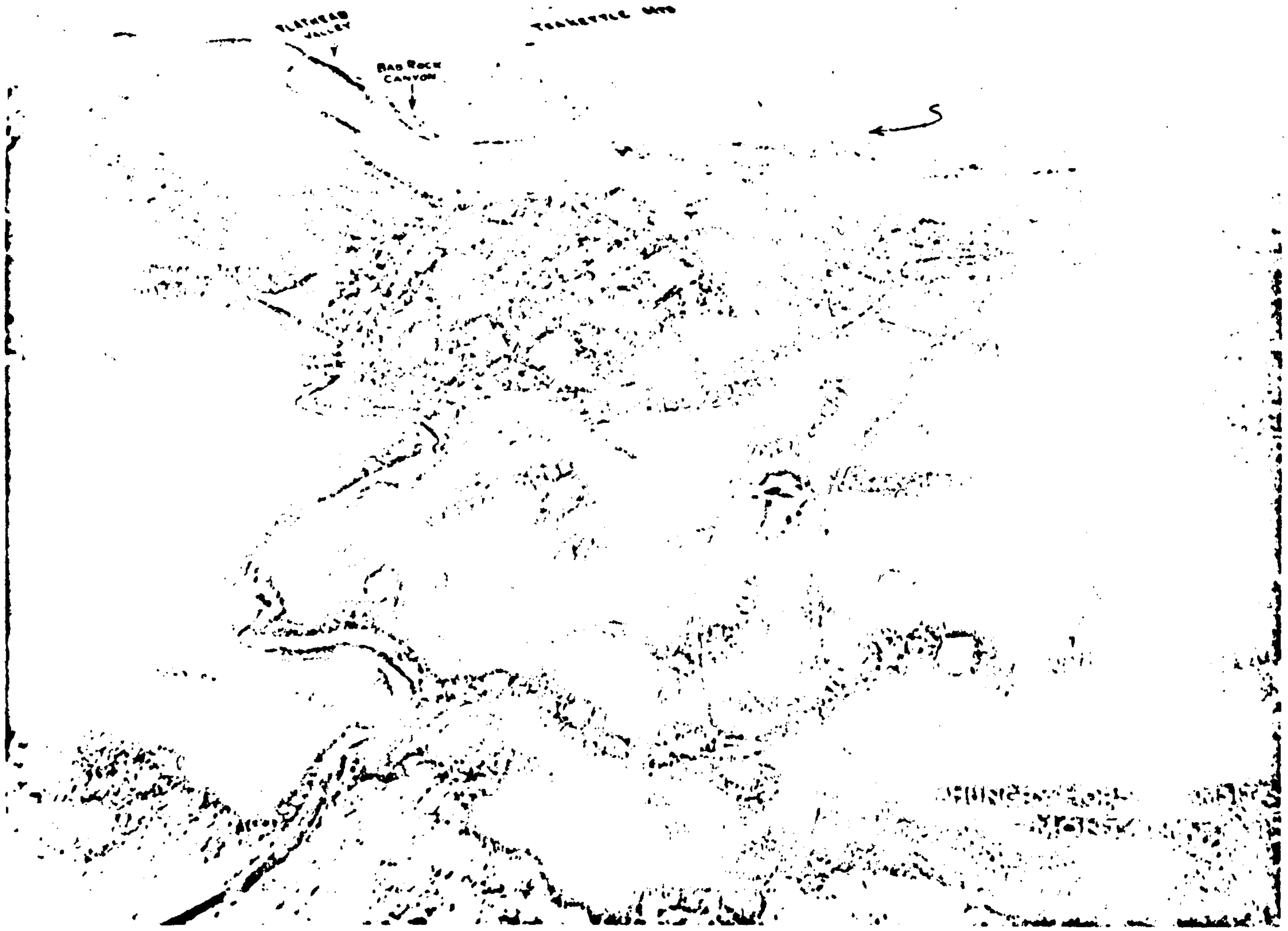


Figure 5.--Aerial view of the South Fork of the Flathead River canyon looking downstream, showing respective positions of the dam and reservoir, and outline of Lion Hill and Abbott Gorges. P447-105-3022, October 3, 1950.

In 1939 the Corps of Engineers drilled 11 diamond drill holes at mile 4.1. In 1944 the Bureau of Reclamation was designated to build Hungry Horse project. Several axis lines were suggested by Geologists C. E. Erdmann of the Geological Survey and F. D. Jones of the Bureau of Reclamation. The profile at mile 5.2, designated as axis CC', offered the smallest cross section and appeared to be the best axis. The locations and profiles of the various axes studies are shown on figure 6.

16. Foundation Conditions. - Early in 1946 drilling began at axis CC' for a straight gravity dam 420 feet high, or to elevation 3490. These explorations disclosed that the rock was capable of supporting a concrete arch-gravity dam with a flow line at elevation 3560 which would provide a reservoir with a capactiy of 3,500,000 acre-feet.

Forty-nine drill holes, 36 vertical and 13 inclined, ranging in length from 39 to 294 feet, were completed for a total footage of 7,845 feet. Four exploratory tunnels, ranging in length from 109 to 213 feet, were driven in the canyon abutments, and some sluicing was done to expose a fault zone.

17. Construction Geology. - Excavation for the dam commenced in June 1948 and was completed during the summer of 1949. The powerhouse excavation, including that for the tailrace and outlet works, was completed about June 1, 1950. (See chap. XIII.) The foundation area was laid out in 28 blocks, each 80 feet wide. As the foundation areas were uncovered the geology was plotted on a scale of 1 inch = 20 feet, covering five 21- by 36-inch sheets.

In general, the rock was found to be a hard, strongly bedded series of limestone strata with individual beds which varied in thickness from an inch or two up to 2 or 3 feet. During mountain building these beds were elevated and tilted, then eroded to their present form. The river generally followed along the strike of the strata, eroding a comparatively narrow V-shaped canyon in which the dam was placed.

Since the bedding of the rock dips in one direction, the strata of the right abutment dip rather steeply into the valley side, while on the left abutment they dip toward the river. The stripped foundation followed rather closely the parting planes of the beds on the left abutment, while in the bottom and up the right abutment the exposed surface cut across the beds one after the other. Several structural weaknesses were discovered in the foundation rock during excavation, consisting of open bedding planes or lift seams and additional small fault zones.

(a) *Lift Seam.*-- One lift seam or open bedding plane was located on the left abutment extending from block 8 to block 13. It was caused by the competent limestone beds being sprung apart or by their shearing or sliding over each other during mountain building and folding. This seam occurred from 11 to 19 feet below the rock surface between fault zones No. 3 and 4 and from 5 to 15 feet below the surface between fault zones No. 4 and 5. The seam was filled with clay which varied from 1/16 inch to 4 inches in thickness.

18. Faults. - Figure 7 shows the location of faults on an areal map of the dam. In the original exploration for the dam, only one fault, later called fault zone No. 6, was discovered and thoroughly explored by diamond drill cores. During stripping of the foundation six other faults were uncovered. Of these faults, No. 3 in blocks 7 and 8, No. 4 in blocks 9 to 11, and No. 6 in blocks 15 to 18 were large enough to require cutoff shafts and trenching of the zones where they crossed the foundation. Fault zones No. 1 in block 2, No. 2 in block 5, and No. 5 in block 12 were minor faults and required only trenching of the zone. There was no evidence of recent movement along any of these faults. The excavation and grouting of the faults is summarized under the chapter on grouting. A brief description of the faults as disclosed during investigation and later treatment follows:

(a) *Fault Zone No. 1.*-- This is a minor fault zone high on the left abutment partially in block 2. It consists of a shear zone of crushed rock 8 to 10 feet wide at the dam axis and 1.5 feet wide at the toe, with gouge seams up to 1 inch thick along the hanging and foot walls of the zone. There is little weathering along the zone. It strikes N 39° W, dips 50° SW, and the hanging wall is displaced down.

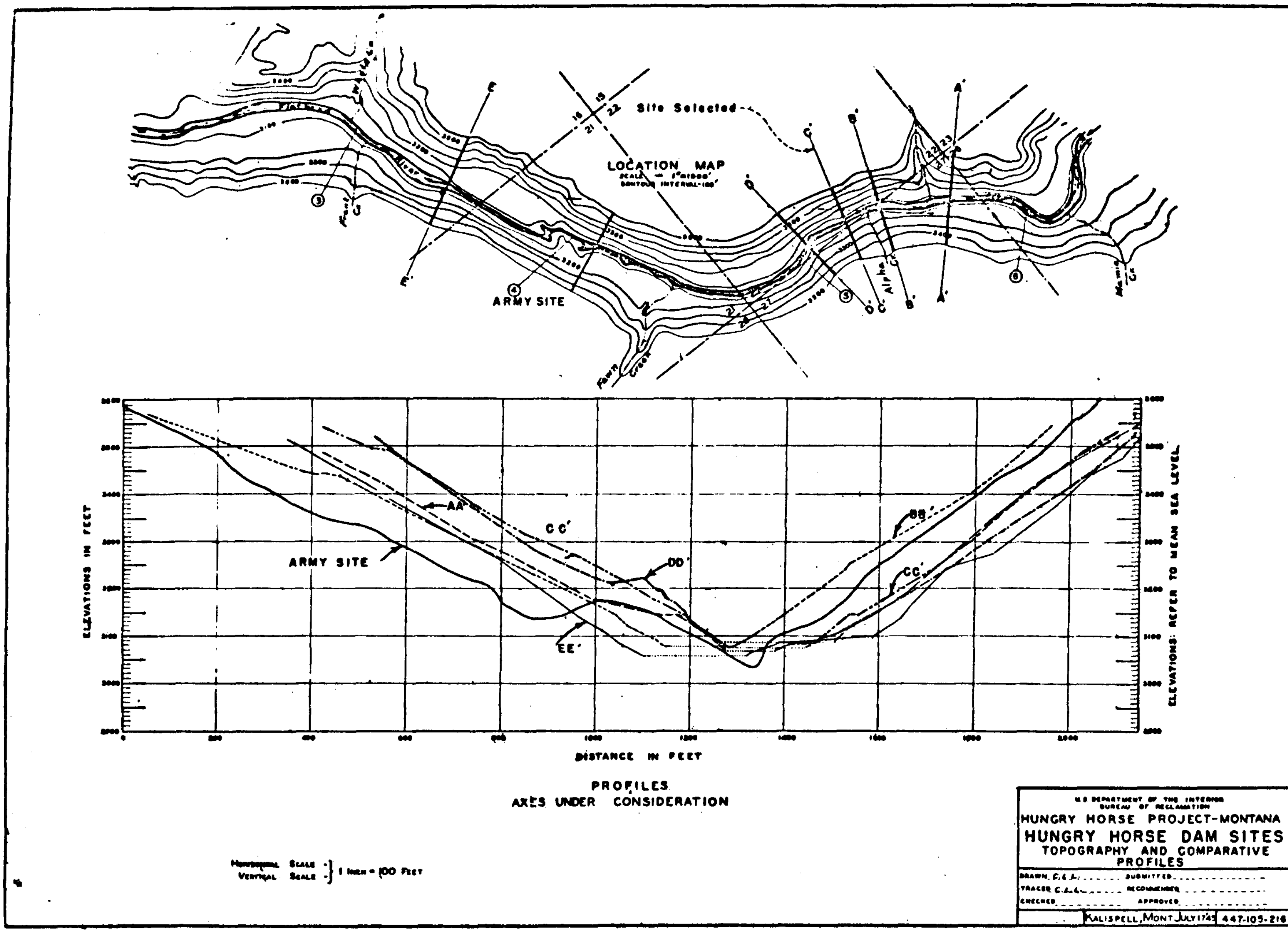


Figure 6.--Topography and comparative profiles of proposed sites.

