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UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC LABORATORY REPORT NO. HYD-6

Technical Memorandum No. 255

STRUCTURAL DESIGN STUDIES AND MODEL TESTS OF HOOVER DAM SPILLWAYS

By

B. W. STEELE, D. C. McCONAUGHY and E. W. LANE

Denver, Colorado
October 2, 1931

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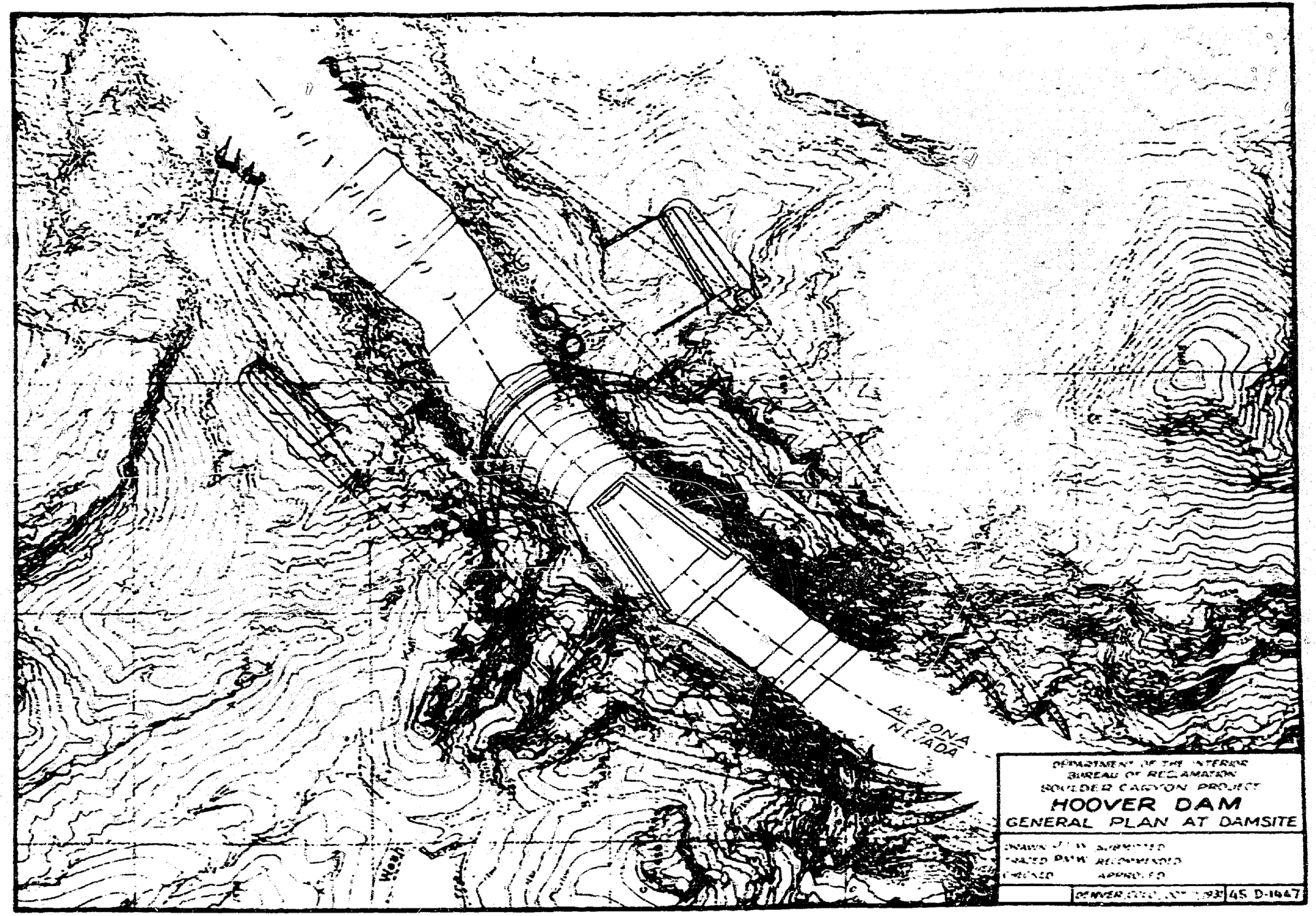

ARIZONA
NEVADA
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM
GENERAL PLAN AT DAMSITE
45 D-1447

TM-255
HYD-.6

Denver, Colorado, October 2, 1931

MEMORANDUM TO THE CHIEF DESIGNING ENGINEER

By F. T. Steele, D. C. McConaughy and E. T. Lane

Subject: Structural Design Studies and Model Tests of Hoover Dam Spillways.

SUMMARY

1. Sketches of the general layouts of the designs now proposed for the Hoover Dam spillways. (Exhibits 1 and 2.) These meet the requirements of the Colorado River Board that the spillway capacity alone must be 400,000 second-feet with lake water surface at El. 1202; and that the total discharge of spillways and power plant must not exceed 95,000 second-feet with lake water surface at El. 1220. The two spillways have been made equal in capacity and the discharge through the power plant to be as 10,000 second-feet. A model of this design, built to a scale of 1" to 40, has been tested at Fort Collins. A model, to a scale of 1 to 20, is being built at Denver and will be completed in time for inspection by the Board at its coming meeting.

2. It will be noted that the design differs from that shown in the specifications, in that it consists of a drum gate crest, 400 feet long, whereas the specifications show a 300-foot free crest with a 50 x 50-foot Stoney gate across the channel at the upper end.

3. The design contemplated by the specifications was prepared for a total spillway and outlet capacity of 400,000 second-feet with lake water surface at El. 1202. In accordance with General

Sibert's letters of December 6, 1930, and December 16, 1930, the capacity was increased to that stated in paragraph 1.

4. Table I shows lengths of fixed crest necessary for a given size of Stoney gate to meet the maximum requirement of 400,000 second-feet total discharge through spillways alone; the discharge at El. 1229, which it is desired should not exceed a total of 75,000 second-feet; and the elevation at which the desired total discharge of 75,000 second-feet through spillways and power plant would be reached. It is apparent from the table that a sacrifice in the elevation at which the total discharge of 75,000 second-feet is reached would be greatly in the interest of economy. The combination of a 60 x 80-foot gate with a 400-foot crest was selected for study and designs were prepared. The design for the Nevada side is attached (Exhibit 3).

TABLE I

FIXED CREST WITH STONEY GATE AT END OF CHANNEL

Discharge of Two Spillways and Power Plant for Various Fixed Crest Lengths with Stoney Gates

Total Capacity Spillways Alone 400,000 Sec.-Ft. at El. 1232.

Size Gate-Ft.	50 x 80			55 x 80			60 x 80			
Length Fixed Crest - Ft.	400	500	600	400	500	600	400	500	600	700
Discharge at El. 1229 - Thousand Sec.-Ft.	110	105	100	93	88	83	74	71	66	63
El. at which Discharge is 75,000 Sec.-Ft.	1227.1	1227.7	1227.2	1227.2	1227.6	1228.0	1228.3	1228.6	1228.8	1229.0

5. A model of the Nevada spillway was built at Fort Collins to a scale of 1 to 60. The performance of this model was distinctly disappointing. The curvature in the channel below the gate sets up a wave on the left side of the spillway at the lower end of the spillway crest; which is very greatly magnified by the cross current formed by the water pouring over the open crest. This wave is reflected back and forth across the transition and shaft, causing very rough surface flow and filling the shaft completely with a heavy spray.

6. It is obvious that the action of the cross current is a function of the energy of the water, which in turn is dependent on the difference in elevation between lake and channel water surfaces. A decrease in this free end be accomplished only by raising the water surface in the channel, which would cut down the capacity of the gate, requiring either a larger gate or a longer crest. It also appeared that no curvature should be allowed in the channel below the gate, necessitating throwing the upper end of the channel further into the side of the canyon, which would greatly increase the amount of excavation and result in a sharper curve above the gate. The final result would be a material increase in cost, the amount depending on the required size of the gate, which already exceeded precedent.

7. Closer examination of the site proposed for the Arizona spillway caused doubt as to the nature of the rock and the advisability of placing the spillway in this location at the top of an almost vertical cliff; a doubt which was later confirmed by geological investigations and by drill holes. The upper end of the large draw just

above the dam affords a better site but is not adapted to this type of spillway on account of the heavy approach cut to the gate which is required. Only a limited length of free crest is economically available, requiring a large gate if the drop in water surface from lake to channel is made small.

8. In view of the above, it was decided that a conservative policy dictated the change to the drum gate crest, which requires no unprecedented size of gate, or sacrifice in requirements, and is better adapted to the topographic conditions and to the balancing of discharges as required; i. e., 31,500 second-feet uncontrolled discharge with water surface at El. 1229, and 200,000 second-feet maximum with water surface at El. 1232, for each spillway.

9. Simultaneously with the Stoney gate experiments at Fort Collins, a model of a design for a drum gate crest was built at Montrose to a scale of 1 to 20. A very large reduction in the estimated cost could be made by increasing the velocities toward the lower end of the channel; so as to have as small a tunnel section as possible at the portal. The channel floor, beginning about 150 feet from the lower end of the crest, was accordingly steepened so as to throw a part of the transition curve into the channel itself. This model showed the same disappointing feature as the Fort Collins Stoney gate model; namely, that any considerable drop from lake to channel water surface resulted in too great energy of cross flow, creating conditions unsuited to a closed section. While the structure was not lacking in capacity, the flow conditions were bad. So far as flow in the shaft

and tunnel were concerned, a satisfactory condition was created by choking the tunnel entrance to form, at discharges exceeding about 150,000 second-feet, a submerged orifice, with a bell mouth entrance and an air duct below. At maximum capacity, conditions were fairly satisfactory; but at low discharges flow in the channel itself was rough. Numerous types of baffles placed in the channel effected only minor improvement. This design was rejected and a model of a new design built at Fort Collins.

10. In this new design the channel was modified to omit the extra fall given to the lower end of the channel floor in the Montrose model. The new model had an open top from the lower end of the spillway crest to the lower end of the vertical transition curve so that flow conditions around this curve and in the shaft could be observed readily. The vertical curve began at the elevation of the channel floor.

11. Numerous different set-ups were made by means of false-work placed in this model, these experiments covering a period of several weeks. Baffles effected only minor improvement. It was evident, both from this model and that at Montrose, that deep water in the channel was by far the most practical means of producing smooth flow. Since the water must pass through the critical depth at the beginning of the vertical transition curve, it appeared necessary to make the water surface elevation at critical depth coincide with an elevation which would give a satisfactory depth of water at the lower end of the channel; found by experiment to be about 25 feet.

12. This can be accomplished by a rise in the bottom downstream of the end of the spillway crest, by a contraction of the sides, or by a combination of the two. By means of experimenting with numerous combinations the method embodied in the design now proposed was evolved; consisting of a weir across the lower end of the channel, with a moderate contraction on the crest side.

13. From experiments it was also found that some distance between the lower end of the spillway crest and the point of critical depth was necessary for smooth flow; the distance (60 feet) shown on the design now proposed was found to be best. Less distance was not effective, and more showed either no improvement or a deterioration.

14. Baffles along the bottom of the channel, while not considered essential, still effect enough improvement to be thought well worth while. The type shown has so far given the best results of many tried; experiments with other types are still in progress.

15. The curve of the transition was also the subject of considerable study. The method often employed of using a parabola whose equation is derived from the mean velocity of the water and the acceleration due to gravity gives a curve much too sharp to be suitable. This was anticipated, and the first designs substituted for the mean velocity in this method the total energy of the water (depth plus velocity head) converted into velocity. Even this gave too abrupt a curve. The curve used was determined by experimenting till a satisfactory condition was obtained.

16. The depths in the shaft and tunnel of the models check the computed depths very closely. The water surface elevations in the side channel are consistently lower than those computed, by a small amount, the difference being due principally to a lack of agreement between the observed and the computed water surfaces near the control section at the lower end of the channel. This difference probably is due to the assumption of a uniform (mean) velocity over the entire wetted section; an assumption which, always in error, probably is much more than usually so in this particular case.

CONCLUSIONS

17. From the results of the structural design studies and model tests, it is concluded:

(a) That the type of spillway shown in specifications No. 510 is not as well adapted to the conditions at Hoover Dam as a drum gate crest of a design similar to the one shown in Exhibit 1 attached.

(b) That the combination of a Stoney gate and uncontrolled crest produces unsatisfactory hydraulic conditions.

(c) That the drop from the reservoir water surface to the water surface in the channel should be the minimum that conditions will permit.

(d) That an offset in the channel at the end of the crest is desirable to provide a satisfactory condition of flow on that side of the channel at that point.

(e) That it is essential to place the beginning of the vertical curve well below the end of the spillway crest.

(f) That a weir with practically a vertical face across the channel at the beginning of the vertical curve is necessary to control the depth of water at the lower end of the channel and provide smooth flow in the transition and upper reaches of the inclined shaft.

(g) That the curve from the channel to the inclined shaft must be flatter than theoretical considerations would dictate in order to provide satisfactory condition of flow in the inclined shaft and vertical bend at the bottom of the shaft.

(h) That, while the above conditions will produce a satisfactory condition of flow, baffles in the channel produce a worthwhile improvement.

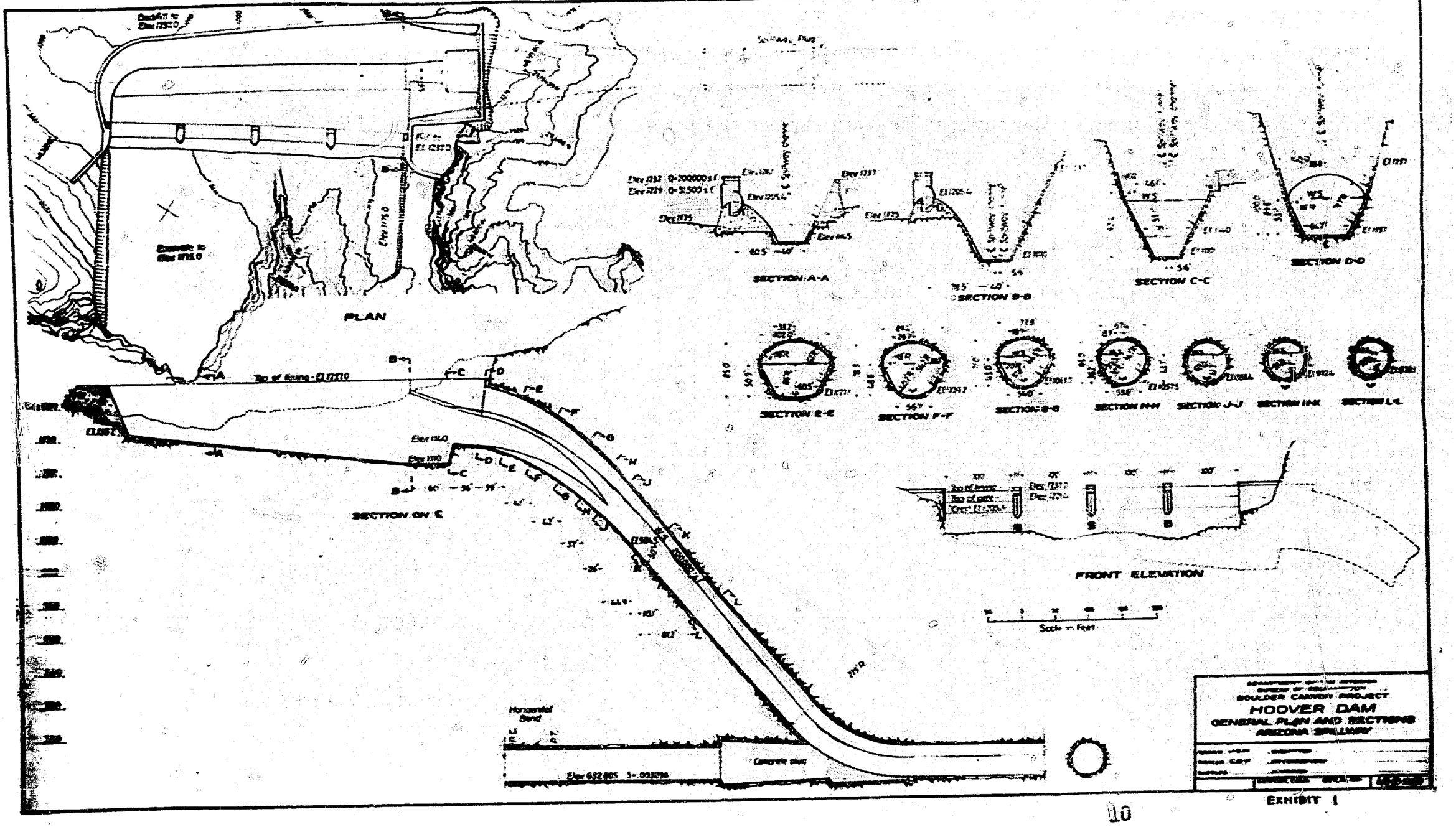

PLAN
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
SECTION F-F
SECTION G-G
SECTION H-H
SECTION J-J
SECTION K-K
SECTION L-L
SECTION ON E
FRONT ELEVATION
Scale in Feet
Horizontal Bend
Concrete Pipe
BOULDER CANYON PROJECT
HOOVER DAM
GENERAL PLAN AND SECTIONS
ARIZONA SPILLWAY

EXHIBIT I

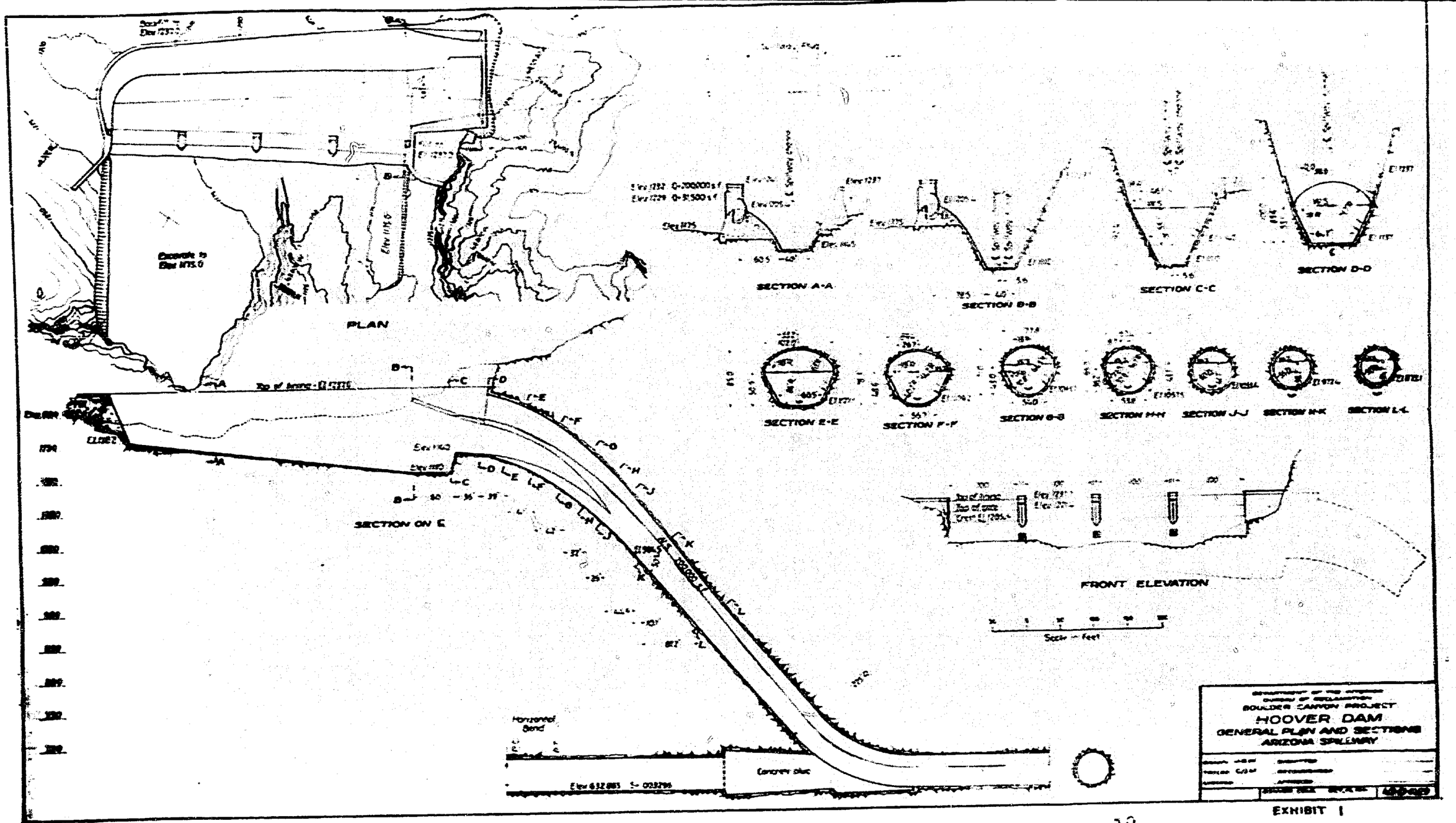

PLAN
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
SECTION F-F
SECTION G-G
SECTION H-H
SECTION J-J
SECTION K-K
SECTION L-L
SECTION ON E
FRONT ELEVATION
Scale - Feet
Horizontal Bend
Canyon disc
BOULDER CANYON PROJECT
HOOVER DAM
GENERAL PLAN AND SECTIONS
ARIZONA SPILLWAY