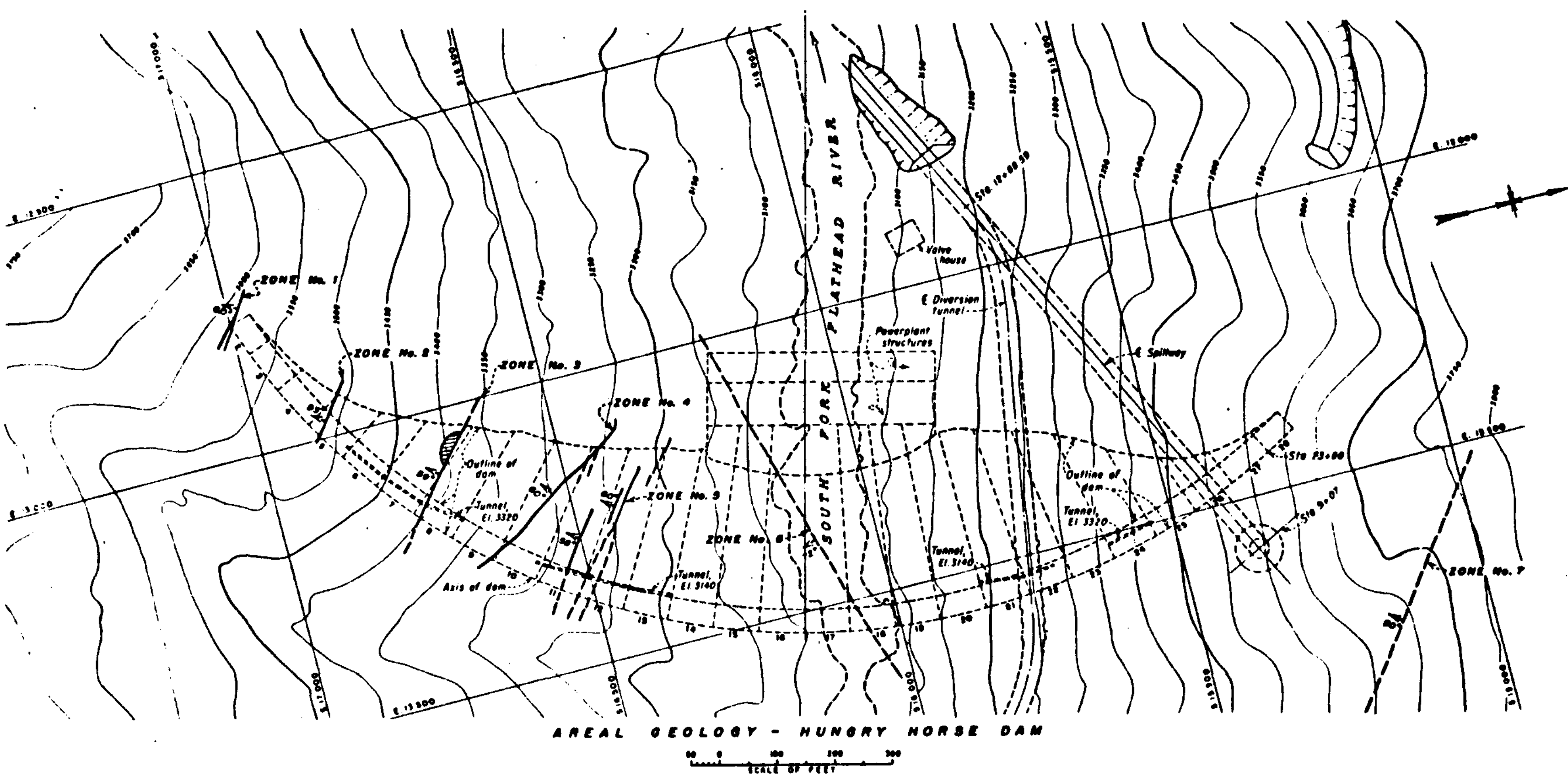


Figure 7.--Areal geology of the selected dam site.

(b) *Fault Zone No. 2.*-- This zone is a minor feature consisting of a wavy slip extending diagonally across the left half of block 5. It is approximately 3 inches wide and is filled with clay and crushed rock. The hanging wall is displaced 1.1 feet down.

(c) *Fault Zone No. 3.*-- This zone crosses the boundary line between blocks 7 and 8, and is one of the major fault zones at the dam site. It strikes N 50° W and dips 56° SW into the left abutment with the hanging wall displaced 8 to 12 feet down. The fault is about 9 feet wide at the axis, decreasing to 4 feet wide about 45 feet downstream from the axis then increasing to 45 feet wide at the downstream toe. The rock is intensely crushed with much gouge in the narrow zones and only slightly sheared in the wide zones. Most of the material in the fault is tight and fresh, with only small areas of slight weathering.

(d) *Fault Zone No. 4.*-- This zone extends diagonally across block 10. It strikes N 26° W and dips 56° SW or into the left abutment. The zone is only 8 to 15 inches wide at the axis of the dam and increases to 42 inches at the toe of the dam. The gouge is moderately weathered to fresh in the exposure of the fault as it crosses the dam.

(e) *Fault Zone No. 5.*-- This zone trends in a southwesterly direction across the foundation of the dam. It extends across blocks 11 and 12 and intersects fault zone No. 4 at the downstream toe of the dam. The fault is a narrow zone from 6 inches to 2 feet in width containing some crushed rock, clay, and small deposits of calcite. At the axis the hanging wall shows a displacement of 11 feet, which decreases downstream where the movement is dissipated along many minor slips and shear zones.

(f) *Fault Zone No. 6.*-- This fault zone is the largest encountered. It cuts diagonally across the river channel from the upstream edge of the dam at block 18 and extends to the downstream edge of the dam at block 15. It strikes N 71° to 75° E and dips 73° to 76° SE towards the left abutment and upstream. The fault zone in the river bottom varies from 16 to 23 feet in width. A gouge layer varying from 2 inches to 7 feet in thickness is present in the fault, and the remainder of the rock in the fault zone is slightly sheared and crushed. The main gouge layer occurs along the hanging wall, and smaller gouge zones occur along the foot wall. Most of the gouge and sheared rock is fresh and unweathered. In places where joints or seams intersect the fault there is a small amount of moderate weathering. Figures 8 and 9 show fault zone No. 6 after the overburden had been removed.

(g) *Fault Zone No. 7.*-- A zone of intensely sheared and broken rock was located during the excavation for the tailtower trackway on the right abutment. The indicated trend of the zone is N 40° E with a dip of about 50° SW or towards the dam. The zone consists of intensely sheared and broken rock. Since this fault is well outside the excavation area, it was believed the fault would not affect the dam.

19. Minor Defects. - On the right abutment two bedding plane slips were found to be more prominent then any of the others. Several other joints and small lenses and shear zones were found. None of these required special treatment outside of excavating to shallow depths of 1 or 2 feet below the general level of the rock and washing with high-pressure air and water.

20. Powerhouse. - The rock in the powerhouse area was found to be of good quality. Fault zone No. 6 is located under the control bay, and the quality of the rock decreases slightly near the fault. Two minor areas of crushed rock were found in the foundation for generating unit 3. A minor bedding plane slip with 1/4 inch to 3 inches of fresh gouge along it cuts across unit 2.

All the rock in the powerhouse foundation is extensively cut by joints. Some movement has occurred along a few of the joints, resulting in the production of crushed rock and clay seams up to 4 inches in thickness. In the main, the joints are fresh and show little weathering.

A mass concrete retaining wall was placed under the control bay to support the rock. This was necessary because numerous joints, the nearness of fault zone No. 6, and the undercut bedding in the generator foundation produced a condition where a rock slide could move from under the control bay down the bedding and into unit 4.

All the rock under the outlet tubes and the valve house is of good quality. It is cut by many northwest striking joints and slips, some of which have a thin seam of gouge along them.

21. Foundation Tunnels. - Four foundation tunnels were excavated in the abutments of the dam. Two are located on the left abutment at elevations 3140 and 3320 and two on the right abutment at the same elevations.

(a) *Left Abutment Tunnel at Elevation 3320.*-- The portal of this tunnel is formed by the hanging wall of fault zone No. 3. The tunnel is 262 feet long. The rock in the tunnel is of good quality but is extensively jointed. Six minor bedding plane slips containing 1/8 to 2-1/2 inches of moderately crushed rock and damp clay were crossed in the tunnel.

(b) *Tunnel at Elevation 3140.*-- This tunnel is 150 feet long. Fault zone No. 5 is cut by the tunnel near the portal. This fault zone splits as it crosses the foundation and its displacement is dissipated over a number of slips which strike to the northwest. Several of these slips can be seen in this tunnel.

(c) *Tunnel at Elevation 3320.*-- This tunnel is 47 feet long. Numerous joints with fresh surfaces which strike N 70° E with an average spacing of 4 inches were cut by this tunnel. Movement has taken place along a few seams which parallel these joints, and the seams have from 1/16 to 1 inch of soft sticky clay along them with the sidewalls weathered to depths of 6 inches.

(d) *Right Abutment Tunnel at Elevation 3140.*-- This tunnel is 114 feet long. The main feature of the rock in this tunnel is the prominent, closely spaced joints which strike N 69° to 76° E and dip 68° to 75° SE. These joints are spaced 1 to 2 inches apart and have fresh surfaces except for minor amounts of limonite staining. From station 0+15 to station 0+90, the left abutment is made by joints and minor slips in a set which strikes N 3° E and dips 58° to 61° NW. Three bedding plane slips with 1/16 inch to 5 inches of crushed rock and clay along them were intersected in the tunnel.

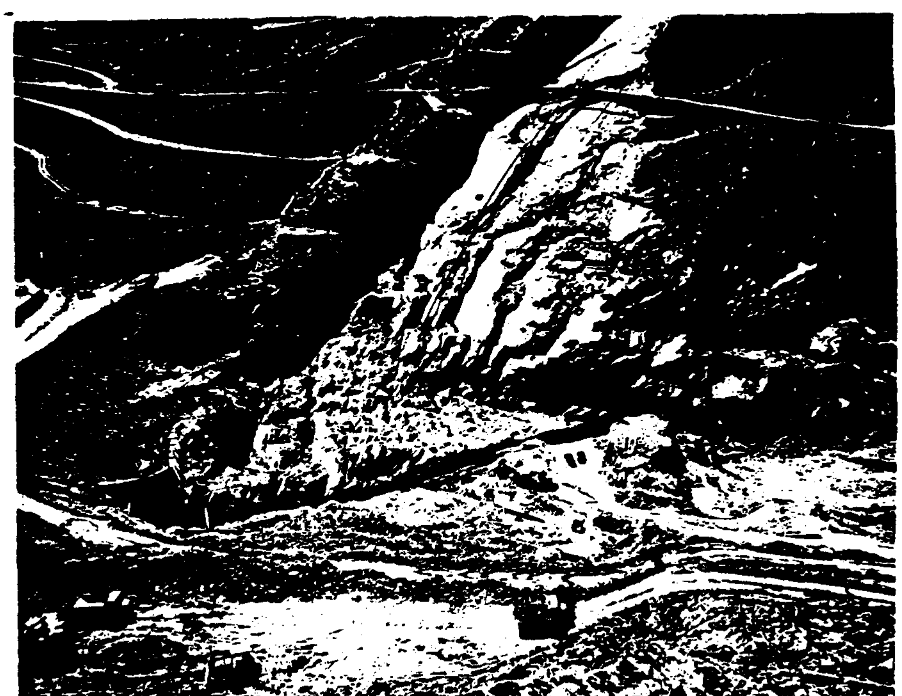


Figure 8.--Completed left abutment excavation and early stages of excavation for fault No. 6, as viewed from the right abutment at elevation 3400. The diagonal trench in river channel marks fault No. 6. P447-105-1631, August 26, 1949.

22. Spillway Tunnel. - The rock in the morning-glory spillway tunnel was found to range from good to very good in quality. Numerous joints and some slips were crossed, and from station 5+07 to station 6+00 the rock was very blocky.

In the roof and floor of the inclined section of the tunnel, the rock generally broke to the joints which struck to the northwest and dipped to the southwest at approximately the same angle as the tunnel. In the sidewalls of the tunnel the rock broke mainly to the joints that struck N 70° E and dipped 70° SE.

In the horizontal section the rock was of very good quality and breakage closely followed the drill holes.

Three bedding plane slips which varied from 4 to 15 inches in thickness were intersected in the inclined portion of the tunnel. The material along these slips ranged from clay to crushed rock, and it was generally fresh and tight. Fourteen other small bedding plane slips ranging in thickness from a film of clay to 3 inches were intersected in the tunnel.

23. Diversion Tunnel. - Rock encountered in the tunnel varied from faulted and broken to very good in quality. Fault zone No. 6 crossed the tunnel in the area bounded by stations 2+30 and 2+56, and several unsuccessful attempts were made to establish a portal in this area. The proposed portal was then moved to station 2+56 and a pioneer tunnel bore was driven from station 3+03 by following just below the roof of the tunnel. The intake to the tunnel was later excavated to full size without supporting the rock.