EXHIBIT I

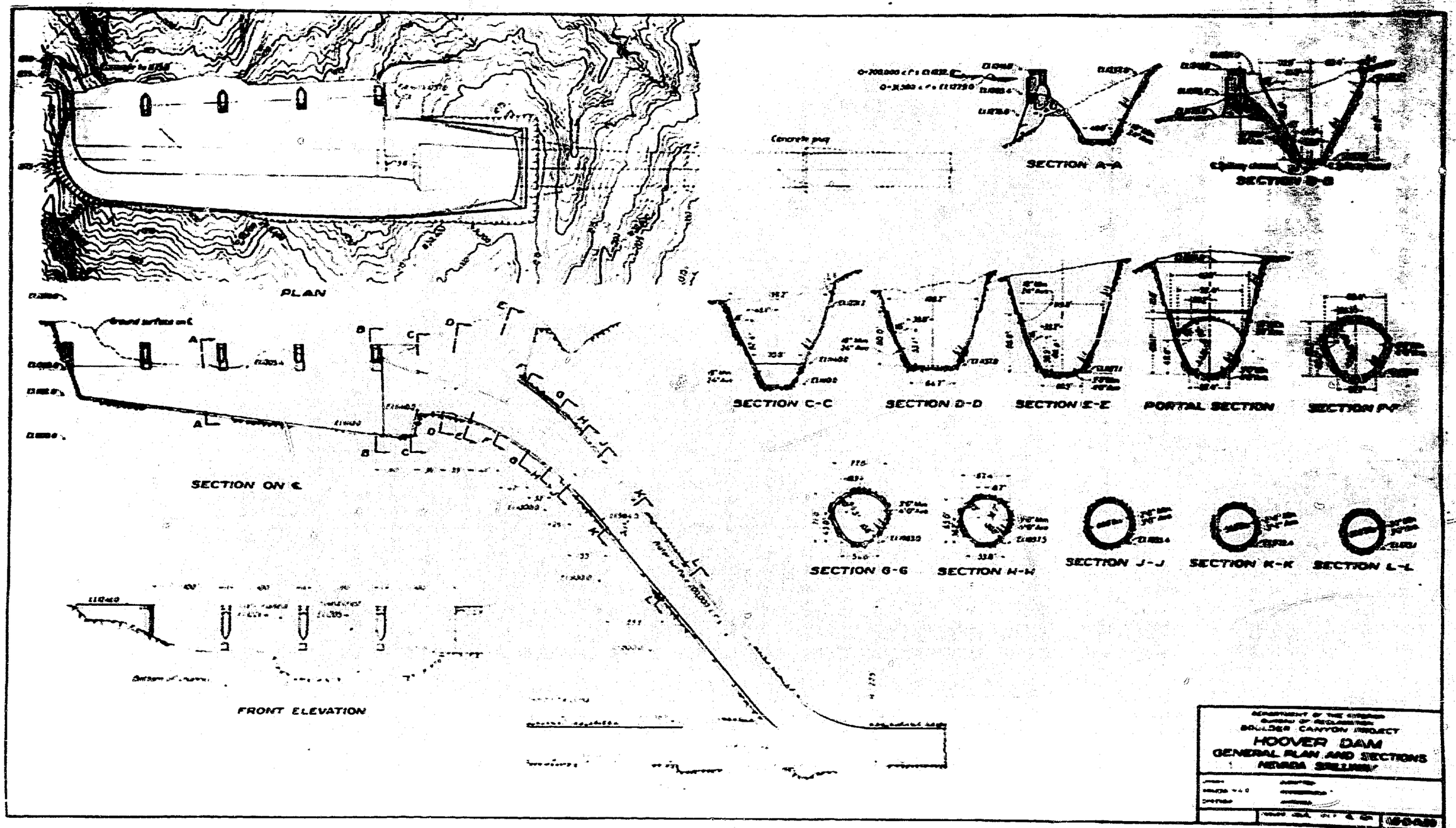
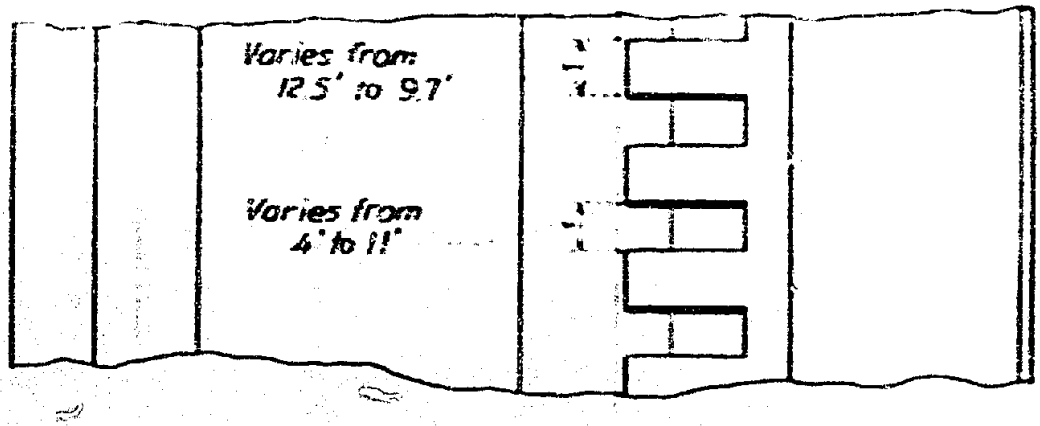
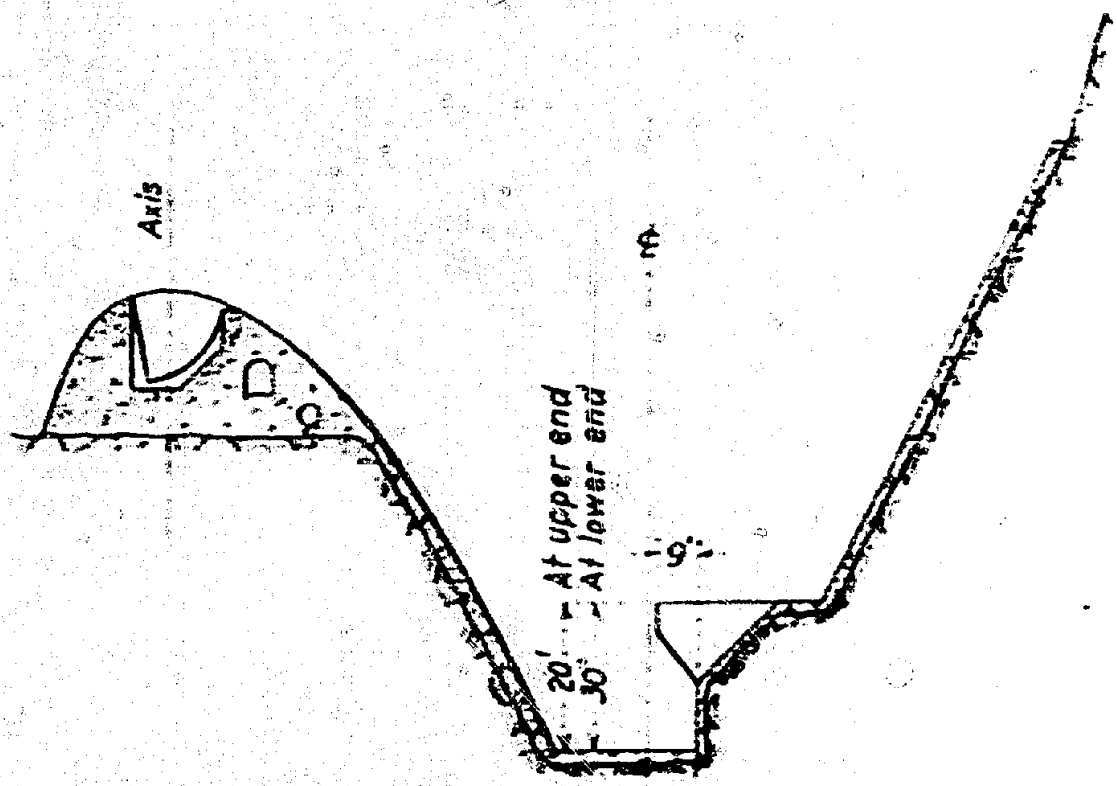

PLAN
SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
PORTAL SECTION
SECTION F-F
SECTION ON ℄
SECTION G-G
SECTION H-H
SECTION J-J
SECTION K-K
SECTION L-L
FRONT ELEVATION
BOULDER CANYON PROJECT
HOOVER DAM
GENERAL PLAN AND SECTIONS

EXHIBIT 2

EXHIBIT 3.



PLAN



- 5' - At upstream end of channel.
- 12' - At downstream end of channel.

TYPICAL CROSS-SECTION

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT - HOOVER DAM

CHANNEL BAFFLES AND SILL FOR DRUM GATE SPILLWAYS

DESIGN H.L.P. SUBMITTED
TRACED O.C.B. RECOMMENDED
CHECKED APPROVED

Denver, Colo. Oct. 6, 1931 | 45-D-1446

EXHIBIT 4

43-A-0. Set-up.

43-A-5. Flow 40,000 c.f.s.

43-A-3. Flow 120,000 c.f.s.

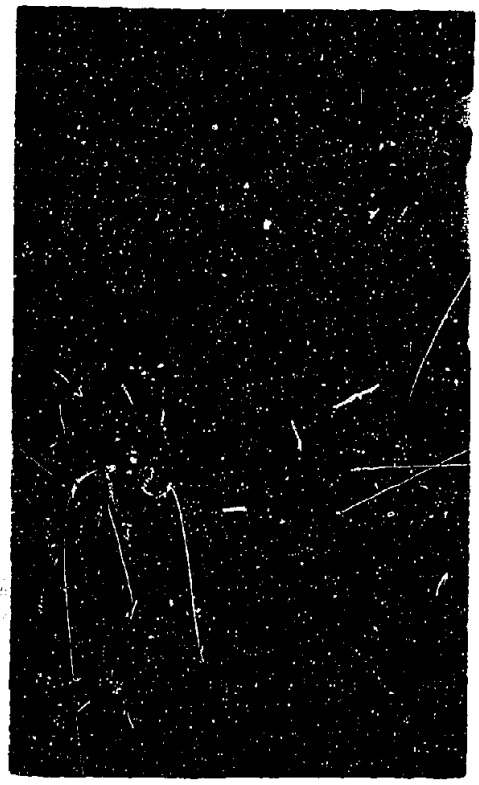
43-A-1. Flow 200,000 c.f.s.