From near station 2+70 to station 11+00 the rock was very good in quality, with numerous joints and slips which were generally tight. From station 11+00 to the outlet portal at station 13+99 the quality of the rock decreased from very good to poor. A number of bedding plane slips as well as other slips with moderately weathered gouge seams up to 3 inches thick were present in this section of the tunnel. Between stations 12+00 to 12+50 these seams were prominent, especially in the roof, and at places some overbreak occurred.

CHAPTER III--GROUTING

24. Basic Grouting and Drainage Plan. - The general plan for grouting the foundation rock beneath the dam provided for preliminary low-pressure shallow grouting which would be followed by intermediate and final high-pressure deep grouting, as shown on figure 10. The shallow holes for low-pressure grouting to consolidate the near-surface zone of rock were known as "B" holes. The holes along the upstream edge of the dam, which formed an auxiliary cutoff curtain, were known as "C" holes and were grouted under intermediate pressure. The main cutoff curtain beneath the dam was formed by drilling deep holes from the grouting and drainage gallery and grouting them under high pressure. These were known as "A" holes. In addition to the above grouting, the rock surrounding the spillway tunnel, the diversion tunnel-plug section, and the intake structure of the diversion tunnel was grouted. Also, the original plans provided for drilling and grouting holes in the area to be covered by the downstream portion of the dam and powerhouse foundations to seal the major surface seams, crevices, and fractures. Drilling and grouting were performed under specifications No. 2122 (for the prime contract).

25. Fault Zone Treatment. - After the excavation for the dam had been started, several defective areas were found in the foundation rock. The foundation was cut by one major and several minor shear zones and by one major bedding plane slip. A total of seven fault zones were uncovered, and most of these required special treatment. Five of the fault zones occurred between the bottom of the river channel and the top of the left abutment and one occurred in the right abutment area beyond the end of the dam. The largest fault zone, No. 6, cut diagonally across the river channel in the bottom of the canyon. These fault zones are described in chapter II.

Because of the serious nature of some of these faults, a Board of Consultants was called to review the conditions and to make recommendations for remedial treatment. The criterion set forth by the Board of Consultants for the treatment of the faults was to construct cutoffs which would provide a percolation distance along the contact surface of the concrete and the crushed or sheeted material equal to not less than two and one-half nor more than four times the hydraulic head between the maximum water surface and the minimum tailwater surface or downstream toe of the dam. The treatment of the individual fault zones was therefore dependent on the field conditions encountered.

The treatment of each fault zone was as follows:

(a) *Fault Zone No. 1.*-- This minor fault zone was treated by drilling several holes to intersect the fault below the surface and washing with water and compressed air before grouting. The material in the fault proved tight and accepted practically no grout.

(b) *Fault Zone No. 2.*-- A total of six holes were drilled to intersect this minor zone. After washing with water and compressed air an attempt was made to grout it, but the zone proved tight.

(c) *Fault Zone No. 3.*-- In compliance with the criterion set forth by the Consulting Board, two cutoffs were constructed in this fault zone, which was one of the major faults at the site. The dimensions of the shaft for the upstream cutoff ranged from 7-1/2 by 8 feet to 7-1/2 by 12 feet in cross section and the depth was 130 feet. The downstream shaft varied from 7-1/2 by 7-1/2 feet to 7-1/2 by 10 feet and was 66 feet deep. The surface of the zone was trenched to a depth ranging from 5 to 12 feet below the downhill edge of the fault zone exposure. The excavated fault zone and cutoff shafts are shown in figure 11.

Prior to placing concrete, "B" holes were drilled adjacent to the rock-cement contact along the fault zone at approximately 10-foot centers. Holes were drilled to depths of 30 feet between the cutoff shafts and to depths of 20 feet downstream from the downstream cutoff shafts. This required drilling 600 linear feet of grout holes into which 212 cubic feet of grout were injected for an average take of 0.35 cubic foot per linear foot of hole.

From the bottom of the upstream shaft, six grout holes each 150 feet in length were drilled into the rock below the shaft, covering a fan-shaped area. These holes were

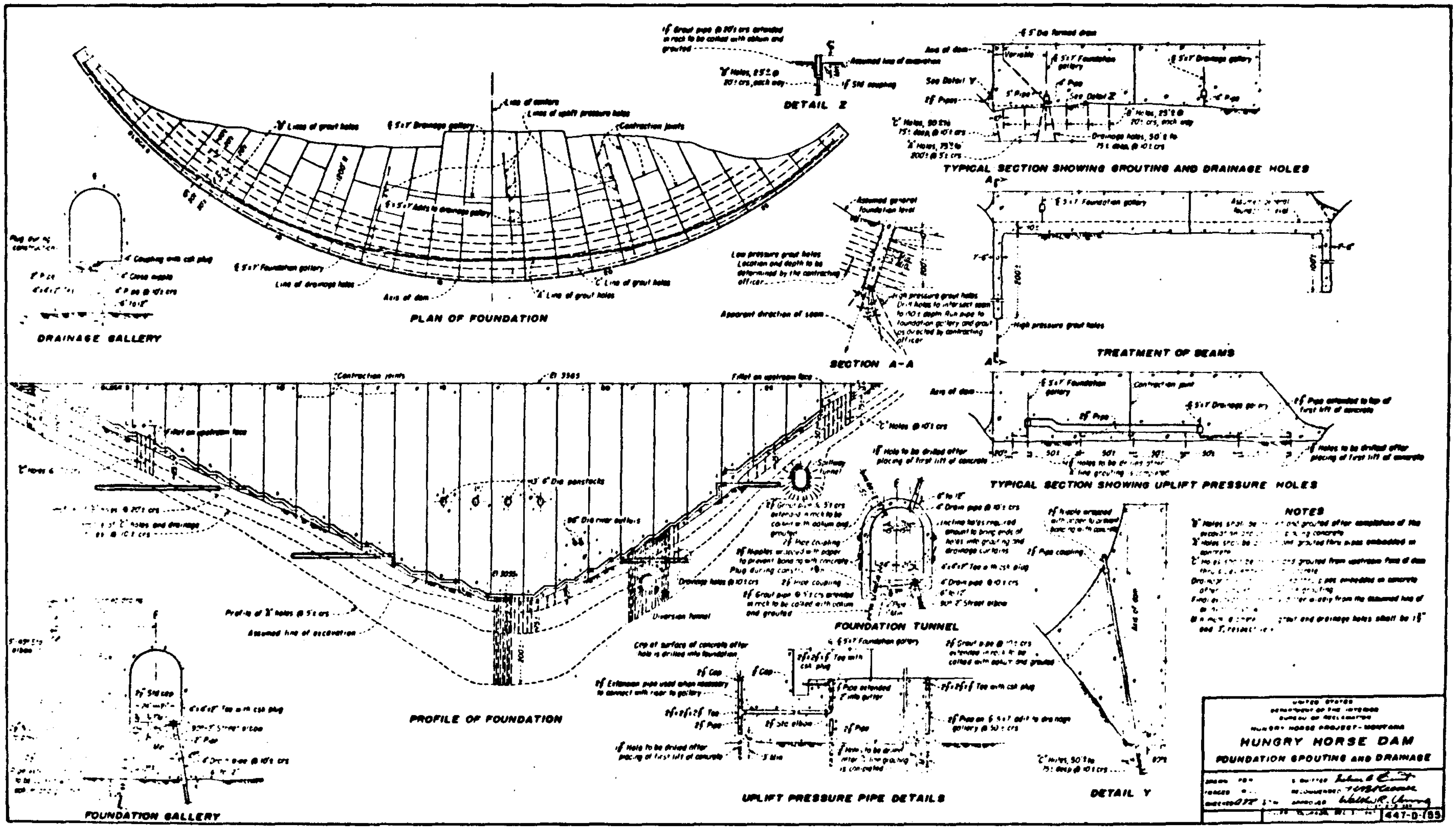


Figure 10.--Foundation grouting and drainage system.

Hungry Horse Dam

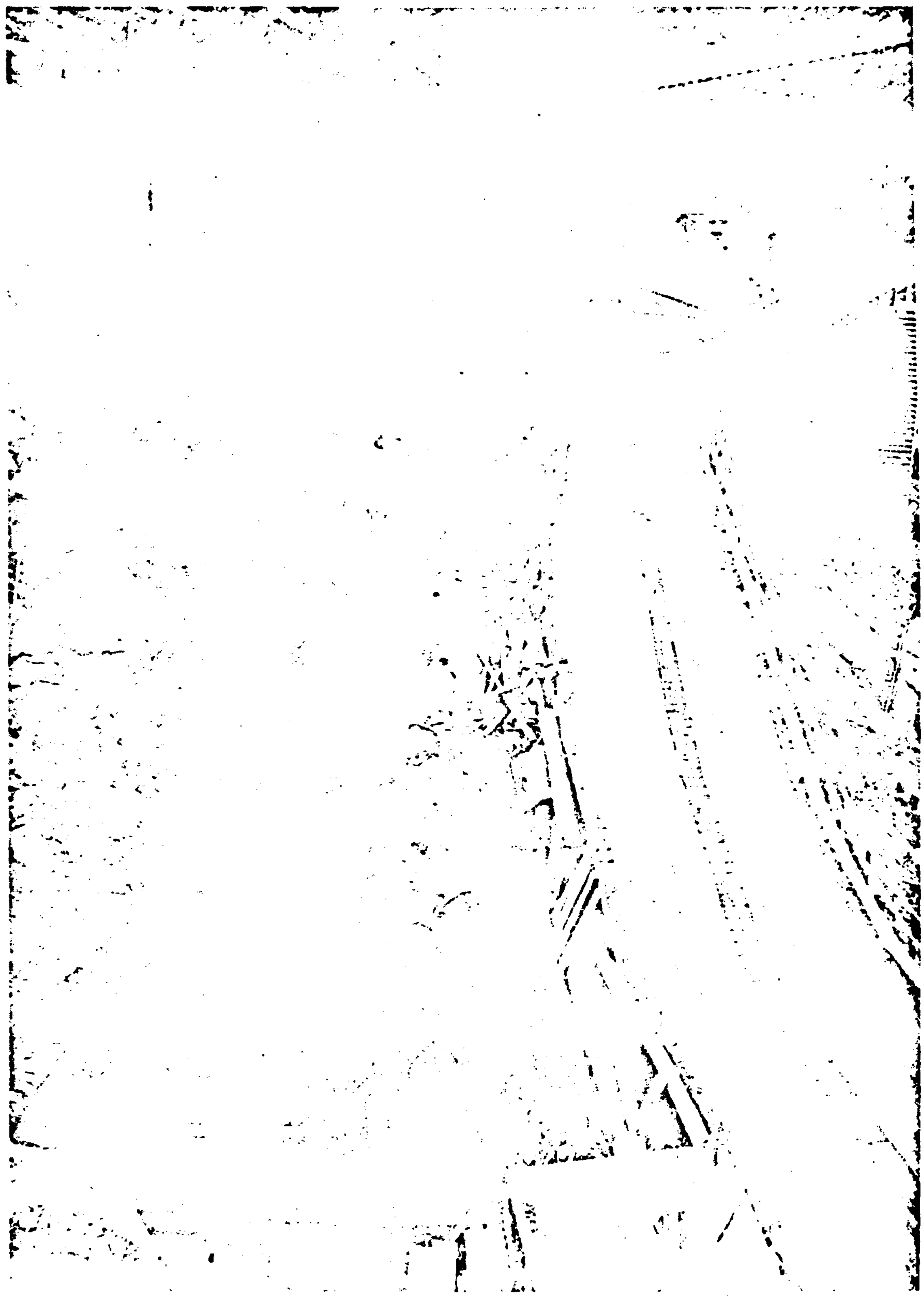


Figure 11.--Excavation of trench between cutoff shafts in fault zone No. 3. Upstream shaft in fault is located at left of square concrete chute at bottom of photograph, and downstream shaft is at left of clamshell bucket in background. P447-105-3043, October 12, 1950.

grouted before the shaft was backfilled with concrete. A total of 66 cubic feet of cement was injected at maximum pressures of 175 to 225 pounds per square inch. The hanging and foot walls were grouted by drilling holes 30 feet deep and spacing them 20 feet apart vertically up the shaft. Twelve holes in the walls took only 31 cubic feet of cement at maximum pressures ranging from 75 to 175 pounds per square inch.