Model of Design Shown by Exhibits 1 and 2.
(Fort Collins - Scale 1:60)

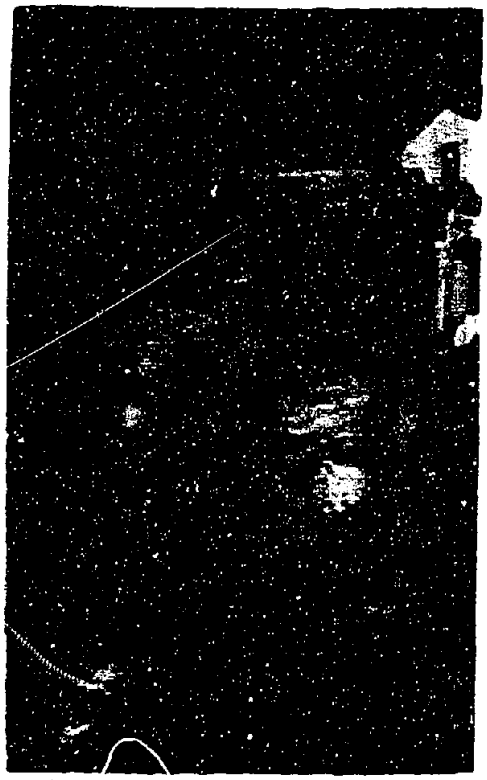
PLATE 1.

44-A-0. Set-up.

44-A-5. Flow 40,000 c.f.s.



44-A-3. Flow 120,000 c.f.s.



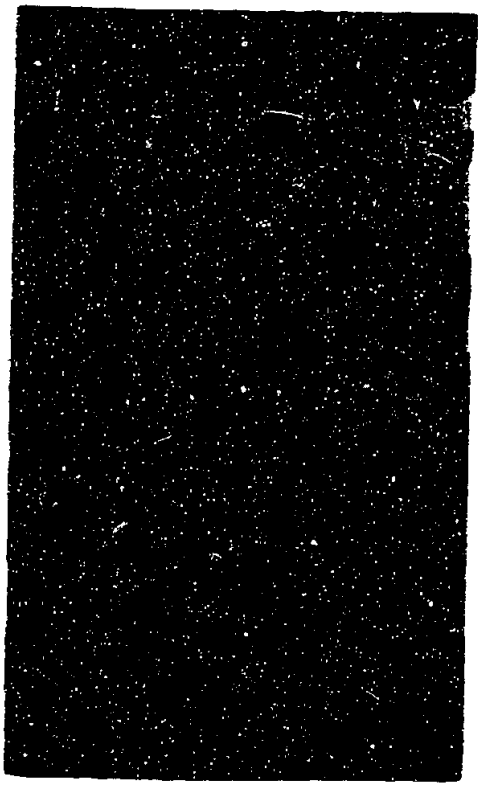
44-A-1. Flow 200,000 c.f.s.

Model of Design Shown by Exhibits 1 and 2.
Modified by Baffles in Channel
(Fort Collins - Scale 1:60)

PLATE 2.

44-A-0. Set-up.

44-A-5. Flow 40,000 c.f.s.



44-A-3. Flow 120,000 c.f.s.

44-A-1. Flow 200,000 c.f.s.

Model of Design Shown by Exhibits 1 and 2.
Modified by Baffles in Channel
(Fort Collins - Scale 1:60)

PLATE 2.

TABLE II

HOOVER DAM

COST AND QUANTITY ESTIMATE

SIDE CHANNEL SPILLWAYS

EXHIBITS 1 AND 2.

No.	Item	Cont. Cost	U. S. Cost	Nevada Quantity	Nevada Cost	Arizona Quantity	Arizona Cost
12	Excavation, open cut	$2.60	$ --	305,555	$ 794,443	282,650	$ 734,890
13	Excavation, tunnel	8.20	--	51,959	426,064	66,844	548,121
39	Drilling & grouting for anchor bars	1.00	--	9,490	9,490	7,946	7,946
58	Concrete crest	11.35	3.65	21,650	324,900	20,500	308,[illegible]
59	Concrete lining	10.00	3.65	12,016	164,019	8,881	121,[illegible]
56	Concrete dam	10.00	3.65	810	11,056	13,559	185,067
57	" tunnel lining	10.00	4.30	10,831	154,883	13,120	187,616
73	Reinforcing bars	0.0075	0.03	2,116,000	79,350	2,353,678	88,263
74	Anchor bars	0.02	0.03	61,100	3,055	51,450	2,572
—	Drum gates				277,200		277,200
	TOTAL FIELD COST				$2,244,460		$2,461,301

DETAIL REPORT

15. Geological examinations. The spillway sites were examined by Dr. F. L. Ransome, consulting geologist. Of the Nevada site, Dr. Ransome said:

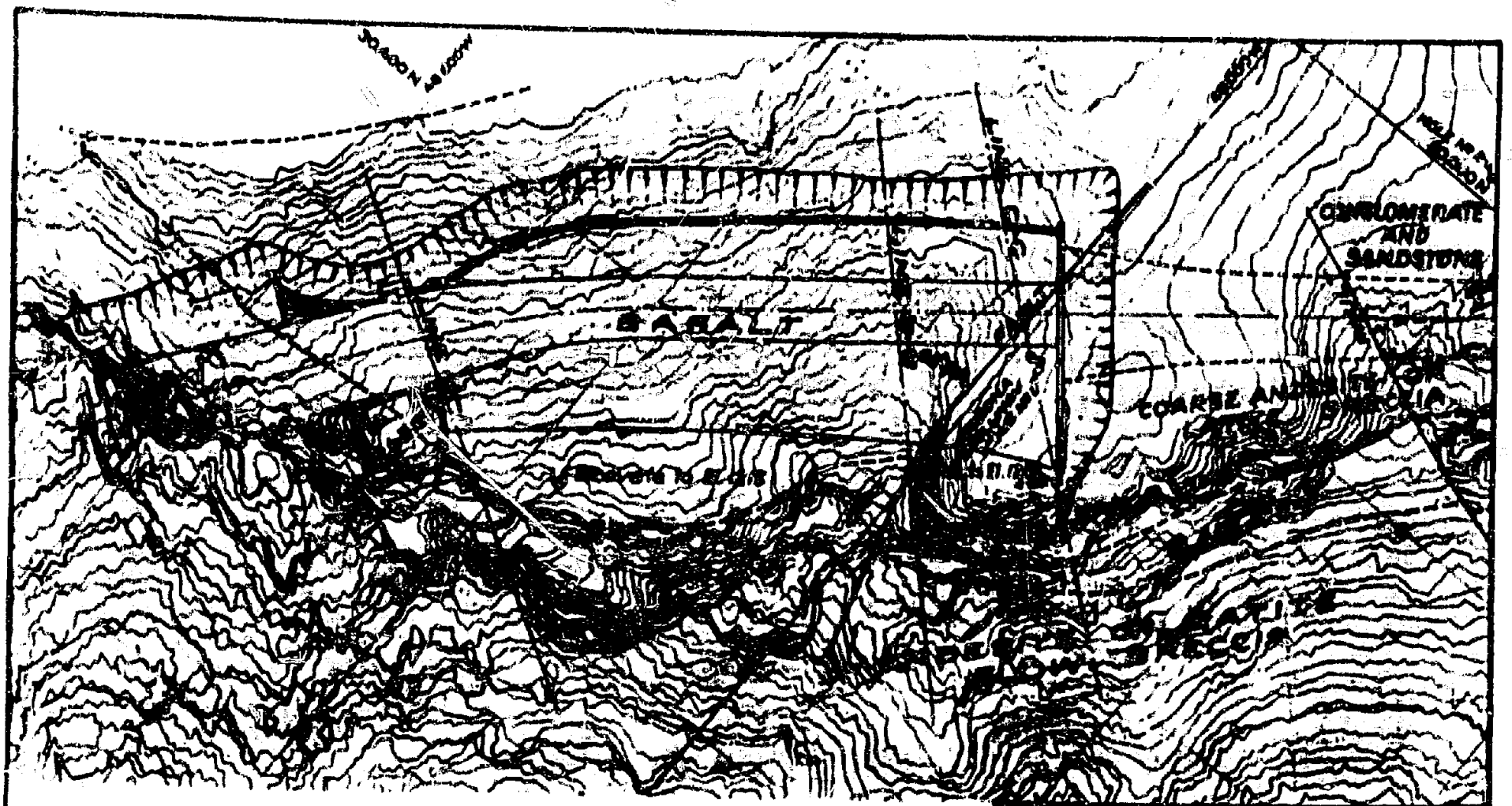
"The spillway on the Nevada side presents no geological difficulties whatever and will be in hard, strong rock, practically all in latite flow breccia."

With regard to the location on the Arizona side, shown in Exhibit 5, Dr. Ransome says, in his report of February 14, 1931:

"The geology was mapped in a general way (Exhibit 5) on the topography and some sections were made (Exhibit 6). The spillway will cross two faults, but these, like other faults in the canyon, have long since ceased to move, contain no soft gouge, and should cause no trouble. The general dip of the breccias and of some sedimentary beds associated with them is southeasterly. At the south end of the spillway the dip is from 15° to 20°. Near the north end it is about 40°."

In his report of March 30, 1931, he says:

"The geological structure is much more complex than I had anticipated. Faults are numerous and in many places difficult to trace owing to the topographic conditions and the lack of contrast between the various rocks involved. Fortunately the faults are all old and contain no soft gouge. The original crushed rock along them has been cemented to rock that is pra-



Note.- Geology from sketches by Dr F.L. Ransome with report of February 14, 1931.

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT

HOOVER DAM

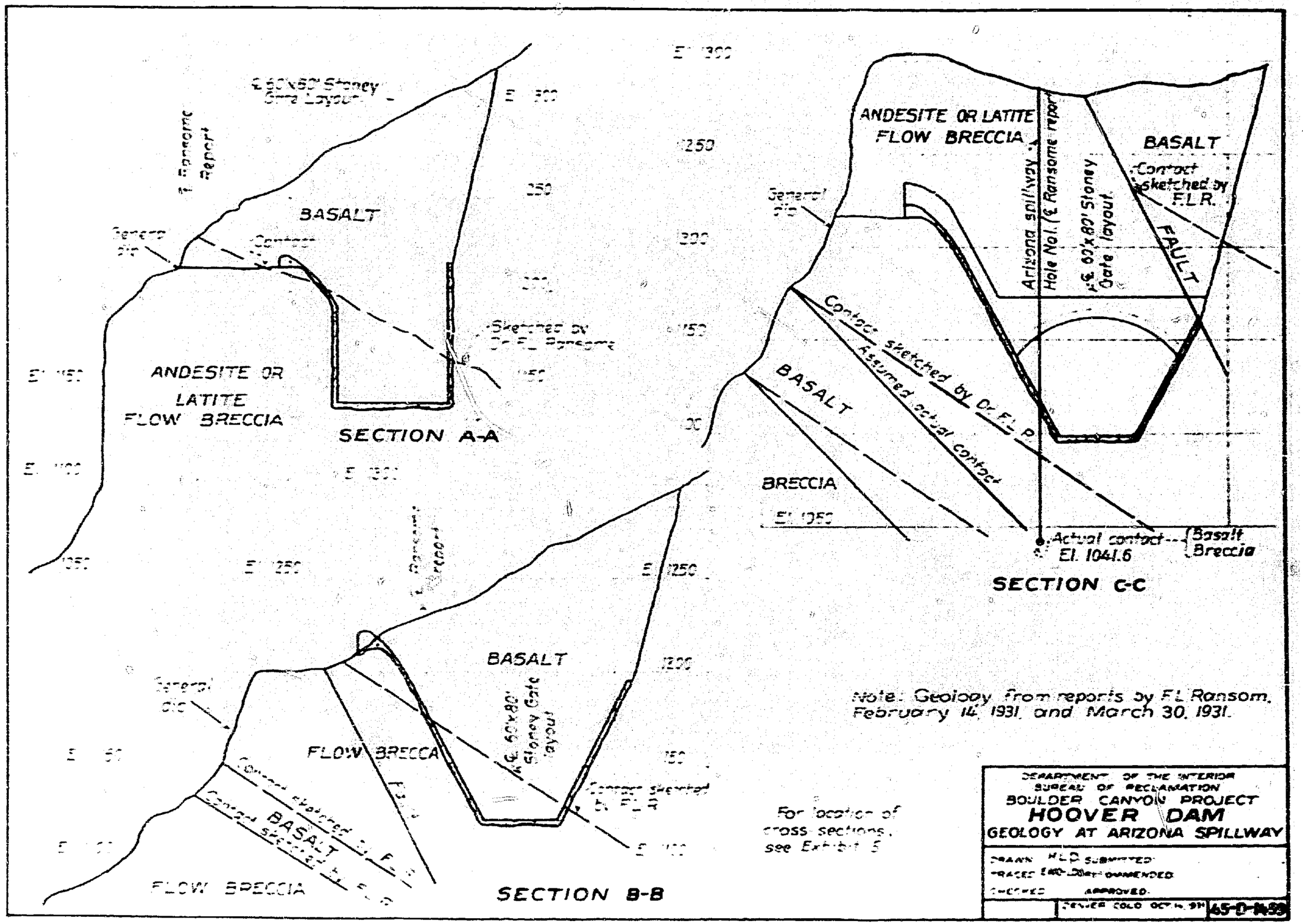
GEOLOGY AT ORIGINAL ARIZONA SPILLWAY SITE

DRAWN: H.L.D.	SUBMITTED
TRACED: PMW	RECOMMENDED
CHECKED	APPROVED

DENVER, COLO. OCT. 16, 1931

EXHIBIT 6

EXHIBIT 5



BASALT
ANDESITE OR LATITE FLOW BRECCIA
SECTION A-A
Sketched by Dr. F.L. Ransome
BASALT
FLOW BRECCA
BASALT
FLOW BRECCIA
SECTION B-B
ANDESITE OR LATITE FLOW BRECCIA
BASALT
Contact sketched by F.L.R.
FAULT
Arizona spillway
Hole No1. (℄ Ransome report)
℄ 60'x80' Stoney Gate layout
Contact sketched by Dr. F.L.R.
Assumed actual contact
BASALT
BRECCIA
Actual contact El. 1041.6
Basalt
Breccia
SECTION C-C
Note: Geology from reports by F.L. Ransom, February 14, 1931, and March 30, 1931.
For location of cross sections, see Exhibit 5
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM
GEOLOGY AT ARIZONA SPILLWAY
DRAWN H.L.D. SUBMITTED
TRACED
CHECKED APPROVED
DENVER COLO. OCT. 14, 1931

EXHIBIT 6

tically as hard as the unfractured rock of the fault walls. No fault was seen that is likely to give any trouble in connection with dam, spillways or tunnels."

"Basalt is more abundant than had been supposed. This rock is plainly intrusive and is likely to be found almost anywhere in drilling. It has been injected along faults, and in places forms irregular bodies of considerable size."

"The main question with reference to the Arizona spillway is whether the rock which will form the east side of the spillway will be strong enough to withstand the pressure of a full reservoir against an empty spillway. Concerning the rock which in my report of February 14 I called 'coarse andesite or latite breccia,' I have no misgivings. The material is strong and the bedding planes, although they dip into the spillway, are of such character as to render slipping along them highly improbable. The most doubtful feature of the weir rim is the presence of the basalt. Whether the upper surface of this basalt is even and lies in general conformity with the overlying coarse breccia is not at present determinable. If this contact is a smooth plane and emerges in the spillway a short distance above the bottom of the trench, there might be more danger of the whole mass of the weir rim slipping into the spillway along this contact than along the visible bedding planes in the coarse breccia."

In a letter dated June 25, 1931, Dr. Ransome says:

"I should like to see the contact as determined by hole No. 1 plotted on section C-C (Exhibit 6) of my original report. The easterly inclination of the rock would have to be steep, certainly over 45 degrees, to arouse any apprehension of a sliding of the rock forming the spillway wall into the spillway cut."

It will be noted on section C-C, that if the contact is a straight line from the point of contact in the drill hole to the visible contact on the face of the cliff, the angle is about 45 degrees.

The construction engineer was then notified to consult with Dr. Ransome concerning the present location of the spillway. In telegram of June 29, 1931, he advised that this site would be in volcanic breccia upstream and flow breccia downstream of a fault in the draw. This fault crosses the spillway near the lower end. Dr. Ransome stated that if the spillway were moved about 150 feet upstream from its present location, the crossing of the fault would be avoided; that he did not regard the presence of the fault as a serious objection, but would prefer to avoid it if this did not entail decided disadvantages in other respects; that no great expense should be incurred to avoid this fault.

Since heavy expense would be involved in avoiding this fault, and since, if the spillway were placed as suggested by Dr. Ransome, the fault would still be crossed by the shaft, the present location was adopted.

Detailed logs of the holes are shown on the following pages; the locations of the holes are shown on Exhibit 5. The crest elevation of the spillway was at elevation 1221.3 and the bottom of the channel from elevation 1099 to elevation 1141. The "remarks" in the log concerning the rock over this range are not such as to inspire confidence.

BLACK CANYON D (HOOVER) DAMSITE

P. & W. Co-Ordinates, N. 29,963, W. 45,184.

Hole #2,
Arizona Spillway.

Elevation at Collar, 1265.0

DRILLING RECORD: ELEVATION From	To	DEPTH BELOW COLLAR-FEET From	To	Classification.	Sea notes for core diameter FEET	%	REMARKS
1265.0	1228.0	0	37	Breccia	27.5	74.3	
1228.0	1217.0	37	48	"	9.5	86.4	Broken & seamy. Lost water at 42 feet. Cemented hole & lost water again at 47 feet.
1217.0	1184.0	48	81	"	23.5	71.2	
1184.0	1170.0	81	95	"	9.5	67.8	Broken. Lost water at 100.
1170.0	1151.5	95	113.5	"	7.5	40.5	Cemented hole & lost it again at 113.5.
1151.5	1124.0	113.5	141.0	"	14.0	50.9	Fairly solid.
1124.0	1110.0	141.0	155.0	"	4.5	32.1	Soft & broken. Lost water at 146 ft. Cemented hole and recovered water.
1110.0	1098.0	155.0	167.0	"	5.5	45.8	Fairly solid.
1098.0	1088.5	167.0	176.5	"	3.5	36.8	Soft and broken.
1088.5	1073.0	176.5	192.0	"	7.0	45.2	
1073.0	1013.0	192.0	252.0	"	19.0	31.7	Soft & broken.
1013.0	1008.0	252.0	257.0	Basalt	5.0	100.0	Contact at 252.0. Tight seam. No loss of water.
		Total core recovery			134.0	52.9	

NOTES:-

This hole was drilled with "bx" equipment to depth 60 feet in the endeavor to secure more core, and was cased with "E" casing to 60-foot depth. Below the 60-foot depth drilling was carried on with "E" equipment. Size of core 15/16" diameter.

Cores were examined by Dr. F. L. Ransome and Field Engineer Lowry on June 25, 1931. Contact between breccia and basalt was determined by Dr. Ransome after personal examination.

[illegible] - D (Hoover) Diversion

[illegible] Co-ordinates, N. 33,272, W. 49,012.

Hole #1,
Arizona Spillway. Elevation at Collar, 1279.6.

Drilling Rate Ft. Elevation From	To	Depth Below Collar From	To	Classification	0.55 Lb/In² Pressure Rate Feet	Gal.	Remarks
1279.6	1232.6		47	Dolerite	22.0	47.6	Soft & broken core grinds up.
1232.6	1209.6	47	71	Diabase	19.0	79.1	
1209.6	1194.6	71	86	"	16.5	76.6	Soft & broken vertical seams.
1194.6	1181.6	86	99	Breccia	7.0	62.8	Solid but quite soft.
1181.6	1179.6	99	[illegible]	"	[illegible]	2[illegible]	Hard but broken.
1179.6	1166.1	[illegible]	[illegible]	"	12.17	91.1	Hard and solid.
1166.1	1161.6	[illegible]	12[illegible]	"	[illegible]	[illegible]	Soft & seamy. Lost water 121. to 127. Had a little [illegible] at 127. From 127 to [illegible] soft breccia of green-ish color. Recovered 5 inches of core.
[illegible]	114[illegible]	[illegible]	[illegible]	"	[illegible]	44.1	Had return water 128 to 130. Lost water at 132. No [illegible] evident.
[illegible]	[illegible]	[illegible]	[illegible]	"	[illegible]	57.1	Soft and broken but setting harder. No change in formation.
[illegible]	[illegible]	[illegible]	[illegible]	"	1.02	[illegible]	[illegible] solid.
[illegible]	[illegible]	[illegible]	[illegible]	"	[illegible]	49.5	Soft and broken.
[illegible]	[illegible]	[illegible]	[illegible]	"	[illegible]	[illegible]	Solid.
[illegible]	[illegible]	[illegible]	[illegible]	"	[illegible]	[illegible]	Soft and broken.
[illegible]	[illegible]	[illegible]	[illegible]	"	[illegible]	61.8	Solid.
[illegible]	[illegible]	[illegible]	[illegible]	"	[illegible]	[illegible]	Core material is somewhat harder in color but same [illegible] as above.
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	Soft and broken.
[illegible]	1020.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
		Total [illegible]			14.9	[illegible]	

Remarks:

Hole was operated from 13[illegible] feet to 14[illegible] feet on account of losing water. No water lost after drilling out cement.