In the downstream shaft 10 holes were drilled to depths of 30 feet in the side walls and bottom. The rock in this area was tight, as only 4 cubic feet of grout could be injected at the maximum allowable pressures which ranged from 100 to 150 pounds per square inch.

After the cutoff shafts had been backfilled with concrete, the total length of the path of percolation along the contact of the rock and concrete was approximately 550 feet, or only 25 feet short of that required for the two and one-half factor requested by the Consulting Board.

(d) *Fault Zone No. 4.*— This fault zone was considered large enough to warrant the use of cutoff shafts, and two were excavated in this zone. The upstream shaft varied from 7-1/2 by 12 feet to 7-1/2 by 8 feet and was 75 feet deep. The downstream shaft was about the same size and was excavated to a depth of 46 feet.

"B" holes were drilled on both sides of the fault zone between the shafts and in block 10 in the upstream part of the block. These holes were 20 to 30 feet deep and were grouted at low pressure. Six hundred and thirty linear feet of grout hole were drilled into which 573 cubic feet of grout were injected.

Five 150-foot-deep grout holes were drilled in the bottom of the upstream cut-off shaft and fanned out across the fault zone. These holes required 294 cubic feet of grout. The maximum pressures ranged from 100 to 400 pounds per square inch, and the water-cement ratio of the grout was 8:1 by volume. Two-hundred and fifty-two cubic feet of this grout was injected into the bottom 50 feet of the holes. Grout holes 30 feet deep were drilled into the walls of the shaft at vertical intervals of 20 feet. These holes took only 9 cubic feet of grout at maximum pressures of 50 to 100 pounds per square inch and a water-cement ratio of 8:1.

Five holes drilled to depths of 30 feet were fanned out across the fault from the bottom of the shaft and three holes 30 feet deep were drilled into the side walls. These holes took 26 cubic feet of 8:1 grout at pressures of 50 to 200 pounds per square inch.

(e) *Fault Zone No. 5.*— The regular pattern of "B" holes was augmented in this minor fault zone by drilling extra rows of holes on both sides of the fault. Effort was made to wash as much clay and gouge from these holes as possible in the areas along the fault zone so that this material could be replaced with grout.

(f) *Fault Zone No. 6.*— This was the largest fault zone and required the most extensive treatment. It was excavated to a width of 18 to 22 feet across the foundation and to a depth of approximately 30 feet below the surrounding rock, and cutoff shafts were put down in the fault at the axis and at the downstream toe of the dam in compliance with the criterion of the Board of Consultants. The upstream shaft was excavated to a slope depth of 208 feet and the downstream shaft to a slope depth of 95 feet. The upstream shaft varied from 7-1/2 by 17 feet to 7-1/2 by 22 feet in size. The downstream shaft varied from 7-1/2 by 18 feet to 7-1/2 by 40 feet in size. Part of the excavated fault zone can be seen in figure 12.

Grout holes 30 feet deep were drilled and grouted in the foot and hanging walls of the zone and fanned out from the bottom of the upstream shaft. The holes in the side walls were spaced at 20 feet vertically up the shaft, and 22 of these holes took a total of 34 cubic feet of cement. Seven holes 30 feet deep were fanned out from the bottom of the shaft and these holes took a total of 188 cubic feet of cement. One-hundred and fifty-two cubic feet of cement was injected in one hole which was drilled down along the hanging wall and parallel to the fault. After the grout in the 30-foot-deep holes had set, five of them were washed out and drilled to depths of 150 feet. These five holes were grouted along with the "A" holes and took 83 cubic feet of grout.

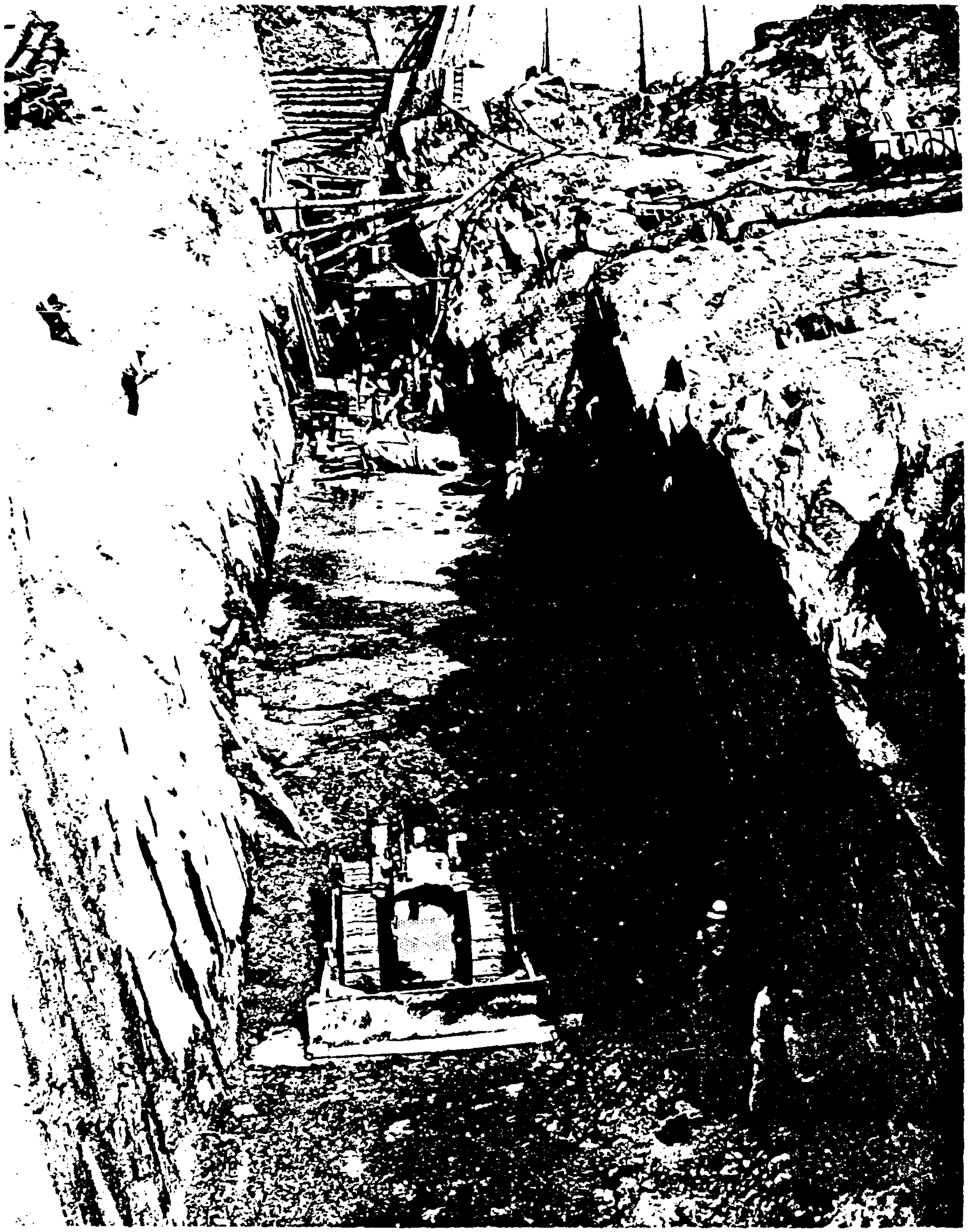


Figure 12.--Excavated channel section in fault zone No. 6, viewed from formwork for downstream shaft. The concrete structure at top of photograph forms a pump sump. P447-105-1664, September 2, 1949.

At the downstream shaft, the rock in the hanging and foot walls was grouted by drilling holes 30 feet in depth, spaced at 20-foot vertical intervals up the shaft, and a group of holes were drilled and grouted at maximum pressures up to 180 pounds per square inch. A total of 23 cubic feet of grout was injected into this rock.

At the completion of this treatment of fault zone No. 6, the path of percolation along the concrete rock contact was approximately 1,095 feet compared to 1,229 feet necessary for a percolation factor of two and one-half. This distance was considered adequate in view of the good quality and freshness of the rock in the fault zone, the narrow width of the gouge seam between elevations 2930 and 2860 in the upstream shaft, and the absence of gouge over the 40-foot reach of the fault near the center of the foundation.

(g) *Fault Zone No. 7.*-- Because this zone occurred some distance beyond the end of the dam, it was felt that no special treatment would be necessary.

26. Lift Seam Treatment. - This seam (sec. 17(a)) was exposed early during the 1949 construction season and was a subject for consideration by the Consulting Board during May of that year. The problem was to decide whether to wash the clay from the seam and then grout the seam or to excavate the sound rock above the seam and the underlying layer of clay. The volume of rock involved was about 7,100 cubic yards, which would have to be replaced with concrete. The Consulting Board recommended that the seam be more extensively explored by means of 6-inch core-drilled holes to determine the extent, thickness, and character of the material along the slip. If from these data it was determined that the seam could be washed satisfactorily and grouted, the overlying rock should be left in place; and the seam would then be washed and grouted through drill holes drilled on a hexagonal pattern with a maximum spacing of 5 feet. If the exploratory drilling indicated the seam could not be grouted, the rock above the seam should be removed.

The initial attempt at washing the clay seam was done in block 13. In this attempt a plan and procedure was followed similar to that employed at Norris Dam by the Tennessee Valley Authority, which required that drilling, washing, and grouting be repeated a number of times in the same pattern of holes in order to wash clay from all the seams.

At Hungry Horse Dam the grout holes for washing were to be drilled on 5-foot centers in the area of the uplift seam. The first drilling, washing, and grouting were done in patterns of 9 to 25 holes, 10 feet apart in rows 5 feet apart, the drilled holes being staggered. After these holes were washed and grouted, the remaining holes in the pattern were drilled, washed, and grouted. This process was repeated in other patterns until an area was completely washed and grouted (fig. 13).

Washing was done by injecting water and air into the holes at pressures not exceeding 30 pounds per square inch. First the water was injected and channels were washed through the seam to other drilled holes. Following the opening of the channels along the seam, the water was blown out by the injection of air at about one-half of the water pressure, the air causing a turbulence that tended to dislodge and remove additional clay. Water and air were then injected and vented from adjacent holes until no clay was ejected (fig. 14). The process was continued until all holes had been used as inlets and also as vents, and the effluent at the vents was clean. This was followed by grouting from the lowest hole, venting excess water, and then grouting from return holes. The grouting was continued until all holes had returns of heavy grout or were all hooked to and pumped. Usually several vent holes were open at slightly higher elevations than the grouted holes. When grout returned from these holes the grouting was stopped.

At the completion of grouting in block 13, a 36-inch-diameter calyx hole was drilled to observe the results of the washing and the condition of the lift seam. Observation indicated that about one-half of the seam was washed free of clay and grouted while the remaining one-half still had clay in place.

Drilling, washing, and grouting of the clay seam progressed to block 12 with more emphasis being placed on the use of air in cleaning out the seam. The procedure

Figure 13.--Clay seam treatment of rock in blocks 11 (foreground) and 10 (background). Nipples are set in holes which have been drilled for washing and grouting of clay seams. P447-105-3083, October 24, 1950.

Figure 14.--Clay seam treatment in left abutment. Water which has been forced into the rock under high pressure spurts from drill holes like miniature geysers. P447-105-2956, September 16, 1950.

of pattern drilling, washing, and grouting was continued. After completion of this area and the adjacent portion of block 11, another 36-inch calyx hole was drilled in block 12 to check the results of the washing. Grout was found all around the perimeter of the hole, but in laminations with thin layers and lenses of a light tan clay. The reddish-brown clay had apparently been washed out and replaced by grout.

It appeared that another method was needed to erode the light tan clay, which was more resistant to the water and air washing. Several methods were attempted such as trying to force a rapidly rotating thin steel cable into the seam; using a vacuum pump on the holes alternated with injections of air and water; attempting to force a thin flexible tubing into the seam with high-pressure water; and using a side jet with high-pressure water. This last method was the only one that appeared to dislodge additional clay when supplied with water at 250 pounds per square inch. This method was therefore continued up the abutment as far as fault zone No. 3. Above the fault, the foundation rock was excavated sufficiently deep to reach the seam.