Contact between the breccia and the basalt was reached at a depth of 1[illegible] feet in the hole. Contact identified and depth determined by [illegible]. Found one hole. This hole was drilled with "D" equipment.

1[illegible]. Use of gates. It is not practicable, without the use of gates, to provide the [illegible] increase in capacity required ([illegible] second-feet) [illegible] -foot rise in water surface allowable.

2[illegible]. Stoney Gate with side channel. The following table shows various combinations, all of which provide the required capacity with water surface at elevation 1232.

TABLE III

Designs for Side Channel with Stoney Gate.

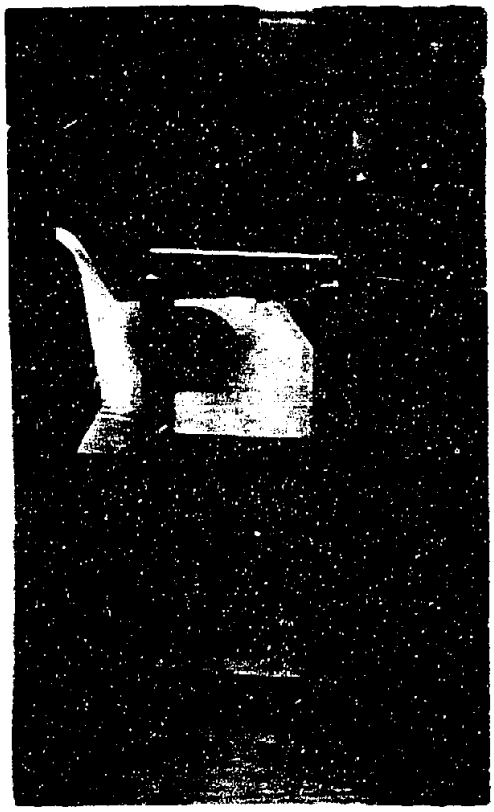
Size Gate	Length: Fixed Crest	1 gallery - gate up: Crest Elev.	Gate Sill Elev.	T.S. at 1232 Q Gate	Q Crest	2 gallery - gates down: Q for T.S. 1232	T.S. Elev. for Q = [illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1226.1
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1227.2
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1227.[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	1227.[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	(1)	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

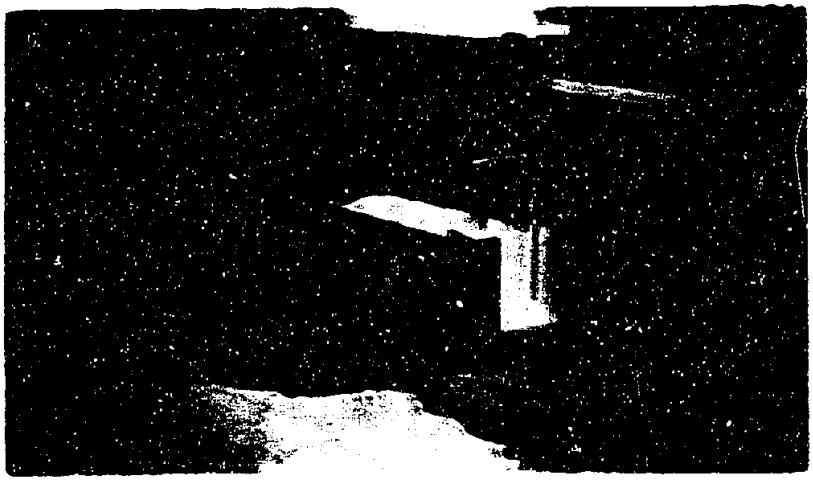
(1) Thousand Exhibit [illegible].

The combination of [illegible] by [illegible] -foot fixed crest [illegible] (Exhibit 3). For economic [illegible] at the elevation of the fixed crest. Assuming that the [illegible] main closed until the water surface [illegible] elevation 1232, the discharge over each uncontrolled crest [illegible] second-feet. At maximum capacity,

with gate fully raised, the discharge through each gate would be 14[illegible],000 second-feet and over each fixed crest [illegible],000 second-feet. The water was assumed to go through the gate at the depth corresponding to maximum discharge. This requires a considerable drop in water surface between the lake and the gate, resulting in a low water surface in the channel, which results in considerable energy of cross flow of the water entering over the fixed crest. A higher water surface either would reduce the discharge through the gate, or would depend on recovery of a considerable part of the velocity head in the side channel. A high percentage of recovery would be expensive on account of the [illegible] velocity and the probable effect of the inflow over the fixed crest. A reduced discharge through the gate would require either a longer gate, which already greatly exceeded precedent in size; or a longer crest which would greatly increase the quantity of excavation required on account of the unfavorable topography, or it would be advisable to curve the channel to fit the ground contours; [illegible] confirmed by the model tests.

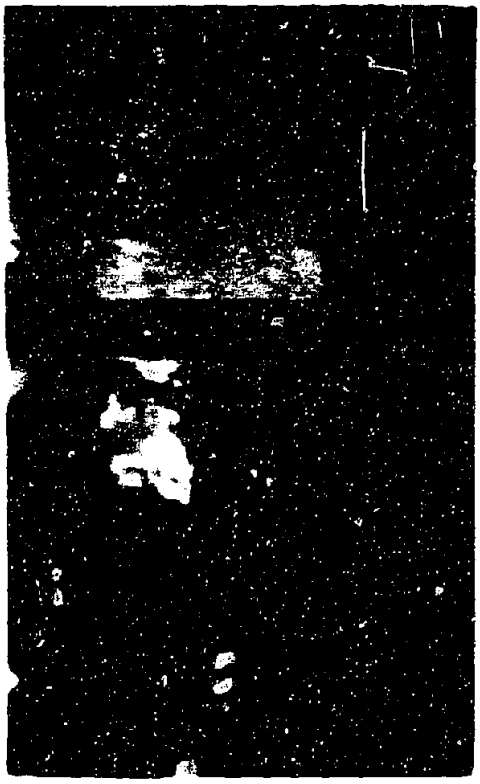
[illegible]. Model tests. A model of this design to a scale of 1:[illegible] was built and tested at Fort Collins. The transition, down to the [illegible] the [illegible]-foot shaft was of laminated wood. The shaft [illegible], the vertical bend, and the tunnel below were of transparent [illegible]. It was found that a wave formed on the [illegible] the curve below the gate, which was reflected [illegible] across the channel striking the right side again at the [illegible]. [illegible] of the model and its performance [illegible]





Model of Design Shown in Exhibit 3.
(Fort Collins - Scale 1:60) PLATE 3.

Discharge 110,000 sec.-ft.



Discharge 110,000 sec.-ft.

Discharge 160,000 sec.-ft.

Discharge 160,000 sec.-ft.

Performance of Design Shown in Exhibit 3.
(Fort Collins - Scale 1:60)

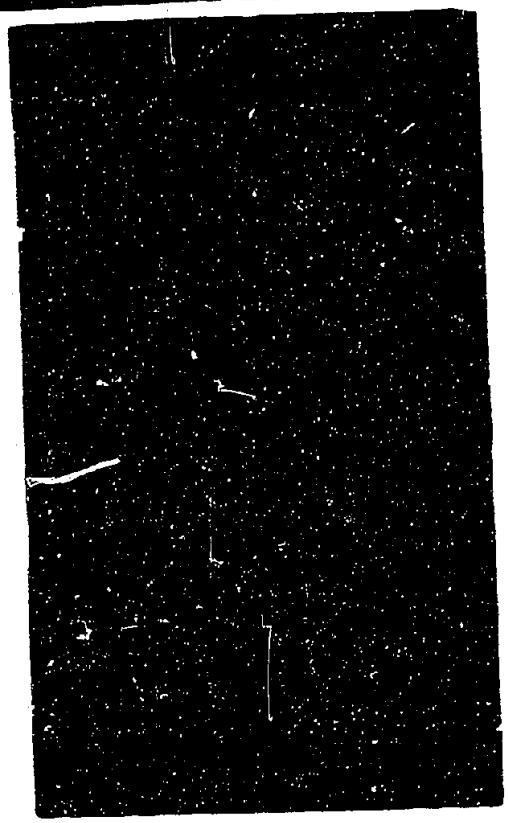
PLATE 4.

Discharge 200,000 sec.-ft.

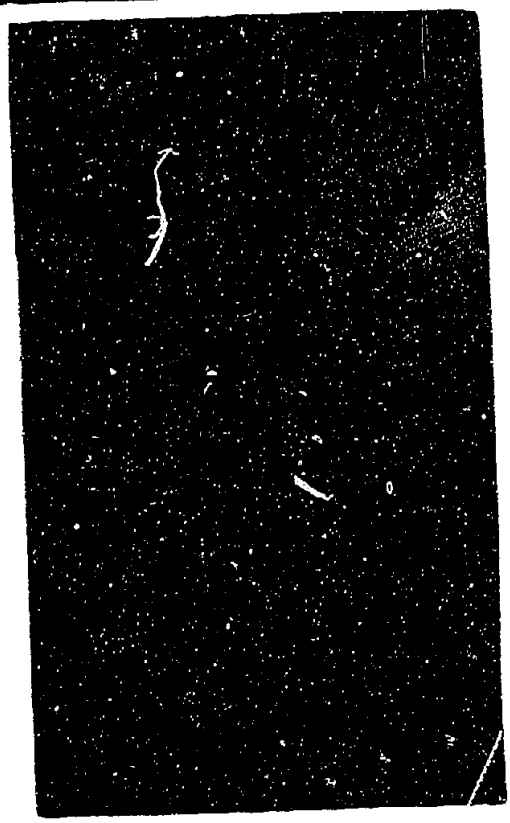
Discharge 200,000 sec.-ft.

Performance of Design Shown in Exhibit 3.
(Fort Collins - Scale 1:60)

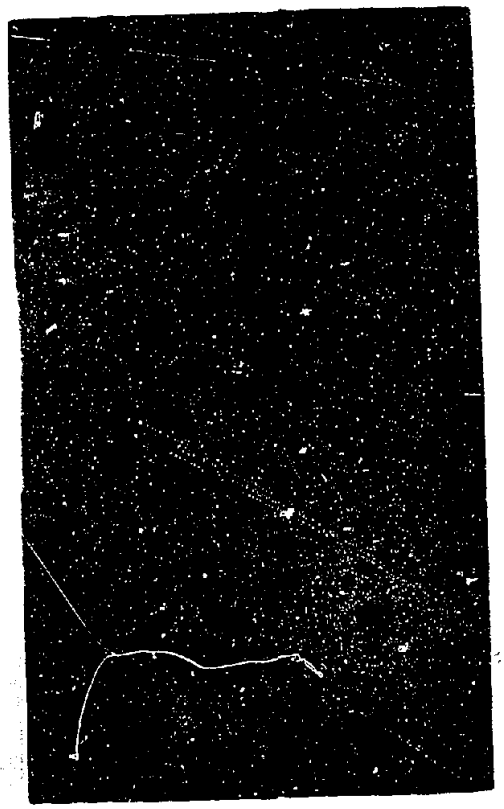
PLATE 5.