Several calyx holes were put down to check the effectiveness of the washing and grouting program. The estimated replacement of clay by grout in the portion of the seams exposed in these holes ranged from 70 to 80 percent.

Most of the holes for the seam washing program ranged between 8 and 20 feet in depth. The total amount of drilling required for this feature of the work was 31,282 linear feet. The amount of cement injected into the lift seam after washing was 8,211 cubic feet.

27. Field Grouting Program Summary. - Foundation drilling and grouting operations at Hungry Horse Dam started August 25, 1949, with the drilling of "B" holes in block 12 and ended with the completion of the "A" line holes in May 1953. After the clay-filled lift seam was encountered in June 1950, the special treatment for washing the clay from the seam started and continued through June 1951. The "C" line grouting started in August 1950 and was finished in September 1952. The "A" grouting was started in September 1951 and was finished in May 1953.

The drilling procedure for the "B" zone grouting in general was to drill holes to their final depth and then grout progressively from the lower portion of the hole upward with a packer, the top portion of the hole being grouted through a direct connection to the pipe nipple which formed the collar of the hole. The grouting pressures were low, a maximum of 90 pounds per square inch being used with packers, and lower pressures being used with a direct connection. At the start of the grouting program the pressures used generally varied between 40 and 20 pounds per square inch. Low pressures were used to avoid any possible uplift of the foundation rock to the left of fault zone No. 6, because the bedding planes were parallel to the excavated surface.

The regular "B" zone grouting continued up to June 1950 when it was expanded into the program of seam washing and grouting described in section 25. This program was the result of recommendations contained in a report dated September 7, 1949, to the Chief Engineer by Mr. J. L. Savage, Consulting Engineer. The regular "B" zone grouting was continued intermittently with other grouting work until July 1952.

The auxiliary cutoff curtain, which was formed by grouting the line of "C" holes, was started in August 1950 and completed in September 1952. This grouting was done using pipe embedded in the concrete of the fillet at the upstream face for the collars of the grout holes. The lower end of the pipe had been anchored in the foundation rock at the time of installation. Drilling was done through the pipe using diamond drills mounted on jack bars clamped to the upper end of the pipe. Before grouting of the "C" holes, it was required that all concrete within a radius of 100 feet be placed to a minimum height of 25 feet above the foundation rock. It was also required that water be circulated through contraction joints in the bedrock lifts to avoid losing any of the joint grouting system due to grout traveling up into the dam through cracks in the concrete or defective metal seals. Maximum pressures used in "C" hole grouting were 150 pounds per square inch when connected directly to the collar of the hole, 200 pounds per square inch when grouting below a packer set at 25-foot depth, and 250 pounds per square inch when grouting with a packer set at 50-foot depth.

The "C" holes in block 18 and the right abutment area were drilled to full depth and stage grouted by use of a packer. The method of closures was used in grouting these holes. The "C" holes in block 17 and the left abutment were drilled by stages in groups of three adjacent holes. Washing of clay from the bedding plane slips was attempted before grouting. A modified method of closures was used in spacing the groups of holes for drilling. When washing was possible, it consisted of the introduction of water at grouting pressure into one hole and venting the clay-bearing water through the other two open holes, the process being rotated through all three open holes. Thin clay seams were encountered at some places in most holes, but only in a few cases were the clay seams continuous through the three holes. Thirty-one groups of holes were drilled in stages and attempts made in washing, but only in five groups of holes was success obtained in washing clay from the seams.

The drilling and grouting of the main cutoff curtain, consisting of the line of "A" holes, was started on September 25, 1952. The first holes grouted were the deep holes extending into the foundation gallery from the bottom of the upstream cutoff shaft in fault zone No. 6. These holes had been drilled in March 1950 to a depth of 30 feet and grouted for the first stage followed by drilling to 150-foot depth. Pipe was extended to the foundation gallery through concrete as the shaft was backfilled. The first holes drilled from the grouting and drainage gallery were widely spaced, being in blocks 16, 18, and 20. Two of the holes were drilled to a depth of 310 feet and the third to a depth of 265 feet. The holes were partially exploratory to observe the grout take at depth.

Between blocks 14 and 22 the "A" grout holes were drilled to the general depth of 200 feet into the foundation rock, with the exception of the holes over the diversion tunnel plug and the "up" holes from the tunnel plug at elevation 3140. The grouting in general followed the pattern of holes shown in figure 10.

The "A" holes were drilled to full depth and grouted from the bottom of the hole upward in stages by means of a packer, except the last 20 feet which was grouted by direct connection to the hole. Pressures up to 500 pounds per square inch were used except in a few cases where the direct hook-up pressure was less due to grout leaks through the concrete of the dam.

The initial drilling and grouting was done with holes spaced 160 feet apart, and this was followed by successively drilling and grouting intervening holes at 80-foot spacing, then 40-foot spacing, 20-foot spacing, and finally 10-foot spacing. It was planned that in the event the closure holes grouted on 10-foot centers took an appreciable amount of grout, the intervening holes on 5-foot centers would be drilled and grouted; otherwise, the holes on 5-foot centers would be left for future drilling and grouting, if necessary. As the grouting progressed, it was found necessary to drill and grout holes spaced at 5-foot centers between blocks 2 and 12 and blocks 23 and 28.

The special grouting included the grouting in the areas of the cutoff shafts which had been put down in the fault zones. This consisted of drilling some shallow holes which were grouted at low pressure and some fairly deep holes which corresponded to the "A" holes beneath the dam. The details of this grouting are shown on figure 15. This grouting was done intermittently between September 1950 and May 1952.

The special grouting included also the grouting of the diversion and spillway tunnels, which was started in August 1951 and completed in September 1952.

The rock surrounding the diversion tunnel was grouted using radially drilled holes and with pressures varying from 100 to 225 pounds per square inch for the holes in the invert and from 25 to 150 pounds per square inch for holes in the arch of the tunnel. The decreased pressure used in the arch was for safety reasons. For the holes that joined with and were a part of the main cutoff curtain of "A" holes, the pressures used were as high as 500 pounds per square inch, but the take of grout was actually less than that of the shallow radial grout holes. The rock surrounding the diversion tunnel was very tight, as shown by the grout taken and also from the fact that seepage in the area grouted was negligible.

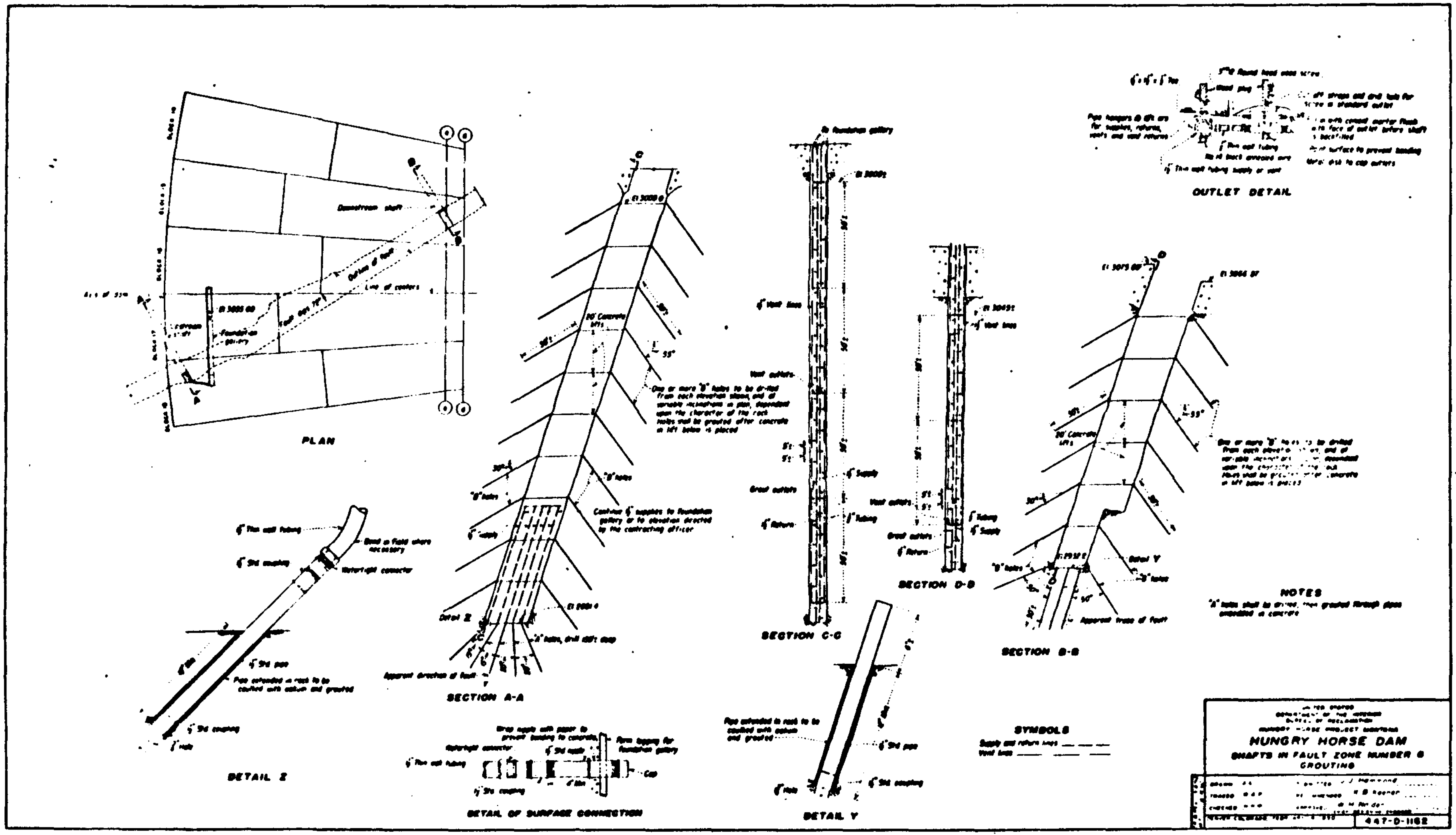


Figure 15.--Shafts and grouting in fault zone No. 6.

The rock surrounding the spillway tunnel was grouted after the backfill grouting of the voids between the concrete lining and the bedrock had been completed. The horizontal section of the tunnel between stations 8+92 and 13+53 was grouted using patterns of drilling radially at each 20-foot station. The patterns consisted of six holes each except at the diversion tunnel junction where one side hole was left out and near the outlet portal where the "up" holes were eliminated. The holes were drilled to depths of 30 feet into the rock and grouted by packer at 20-foot depth with pressures of 250 to 300 pounds per square inch; by packer at 10-foot depth with pressures of 200 to 250 pounds per square inch; and by hooking direct to the grout pipe with pressures of 100 to 200 pounds per square inch.

In the inclined portion of the spillway tunnel between stations 5+09 and 7+15.4, radial holes were drilled to a depth of 30 feet in rock except near the upper elbow where some holes were drilled to lesser depth due to the shallow rock cover over the tunnel. "A" grout holes were drilled in the invert as an extension of the "A" grout holes which were drilled from the foundation gallery of the dam. All holes were drilled to full depth at one drilling and grouted upward with a packer, the last 10 feet being grouted by hooking direct to the embedded grout pipe.

The block of concrete placed at the junction of the diversion tunnel with the spillway tunnel was backfill grouted with the periphery grout system. A total of 1,667 sacks of cement were required to fill the voids between the concrete and the rock in the tunnel arch.

28. Drilling and Grouting Summary. - In specifications No. 2122, foundation grout holes were grouped into seven pay items according to depths ranging up to 310 feet. Holes drilled deeper were to be paid for as extra work. Two methods of injecting grout into the foundation rock were provided, each as a separate item: (a) nipple or single-stage grouting where the supply is connected to the pipe at the top of the hole, and (b) packer grouting whereby a supply pipe with a packer of expansible material is attached to the end and lowered into the hole to the top of the section to be grouted. Nipple and packer grouting were both used extensively, each for a specified purpose. Nipple grouting was used for shallow and low-pressure grouting, while the packer method was employed for grouting at greater depths and pressures. When using the packer method the bottom of the section of a hole was grouted first, and the hole was grouted upward successively by raising the packer to the top of the next higher section to be grouted. Following is a summary of the foundation rock drilling and grouting by item, depth, and zone locations:

Drilling (Linear Feet)

Specifications item No.	Hole depth, feet	Quantity, linear feet	Cost, dollars	Estimated quantity in specs. No. 2122, linear feet
19	0 to 35	112,900	320,638	50,000
20	35 to 60	16,476	46,791	19,000
21	60 to 110	25,228	71,648	25,000
22	110 to 160	15,908	46,929	16,000
23	160 to 210	5,568	16,425	7,000
24	210 to 260	902	2,751	3,000
25	260 to 310	386	1,177	1,000
Total		177,368	506,359	121,000

Grouting (Cubic Feet)

Specifications item No.	Type of hook-up	Grout, cubic feet	Cost, dollars	Estimated quantity in specs. No. 2122, cubic feet
37	Direct	20,843	57,318	84,000
38	Packer	45,863	144,469	40,000
Total		66,706	201,787	124,000

The following tabulation shows the distribution of the quantities of drilling and grouting:

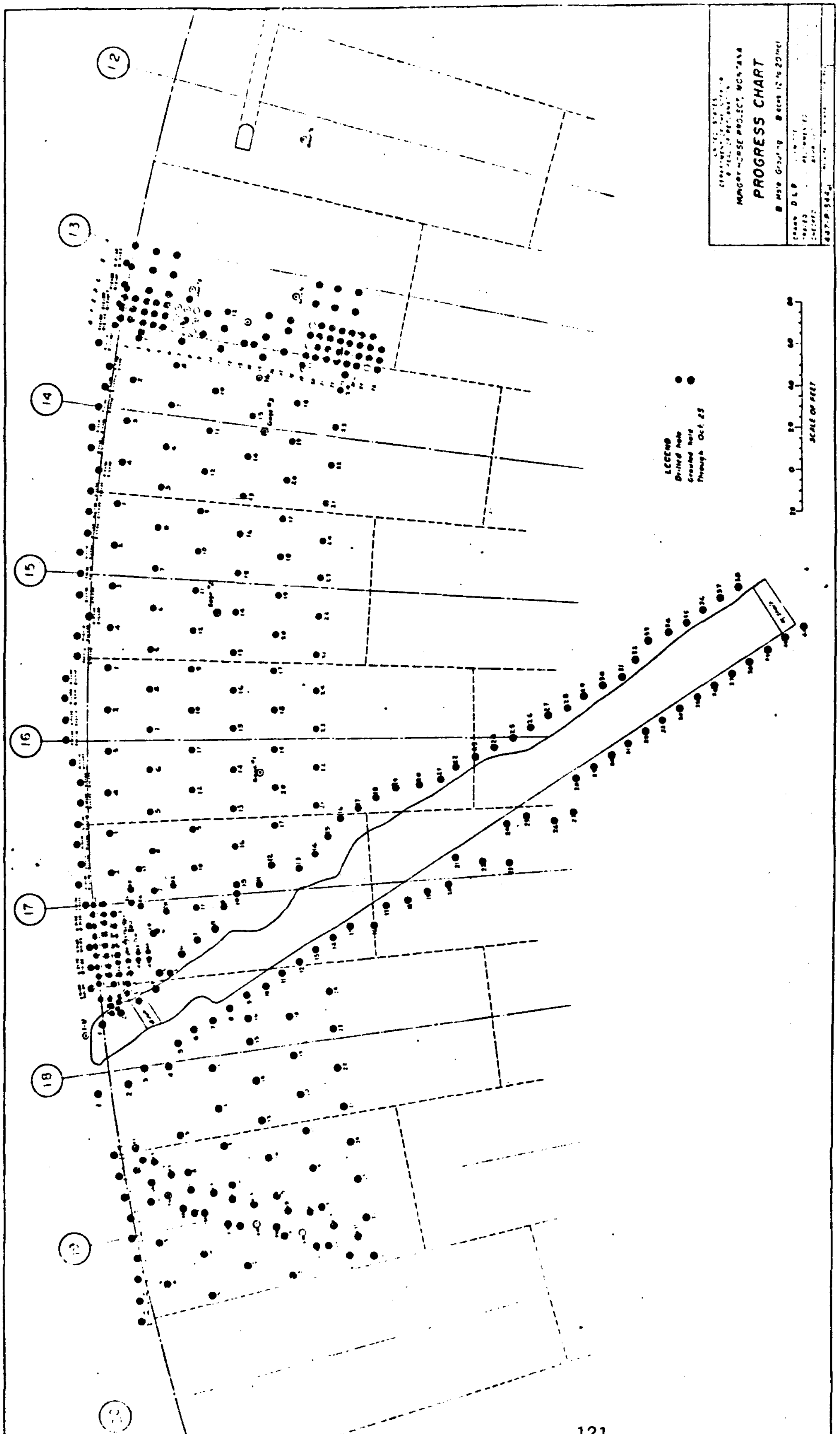
Classification	Drilling, linear feet	Grout, cubic feet
Blanket grouting ("B" holes)	40,319	14,169
Auxiliary curtain ("C" holes)	14,481	6,113
Cutoff curtain ("A" holes)	57,825	29,917
Shafts and faults	4,128	1,082
Seam washing	31,282	8,211
Spillway and diversion tunnels	16,085	6,991
Miscellaneous	13,248	223
Total	177,368	66,706

29. Foundation Drainage System. - The foundation beneath the dam was to be drained by means of a row of holes drilled from the grouting and drainage gallery into the underlying rock. These holes were to be drilled downstream from the line of "A" holes which formed the main cutoff curtain. Their position is shown on figure 10. Short sections of steel pipe 4 inches in diameter were embedded in the concrete floor of the gallery to facilitate starting the drilling of the individual drains. The drain holes varied in depth, being 75 feet deep beneath the center portion of the dam and decreasing uniformly to 50 feet deep at the top of the abutments.

Specifications No. 2122 provided that the minimum diameter of the drain holes at the bottom was to be 2-1/4 inches. The specifications also provided a restriction in drilling in that no drainage hole should be drilled until all adjacent grout holes within a minimum distance of 150 feet had been drilled and grouted.

In the specifications the drilling of the drainage holes was divided into six pay items according to depths, ranging up to 200 feet. Following is a summary of the drilling quantities for the drainage holes according to the pay items:

Specifications item No.	Hole depth, feet	Quantity, linear feet	Cost, dollars	Estimated quantity in specs. No. 2122, linear feet
26	0 to 25	5,644	18,626	5,000
27	25 to 50	4,989	16,464	5,000
28	50 to 75	4,399	14,517	2,000
29	75 to 100	688	2,270	2,000
30	100 to 150	26	90	1,500
31	150 to 200	0	0	500
Total		15,746	51,967	16,000



PROGRESS CHART
LEGEND
Drilled hole
Grouted hole
Through Oct. 25
SCALE OF FEET
12
13
14
15
16
17
18
19
20

(e) *Galleries.*-- The galleries were constructed to a standard section of 5 by 7 feet with a drainage gutter on one side. The foundation gallery is located near the upstream face of the dam and follows the foundation excavation line closely for the entire length of the dam. The grout holes for the principal cutoff curtain and the foundation drain holes were drilled from this gallery.

A drainage gallery similar to the foundation gallery was constructed in the downstream area of the dam extending from block 13 to block 21. This gallery was planned to follow the foundation excavation line closely to points in either abutment that were about 100 feet higher than the excavated foundation in the riverbed. This drainage gallery is interconnected to the foundation gallery by means of three 5- by 7-foot cross adits. It was designed primarily to provide access to the additional drainage holes at some future time to relieve excessive uplift pressure if the upstream grouting and drainage curtains should not be fully effective in controlling the seepage and uplift pressure.

There is also a 5- by 7-foot inspection gallery extending the entire length of the dam at elevation 3550 (15 feet below the roadway of the dam). This gallery was planned as a service gallery to contain the water supply piping for the service of the dam and powerplant. A 5- by 7-foot adit was constructed in block 9 at elevation 3320 to provide access to the foundation gallery in event some future work should be necessary in that area.

There were also such miscellaneous adits as the 5- by 7-foot adit to serve the pumps that remove the accumulations of drainage water in the dam; a 5- by 7-foot filling line gallery in which the valves that control the filling lines to the penstock are located; and a 5- by 7-foot adit to service the air compressor used for furnishing air for the ice-prevention systems for the trashracks.

(f) *Transformer Vaults.*-- Transformer vaults are located in blocks 14 and 19 in the dam near each of the elevator shafts. In these vaults are located the lighting and power service distribution equipment for the dam and powerplant. The floor elevation is 3520.08 and the vaults are 10 feet 6 inches by 23 feet in plan and 10 feet 6 inches high.

43. Elevator Towers and Elevators. - The elevator towers and shafts were constructed at blocks 14 and 19. The walls were checked for earthquake load of 0.15 gravity and wind load of 20 pounds per square foot. The floors were designed for a live load of 150 pounds per square foot with the exception of the lobby floor and elevator machinery room. These floors were designed for a live load of 300 pounds per square foot. The roofs were designed for a live load of 40 pounds per square foot. The stairways were designed for a live load of 100 pounds per square foot.

Two electric elevators are installed in the dam to provide transportation between the top of the dam and the adits. The elevators were designed for passenger service but may be used for handling freight up to the capacity of the cars. The capacity of each car is 60 passengers or 10,000 pounds. The cars are rated at a speed of 500 feet per minute and the travel is 465 feet in block 14 and 480 feet in block 19. There are eight landings in each hoistway. The cars operate between machined steel guides and are equipped with safety devices, lights, heaters, ventilating blower and telephone.

The elevator is entirely automatic, with provision for normal operation by an attendant and whenever desired without an attendant. The operating equipment consisting of motor-generator set, direct-current gearless traction machine, controller, and speed governor are located in the machine room directly over the hoistway.

In the case of failure of equipment causing the car to stop in the hoistway, it is possible to telephone for help or leave the cars through the emergency exits in the side or top of the cars. A system of ship ladders and platforms is provided at one side of the hoistways which can be reached either from the side exits or from the top of the cars. Doors are installed at each adit level for access to the ladders and the doors are to be locked on the adit side at all times. The doors have spring locks on the ladder side. If it should be necessary to enter the hoistway by way of the ladders, the power to the elevator must be cut off.

The elevators were designed and installed in accordance with the provisions of the "American Standard Safety Code for Elevators, Dumbwaiters, and Escalators."

44. Heating and Ventilating System. - The ventilating system has been provided to prevent stagnation and reduce humidity in habitable spaces in the dam. Most of the ventilation for the lower galleries and adits in the dam is provided from the powerplant ventilating system. Fans are installed in certain galleries to provide positive air circulation throughout the structure. This includes both fresh air and recirculated air. The elevator towers utilize a separate outdoor supply. The air supply is taken from vents in the towers and is filtered and tempered to make it suitable for ventilating purposes. Circulation of air in each of the transformer vaults, located in blocks 14 and 19, is accomplished by fans which draw air from the elevator shafts through adits leading to the vaults and force the air out of the vaults through the discharge ducts to the downstream face of the dam. This provides fresh air for persons entering the vaults and removes air heated by the transformers. These fans are normally controlled by means of a reverse acting thermostat, but manual control may be accomplished by manipulation of selector switches located in the fan motor starters.

Heating is provided by wall-mounted heaters in the elevator towers and by duct heaters in the principal galleries.

45. Gantry Crane. - A 125-ton-capacity, outdoor traveling-type gantry crane is installed on top of the dam for the erection, installation, and maintenance of the penstock fixed-wheel gates and hoists and the installation and removal of the outlet gates and penstock intake stoplogs (fig. 27).

The crane has a 125-ton-capacity fixed hoist mounted in the main hoist machine house with the block cantilevered out from the upstream side of the crane and overhanging the penstock gates and hoists. The lower block is suspended from 16 parts of 1-1/4-inch-diameter, 6 by 37 wire rope and supports a lifting eye. The hoist is powered by a 60-horsepower motor and operates at varying speeds up to 5.5 feet per minute. A 25-ton-capacity, trolley-mounted, auxiliary hoist operates on a runway located between the upstream legs of the gantry. The runway is cantilevered out over the stoplog slots on the upstream face of the dam and extends between the legs so that the auxiliary hook may be positioned on the centerline of the crane. The lower block is suspended from eight parts of 3/4-inch-diameter, 6 by 37 wire rope and supports a sister-type hook bored for a horizontal lifting pin which facilitates the attachment of slings and lifting devices. The auxiliary hoist is powered by a 40-horsepower motor and operates at varying speeds up to 17 feet per minute.