Q = 55,000 Over Crest Only.



Q = 60,000 Under Gate Only.



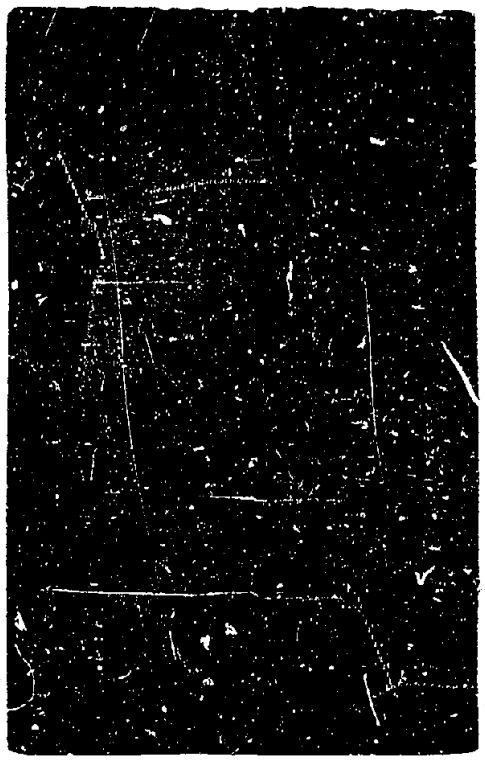
Q = 190,000 Flow Through Gate Only.

Performance of Design Shown in Exhibit 3.
Abnormal Conditions of Discharge.

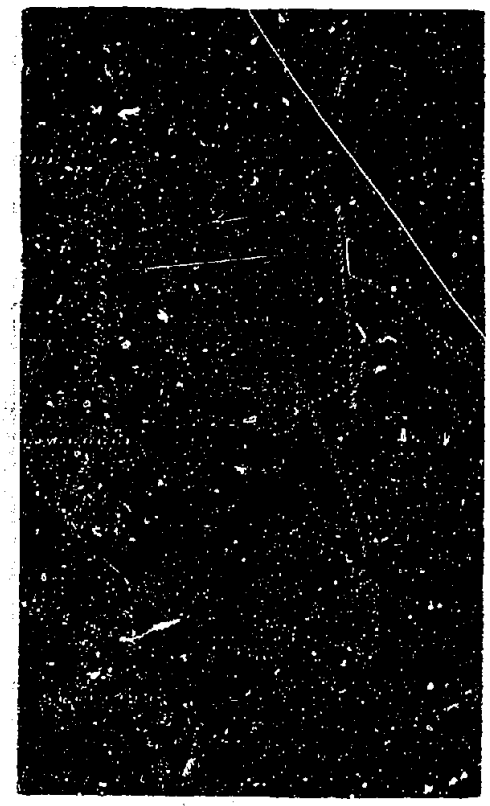
PLATE 6

shown on Plates 3 to 6. The effect of the curve below the gate and of the water coming over the open crest combined to form rough flow in transition and shaft. At maximum capacity the shaft appeared completely choked below the portal; although a removable section had been left in the crown of the transition for observation purposes, it proved useless as the splash through it when opened was sufficient to drench an observer and little could be seen. The portion of the shaft below the end of the transition was removed; it was then apparent that the transition was really running at about its designed depth, and that the appearance of choking was due to a heavy spray. Plate 7 shows the appearance of the jet, and, for comparison, the appearance when the entire flow was forced through the gate alone. In these views it is not possible to distinguish spray from water. The tendency toward spiral flow, which is due to the effect of the water entering over the fixed crest, is absent in the latter case. It was impossible to measure water surfaces below the portal, but visual observations at the lower end of the transition, at the vertical bend, and at the outlet end of the tunnel indicated that an average water surface at these points would agree very closely with that computed.

The tests indicate that the combination of a small Stoney gate with a fixed crest is not a desirable type of spillway. The inflow over the fixed crest causes losses and disturbance out of proportion to its magnitude. It is preferable to rely on the Stoney gate alone; in the present case, however, this would require a gate about 60 by 100 feet.



Designed Conditions.
Discharge through gate
145,000 sec.-ft.
Discharge over crest
55,000 sec.-ft.



Discharge 200,000 sec.-ft.
Through gate only.

Model of Design Shown on Exhibit 3.
Appearance of Jet at Lower End of Transition.
(Fort Collins - Scale 1:60) PLATE 7.

23. Drum gate spillways. The drum gate crest appears to be best suited to the conditions at Hoover Dam, because it can be placed on the steep sides of the canyon with the channel parallel to the general direction of the contours on the river, and, for the length chosen, can be made straight with a moderate amount of excavation.

The gates will act as an uncontrolled crest up to the discharge of 75,000 second-feet in the river (63,300 over both spillways). This discharge will be reached at elevation 1229 and the head on the gate crest will be 7.6 feet. At this stage, the gates would begin lowering and the discharge would increase to about 335,000 second-feet when the gates were completely lowered, with only enough rise in water surface to provide head for automatic operation (to about elevation 1220.5) or with no rise in water surface if operated manually. The increase to the required discharge of 400,000 second-feet would then be accomplished by a rise in the water surface to elevation 1232.

25. The design proposed contemplates the use of four 16 by 100-foot gates for each spillway, with the usual arrangements for either automatic or manual control. These gates are the same size as those to be used for the Hidden Dam. They exceed the gates used in several spillways built by the Bureau and by private companies by only 18 inches in height. The length is greater than has heretofore been used, but as the gates are hinged at short intervals this is of no consequence. The long gate results in considerable economy,

due to saving in steel in the ends and the use of fewer piers, as well as minimizing the danger of clogging by drift. Closely spaced piers reduce the discharge for a given net length and increase the total length of channel required by more than the thickness of the additional piers. The length of channel selected is the longest well adapted to the topography.

24. The crest is designed to have a maximum sliding co-efficient of .65, assuming, with no backwater, full uplift over two-thirds of the base at heel, decreasing to zero at a point 5 feet from the toe, where drainage will be provided; or, assuming backwater, full uplift over two-thirds of the base at both heel and toe. Analyses of the section are shown by Exhibits 7 and 8. In no case will the foundation be higher than elevation 1175; this is the minimum section. On the Nevada side, the slope of the upstream face intersects the ground surface at about elevation 1138 at the lowest point; the bottom of the channel is about one foot higher. This is the maximum height of the crest.

25. Details of the weir crest, piers and control mechanism are shown by Exhibits 9, 10 and 11.

26. It has been suggested that the accidental dropping of one of these gates would materially increase the flow in the river. The maximum increase due to accidental dropping of one gate would be about 30,000 second-feet at a time when the total river discharge was 75,000 second-feet; that is, the river discharge would increase from 75,000 to 105,000 second-feet.