Both the main and auxiliary hoists are equipped with electric brakes having a capacity equal to one and one-half times the rated torque of their respective motors. Block-actuated limit switches which limit the upward travel of the block are installed on each hoist. The auxiliary hoist is equipped with two screw-type limit switches geared to the drum shaft. One switch is arranged to light a red indicator lamp on the operator's control stand when the hook is within 15 feet of its lower end of travel. The second switch is arranged to deenergize the hoist motor and set the electric brake when the block reaches its lower limit of travel.

A 2-horsepower motor, mounted on the trolley frame, is connected through gearing and shafting to one driving wheel on each side of the trolley and moves the trolley at varying speeds up to 29 feet per minute. The trolley drive motor is equipped with an electric brake.

The gantry frame is constructed of welded steel plate and angle girders riveted with gusset plates to four welded steel plate and angle box-type legs. Each leg is pin-connected to an equalizer beam which is supported by one two-wheel idler truck and one two-wheel drive truck. Each of the four drive trucks is powered by a 7-1/2-horsepower motor which travel the crane at varying speeds up to 56 feet per minute. An electric brake is mounted on each of the drive motor shafts. Since the crane runs on a curved track, the diameter of the upstream wheels is 24 inches and the diameter of the downstream wheels is 23.458 inches. Spring bumpers are mounted on one end of the gantry crane to prevent damage to the crane structure in case of collision with the stops located

CHAPTER XIII--*Construction*-- EXCAVATION AND PREPARATION OF SITE

A. Diversion and Care of the River

156. Description of the Tunnel. - The diversion tunnel was a bore with a nominal diameter of 36 feet, horseshoe shaped in section and approximately 1,100 feet long. It was driven in the right bank where adequate coverage was provided with the least amount of excavation, and where previous investigations proved the existance of suitable rock. The tunnel was driven from the downstream portal on a bearing S64°18'46" W, parallels the river on a bearing N 80°41'14" W, and exits on a bearing N 30°41'14" W. The tangents, looking upstream, are connected by curves of 300-foot radius with angles of intersection of 35° and 50° respectively. The lower 250 feet of the diversion tunnel is incorporated in the spillway tunnel. The invert elevation at the upper portal is 3072.09, and at the outlet 3070.00, the gradient being -0.19 percent.

The diversion tunnel passed the waters of the South Fork River under the dam foundation to permit excavation and subsequent concrete placement. Full diversion of the river was accomplished April 11, 1949, permitting the contractors to proceed with unwatering and excavation activities in the river channel.

Upon completion of diversion requirements, the tunnel was closed by means of steel stoplogs set in a concrete inlet gate structure, and a concrete tunnel plug which restores the original impermeability and load bearing qualities of the foundation rock. The gate structure and the tunnel plug were built in accordance with specifications No. 2122.

Preliminary work was started by the contractor November 3, 1947, 6 weeks prior to his receiving notice to proceed. Excavation was pushed steadily throughout the fall, winter, and spring months until final acceptance on June 30, 1948.

157. Changes in Plans. - The tunnel plug keyways shown on specification drawings were omitted from construction, since joint systems in the rock broke in such manner as to eliminate the necessity for this special excavation. Instability of the rock at the upstream portal made it necessary to advance the tunnel portal face from station 2+27 to station 2+65. It was also necessary to discontinue the excavation of the tunnel to a nominal 36-foot diameter at station 3+03 and to hole through to the upstream or intake portal with a narrow pioneer tunnel 16 feet wide. This pioneer bore was wide enough to permit use of excavating equipment and allowed completion of upstream open cut excavation. The unpredictable nature of rock upstream from station 3+03 made it unwise to enlarge the tunnel further under the original contract; hence, the remaining excavation was left until immediately prior to construction of the concrete inlet portal and gate structure under specifications No. 2122. The downstream portal was shifted downstream to give sufficient room for construction of a cofferdam downstream from the toe of the dam.

158. Cofferdams. - Upstream cofferdam construction was started March 1949 by placing concrete cutoff walls and rock-filled timber cribs extending into the river from each abutment (fig. 91). Complete diversion of the river through the tunnel was accomplished on April 11, 1949. However the cofferdam was overtopped by high water in early June resulting in a subsequent delay, until June 23, in unwatering the river channel portion of the foundation. Excavation in this portion could not be continued until this unwatering was accomplished. The cofferdam was not raised during the 1949 season.

Stream runoff forecasts in early 1950 indicated that a near-record flow of the South Fork of the Flathead River could be expected, making the overtopping of the cofferdam a distinct possibility. Such an overtopping would retard or stop all operations for several weeks, and seriously interfere with program accomplishment. For this reason it was decided to raise the cofferdam from elevation 3136 to elevation 3179. This work was started on March 22, 1950, with the Government furnishing the necessary timbers and the prime contractor performing the work. However, heavy runoff over a relatively long period of time reduced the flood danger and there were no high flood peaks to threaten overtopping of the raised cofferdam. The highest flow was 29,000 second-feet at the recording station below the dam. It is thought that the ponding effect of restricted discharge through the diversion tunnel served to level off the peak flow to safe proportions.

Figure 91.--General view of the upstream cofferdam looking [illegible]

B. Exploration and Foundation Tunnels

159. Exploration Tunnels. - These tunnels were excavated into the foundation of the dam abutments to augment surface inspection of the site and to confirm, by observation of undisturbed material in place, the conclusions relative to the character of the rock based upon core drilling. Three tunnels were excavated into the right abutment foundation area at elevations 3110, 3240, and 3320 and one tunnel in the left abutment at elevation 3110.

The two lower tunnels had been previously excavated by Government forces to a nominal 3.5-foot width and 6.5-foot height, and about 100-foot length. These were subsequently enlarged to a nominal 6.5-foot width and 8.5-foot height, and increased in length to 210 feet. The two upper tunnels on the right abutment were to be excavated to the same dimensions as the lower but only 110 feet long. Excavation was sufficient to accommodate standard Bureau 5- by 7-foot concrete-lined inspection and drainage tunnels. These tunnels, to be incorporated into the system of galleries of the dam and powerplant, were to be used to check the efficacy of foundation grouting, to drain off minor seepage through the foundation, and to provide access to the foundation for auxiliary grouting.

The tunnels were located 30 feet downstream from the axis of the originally proposed gravity-type dam. The two lower tunnels, A and B, were enlarged and extended, and a third tunnel C, at elevation 3240, was constructed before the design for the dam was changed from the gravity to curvilinear type. The contractor had excavated only the portal of the fourth tunnel D, elevation 3320; hence, an order for changes permitted relocation of this bore about 350 feet upstream where it could be incorporated in the arch dam. Tunnels A, B and C have been retained, unlined, for inspection purposes. The locations of these tunnels are shown on figure 92.

160. Foundation Tunnels. - Three additional tunnels were excavated in the dam foundation. Two are in the left abutment at elevations 3140 and 3320, their lengths being 157.2 feet and 261.8 feet, respectively. One is in the right abutment at elevation 3140 and its length is 114.2 feet. All were excavated to a nominal 6.5 feet in width and 8.5 feet in height. These three tunnels, together with exploration tunnel D, were concrete lined and incorporated into the system of galleries in the dam to check the efficiency of foundation grouting, to drain off minor seepage through the foundation, and to provide access to the foundation for additional auxiliary grouting.

The foundation conditions encountered in these tunnels are described in chapter II, Geology.

C. Excavation for Dam and Powerplant

161. General Description. - Excavation on the project occurred in the following areas: dam and powerplant foundations, powerplant tailrace, retaining wall on the right side of the tailrace channel, outlet pipe anchors and encasements, valve house structure, spillway inlet, the section of the powerplant service road between the plant and the spillway outlet structure, the cutoff and training walls for the spillway outlet structure, and the site for a 150,000-gallon water storage tank. Excavation in shafts was extended into sound rock on each side of faults or seams to such depths as required to seal them. Additional excavation was performed in the diversion tunnel for a closure structure at the intake portal, to remove slide material from the upper portal open cut, to trim the walls and invert of the outlet channel, and to finish trenching in the invert for a grouting ring.

Contractor supervisory and administrative personnel began to arrive on the project late in April of 1948, and established temporary field offices in Columbia Falls and in the Government camp near the jobsite. Excavation of the dam foundation got under way on the right abutment late in June of 1948 and continued on a round-the-clock, 7-day-week basis. Except for hand stripping, scaling, and excavation of shafts in fault zones, foundation excavation was considered completed in May 1950.

162. Development of New Methods. - The contractors were faced with a difficult task in the excavation of the right bank because of the relatively steep slope. An

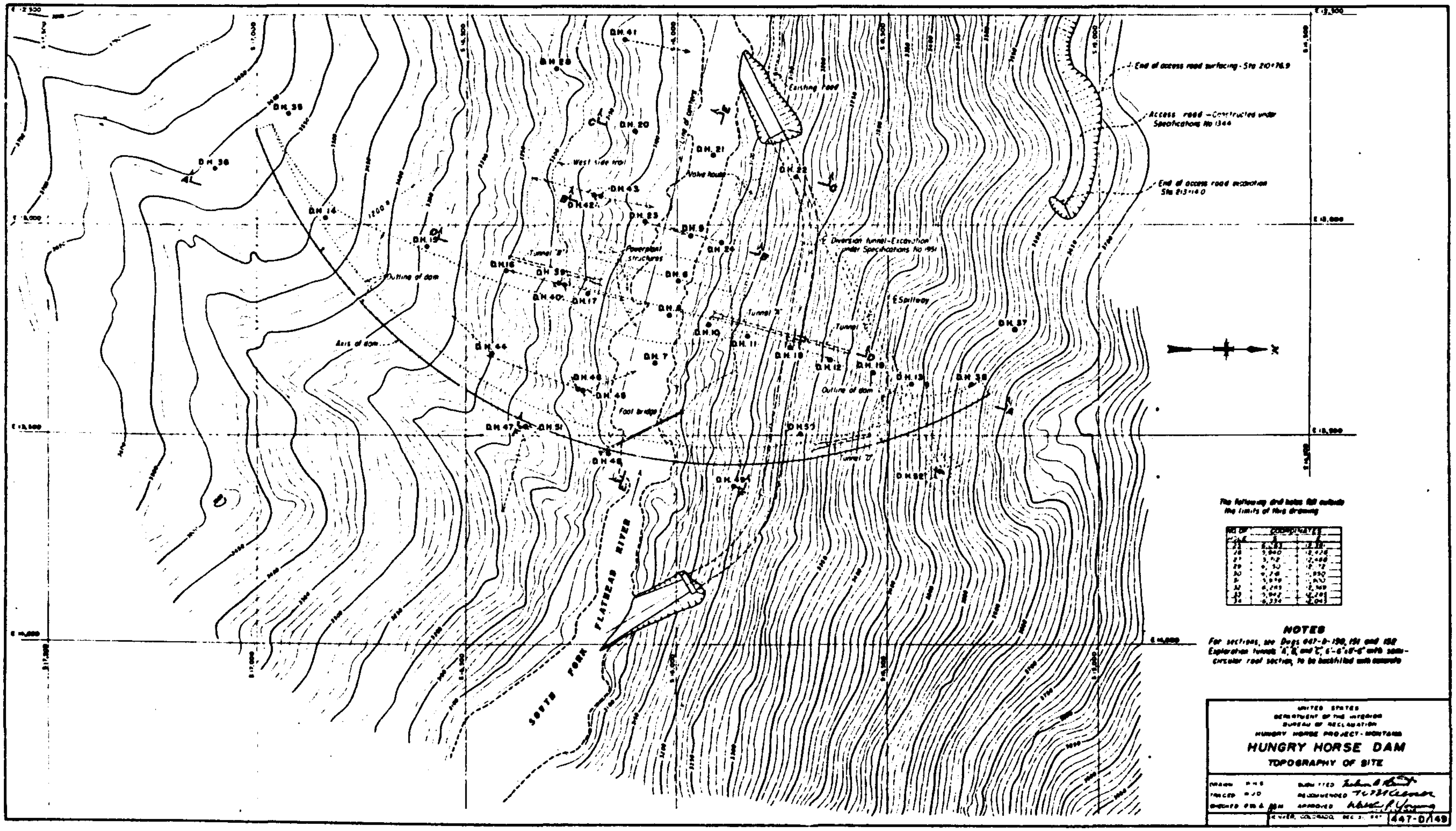


Figure 92.--Topographic map of dam site, showing location of tunnels A, B, C, and D. From drawing 447-D-149.

attempt to use wagon drills was largely unsuccessful due to the inaccessibility of the area and difficulties connected with spotting and moving the heavy pieces of equipment with hand labor and mechanical tuggers.

A new method of drilling was therefore employed. It involved the use of diamond drills and long holes from the elevation 3410 bench on the right abutment. Holes up to 100 feet deep were drilled downward at an angle of approximately 45° from the horizontal through a plane roughly parallel to the exposed face of the abutment, then loaded and blasted. Boyles Brothers of Salt Lake City, subcontractors for the spillway tunnel excavation, had the necessary equipment and personnel on the job, hence were assigned the work.

The diamond drill or "long-hole" method was given a 1-month trial. A very satisfactory foundation with regard to minimum shattering and regular cleavage of joint planes resulted. (Figs. 93 and 94.) Certain areas of the foundation, such as projecting ribs, were drilled with a single machine setup, effecting considerable saving of time in moving and setting up equipment. Diamond drills need be placed only at long intervals on a slope compared with jackhammers and wagon drills. However, diamond drill machines require a constant water pressure of 80 to 100 pounds per square inch and an operating air pressure of at least 100 pounds per square inch. A comparatively level bench is necessary for the drill and guying is sometimes required to obtain stability and minimum vibration.

Diamond drill holes 1-3/4 inches in diameter totaled 8,728 feet. An additional 101 feet were lost through cave-ins. Some 8,350 pounds of 60 percent dynamite and 9,000 feet of detonating fuse cord were used. Excavated rock totaled 18,125 cubic yards, or 2.07 cubic yards per foot of hole drilled. Explosive averaged 0.45 pounds per cubic yard excavated.

Figure 93.--Drilling 100-foot-deep holes in block 25 on right abutment of dam foundation by use of diamond drills. P447-105-1392, July 13, 1949.

APPENDIX C

DOCUMENTS REVIEWED
(Project and Region names omitted)

1. Technical Data Workbook, Hungry Horse Dam, prepared by John N. Carver, Converse Ward Davis Dixon, Inc., July 18, 1980.

2. Computer printouts, Uplift Pressures 1965-1981.

3. Final Construction Report, pp. 6-8 and 28-105.

4. Monthly Grouting Reports, October and November 1949, March through November 1950, March 1951 through January 1952, March 1952, May 1952 through April 1953.

5. Specifications No. 2122, Hungry Horse Dam and Powerplant.

6. "Brief Resume of Hungry Horse Foundation Geology", undated.

7. Logs of drill holes Nos. 6 through 54.

8. "Designers Operating Criteria", Hungry Horse Dam, August 1952.

9. Field trip report dated November 10, 1952 from Engineer R. A. Pettitt to Chief Engineer, subject: "Drilling and grouting operations at Hungry Horse Dam, from April to Oct. 1952".

10. Speed letter dated May 20, 1952, from Acting Chief Designing Engineer to Construction Engineer, Columbia Falls, Montana, subject: "Extension of A-hole grout curtain, Hungry Horse Dam".

11. Letter dated February 29, 1952, from Construction Engineer to Chief Engineer, subject: "Drilling uplift pressure holes, Hungry Horse Project".

12. Field trip report dated November 6, 1951, from F. H. Lippold, Engineer, to Chief Engineer, subject: "Pressure grouting, Hungry Horse Dam".

13. Field trip report dated June 5, 1951, from Chief Designing Engineer (W. H. Nalder) to Chief Engineer, subject: "Visit to Hungry Horse Dam and Eklutna Projects".

14. Letter dated June 4, 1951, from Chief Engineer to Construction Engineer, subject: "Report of May 23, 1951 by Consulting Engineer J. L. Savage on Inspection Trip to Hungry Horse Dam".

15. Savage, J. L., letter report dated May 23, 1951, to L. N. McClellan, Chief Engineer, regarding site inspection of May 20-21, 1951.

16. Letter dated October 20, 1950, from Construction Engineer to Chief Engineer, subject: Results of lift seam washing and grouting, left abutment for Hungry Horse Dam with enclosure of report dated October 17, 1950 from Geologist (Soward) to Field Engineer, subject: "Drill Hole No. 44, 36" diameter calyx hole".

17. Letter dated September 27, 1950, from Construction Engineer to Chief Engineer, subject: "Report on efficacy of clay seam treatment as evidenced by drill hole No. 43, Hungry Horse Project", with enclosures.

18. Letter dated August 15, 1950, from Construction Engineer to Chief Engineer, subject: "Monthly report of activities of project geologist for July 1950, Hungry Horse Project", with enclosures.

19. Memorandum dated July 11, 1950, from Field Engineer to Office Engineer, subject: "Treatment of 'Lift' seam, Left Abutment, Hungry Horse Dam".

20. Letter dated June 30, 1950, from L. N. McClellan, Chief Engineer, to J. L. Savage, regarding Savage letter of June 26, 1950.

21. Savage, J. L. and Berkey, Charles P., letter report dated June 15, 1950, to L. N. McClellan, Chief Engineer, regarding site inspection of June 11-15, 1950.

22. Letter dated June 6, 1950, from Construction Engineer to Chief Engineer, subject: "Report of Geology Unit, Hungry Horse Project", with enclosures.

23. Letter dated April 11, 1950, from Construction Engineer to Chief Engineer, subject: "Clay Seam Treatment, left abutment, Hungry Horse Dam", with enclosures.

24. Letter dated March 10, 1950, from Construction Engineer to Chief Engineer, subject: "Installation of grout tubings for shaft in Fault Zone No. 6, Hungry Horse Project".

25. Letter dated March 3, 1950, from Construction Engineer to Chief Engineer, subject: "Cutoff Shaft Logs, Fault Zone #6, Hungry Horse Dam".

26. Letter dated February 28, 1950, from Construction Engineer to Chief Engineer, subject: "Fault zone cut-off shafts, Hungry Horse Dam".

27. Field trip report dated February 15, 1950, from W. H. Nadler to Chief Engineer, subject: "Inspection of Fault Zone Cutoff Shafts, Hungry Horse Dam".

28. Speedletter dated December 23, 1949, from Construction Engineer to Chief Engineer, subject: "Foundation Excavation and Exploration, Hungry Horse Project".

29. Letter dated November 9, 1949, from Construction Engineer to Chief Engineer, subject: "Report of Geology Unit, Hungry Horse Project".

30. Letter dated October 5, 1949, from Acting Chief Construction Engineer to Construction Engineer, Columbia Falls, Montana, subject: "Grouting off seepage flows, left abutment, Hungry Horse Dam".

31. Field trip report dated September 2, 1949, from F. H. Lippold, Engineer to Chief Engineer, subject: "Foundation grouting, Hungry Horse Dam".

32. Letter dated August 31, 1949, from Construction Engineer to Chief Engineer, subject: "Report of Geology Unit, Hungry Horse Project".

33. Letter dated August 26, 1949, from Construction Engineer to Chief Engineer, subject: "Geologic logs of Diamond Drill Holes, Hungry Horse Project, Montana".

34. Letter dated August 25, 1949, from Construction Engineer to Chief Engineer, subject: "Geologic logs of Diamond Drill Holes, Hungry Horse Project".

35. Letter dated August 25, 1949, from Chief Designing Engineer to Construction Engineer, Columbia Falls, Montana, subject: "Treatment of fault zone, Foundation excavation, Hungry Horse Dam".

36. Letter dated August 12, 1949, from Construction Engineer to Chief Engineer, subject: "Foundation Excavation, Hungry Horse Dam".

37. "Geologic Investigation of the Major Joint Sets on the Right Valley Side, Hungry Horse Project, Montana", by Geologist K. Soward and M. Logan, undated.

38. Letter dated August 9, 1949, from Acting Construction Engineer to Chief Engineer, subject: "Foundation Grouting, Hungry Horse Dam".

39. Telegram dated August 4, 1949 from Nalder to Coram, subject: "Treatment of fault zone, Foundation excavation, Hungry Horse Dam".

40. Letter dated August 2, 1949, from Construction Engineer to Chief Engineer, subject: "Treatment of fault zone, Foundation excavation, Hungry Horse Dam".

41. Letter dated August 1, 1949 from Chief Construction Engineer to Construction Engineer, Columbia Falls, Montana, subject: "Foundation grouting, Hungry Horse Dam".

42. Letter dated July 15, 1949, from Construction Engineer to Chief Engineer, subject: "Foundation excavation, Hungry Horse Dam and Powerplant".

43. Letter dated July 1, 1949, from Head, Dams Division, to Construction Engineer, Columbia Falls, Montana, subject: "Treatment of clay seam on left abutment bed plane of Hungry Horse Dam".

44. Letter dated June 9, 1949, from Chief Engineer to Construction Engineer, subject: "Report of May 27, 1949 by Consultants J. L. Savage and Charles P. Berkey, Hungry Horse Dam".

45. Field trip report dated June 17, 1949, from E. N. Harshman, Geologist, to Chief Engineer, subject: "Consulting Board review of Hungry Horse Dam foundation".

46. Geologic Report on the Hungry Horse Dam Site, Montana, by Kenneth Soward, Geologist, January 1949.

47. Field trip report dated November 3, 1948, from K. P. Keener to Chief Engineer, subject: "Visit at Hungry Horse Damsite".

48. Letter dated June 14, 1948, from E. N. Harshman to Chief Engineer, subject: "Geological Inspection of Hungry Horse Project".

49. Letter dated February 17, 1948, from Construction Engineer to Chief Engineer, subject: Minor crushed zone at Hungry Horse Dam Site".

50. Field data on Geologic Investigations in Abbott Gorge, Hungry Horse Project, Montana" with drill logs, January 1948.

51. Letter dated December 16, 1947 from W. H. Irwin, Geologist to Chief Engineer, subject: "Field inspection at Hungry Horse Damsite and Abbott Gorge areas".

52. "Examination of Hungry Horse Damsite" by Board of Consultants; J. L. Savage, Chairman, Dr. Charles P. Berkey, William H. Irwin, November 18, 1947.

53. "Hungry Horse Reservoir, A Geological Study", Based on a Re-examination of the field in July 1947 by Charles P. Berkey, Geologist, September 5, 1947.

54. Memorandum dated July 30, 1947, from C. J. Okeson, Geologist to Regional Director, subject: "Report of Field Trip to Hungry Horse Project, July 3-8, 1947."

55. Office memorandum dated June 13, 1947, from D. P. Barnes to W. H. Nalder, subject: "Hungry Horse Drill Program".

56. Letter dated May 7, 1947, from Project Engineer to Chief Engineer, subject: "Exploration for extension of broken zones in drill holes 17, 39 and 20, Hungry Horse Project".

57. Letter dated April 22, 1947, from Project Engineer to Chief Engineer, subject: "Geologic section on proposed axis G-G, Hungry Horse Project, Montana".

58. Letter dated April 9, 1947, from Project Engineer to Chief Engineer, subject: "Geologic Section, Hungry Horse Dam Site".

59. Letter dated April 9, 1947, from Assistant Chief Engineer, Civil, to Project Engineer, Kalispell, Montana, subject: "Exploration Program, Hungry Horse Damsite".

60. Letter dated March 28, 1947, from Project Engineer to Chief Engineer, subject: "Surface geologic map, Hungry Horse Project, Montana".

61. Letter dated December 10, 1946, from Chief, Engineering and Geological Control and Research Division to Project Engineer, Kalispell, Montana, subject: "Memorandum on exploration tunnel B, Hungry Horse Project".

62. Field trip report dated August 30, 1946, from E. N. Harshman to Chief Engineer, subject: "Geologic inspection of Hungry Horse Damsite".

63. Memorandum dated July 25, 1945, from Fred O. Jones, Geologist, to Project Engineer, Paul A. Jones, subject: "Geological investigations, Hungry Horse Dam and Reservoir Site".

Mission of the Bureau of Reclamation

The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.

The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.

Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.

A free pamphlet is available from the Bureau entitled "Publications for Sale." It describes some of the technical publications currently available, their cost, and how to order them. The pamphlet can be obtained upon request from the Bureau of Reclamation, Attn D-822A, P O Box 25007, Denver Federal Center, Denver CO 80225-0007.