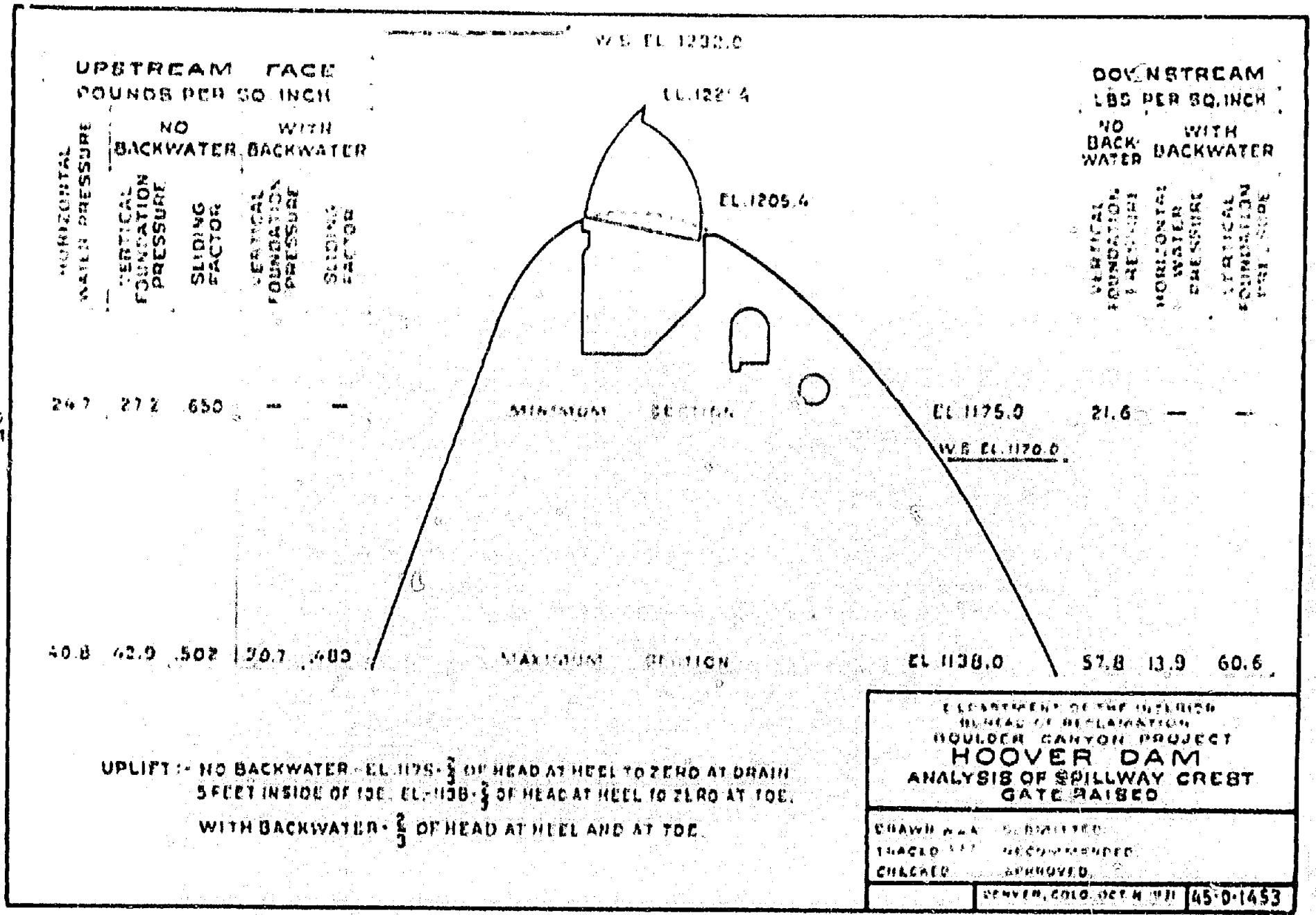

UPSTREAM FACE
POUNDS PER SQ. INCH
HORIZONTAL WATER PRESSURE
NO BACKWATER
VERTICAL FOUNDATION PRESSURE
SLIDING FACTOR
WITH BACKWATER
VERTICAL FOUNDATION PRESSURE
SLIDING FACTOR
W.S. EL. 1232.0
EL. 1221.4
EL. 1205.4
DOWNSTREAM
LBS PER SQ. INCH
NO BACKWATER
VERTICAL FOUNDATION PRESSURE
WITH BACKWATER
HORIZONTAL WATER PRESSURE
VERTICAL FOUNDATION PRESSURE
24.7 27.2 .650 — —
MINIMUM SECTION
EL. 1175.0
21.6 — —
W.S. EL. 1170.0
40.8 42.9 .502 30.7 .480
MAXIMUM SECTION
EL. 1138.0
57.8 13.9 60.6
UPLIFT :- NO BACKWATER - EL. 1175 - 2/3 OF HEAD AT HEEL TO ZERO AT DRAIN 5 FEET INSIDE OF TOE. EL. 1138 - 2/3 OF HEAD AT HEEL TO ZERO AT TOE.
WITH BACKWATER - 2/3 OF HEAD AT HEEL AND AT TOE.
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM
ANALYSIS OF SPILLWAY CREST
GATE RAISED
DRAWN
SUBMITTED
TRACED
RECOMMENDED
CHECKED
APPROVED
DENVER, COLO. OCT. 8 1931
45-D-1453

EXHIBIT-7

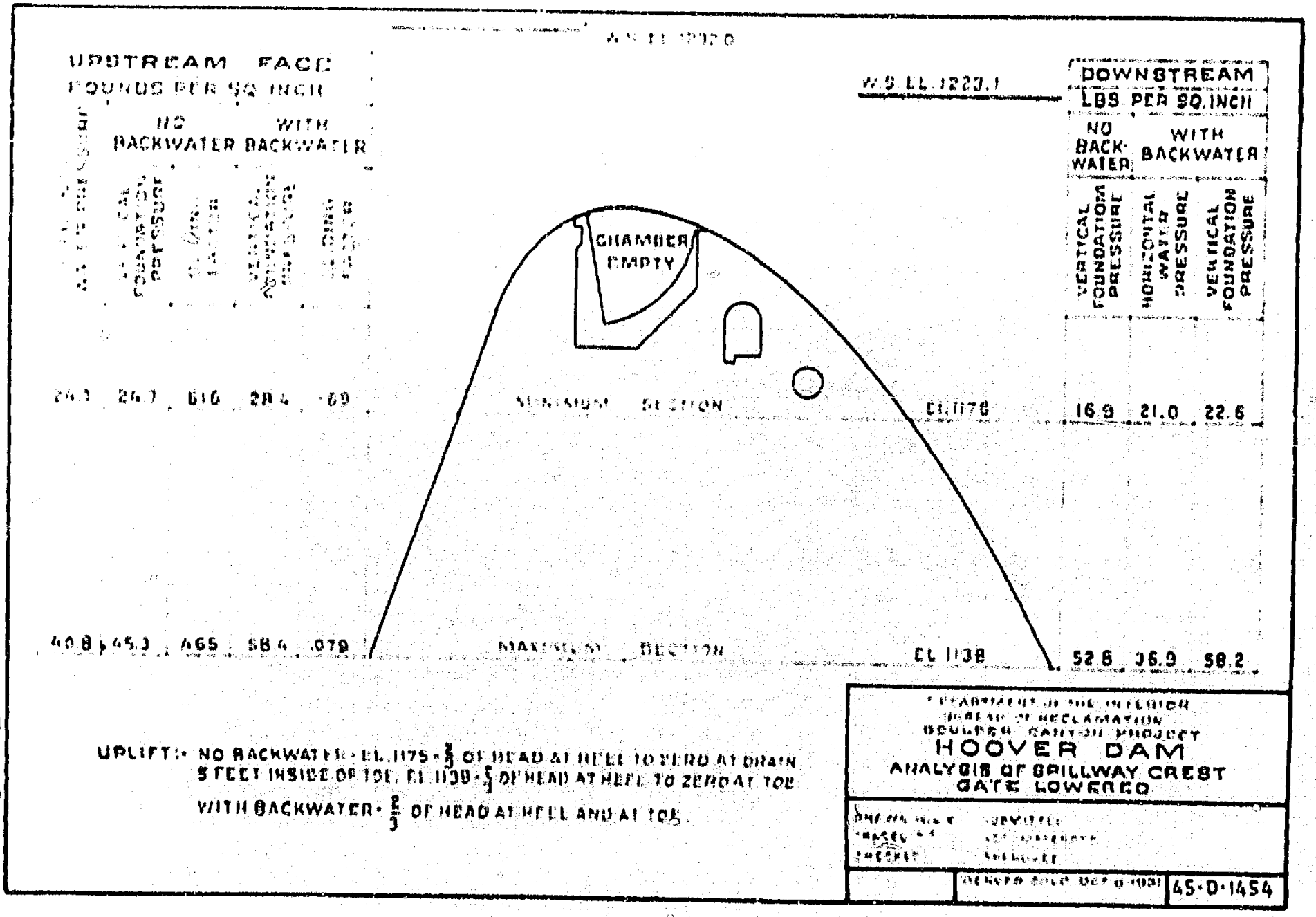

UPSTREAM FACE
POUNDS PER SQ INCH
NO BACKWATER
WITH BACKWATER
W.S. EL. 1223.1
DOWNSTREAM
LBS PER SQ. INCH
NO BACK-WATER
WITH BACKWATER
VERTICAL FOUNDATION PRESSURE
HORIZONTAL WATER PRESSURE
VERTICAL FOUNDATION PRESSURE
CHAMBER EMPTY
MINIMUM SECTION
EL. 1175
16.9
21.0
22.6
MAXIMUM SECTION
EL. 1138
52.8
36.9
58.2
UPLIFT:- NO BACKWATER - EL. 1175 - 2/3 OF HEAD AT HEEL TO ZERO AT DRAIN 5 FEET INSIDE OF TOE. EL. 1138 - 2/3 OF HEAD AT HEEL TO ZERO AT TOE
WITH BACKWATER - 2/3 OF HEAD AT HEEL AND AT TOE.
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM
ANALYSIS OF SPILLWAY CREST
GATE LOWERED
45-D-1454

EXHIBIT-8

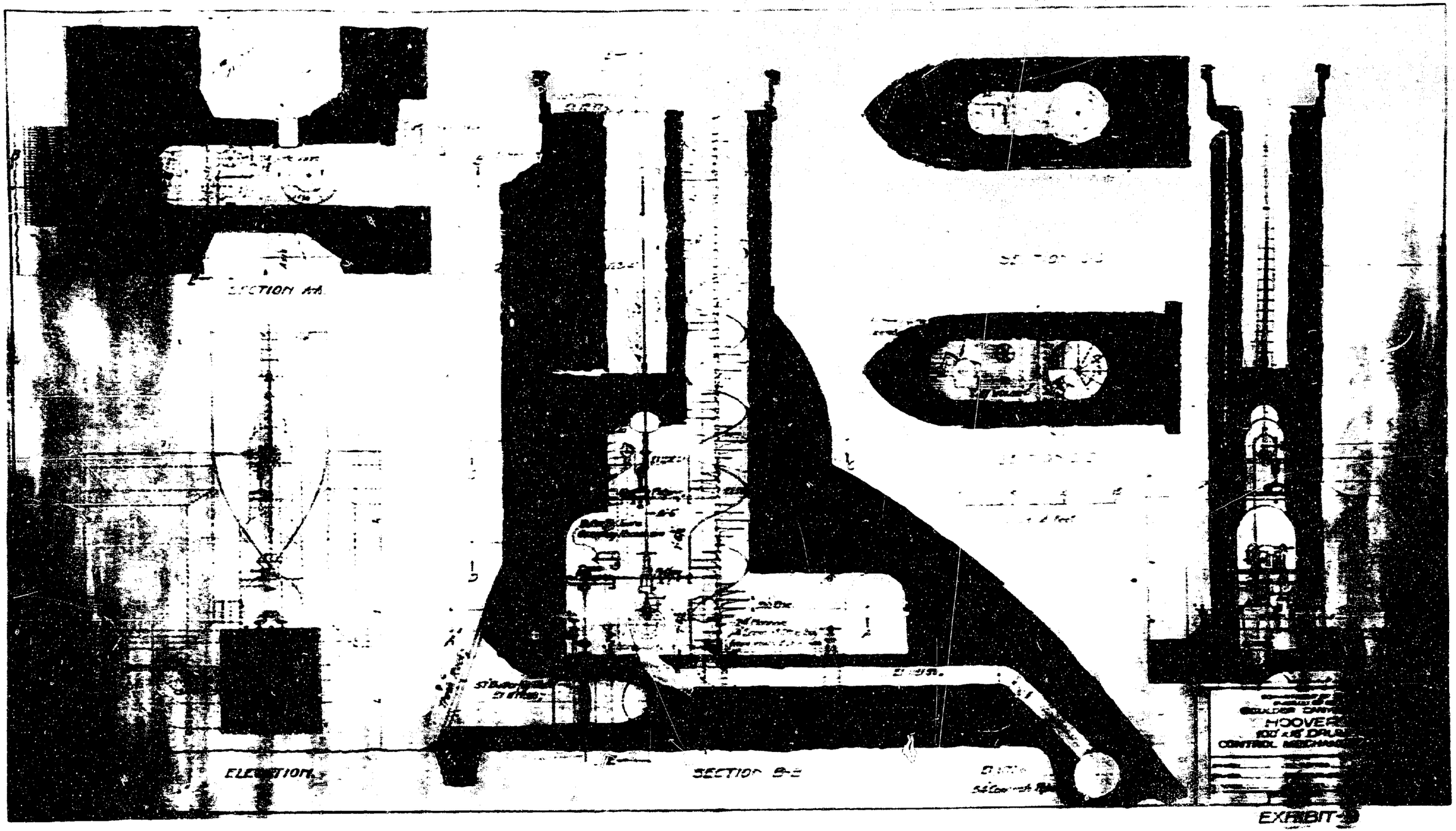


EXHIBIT 3

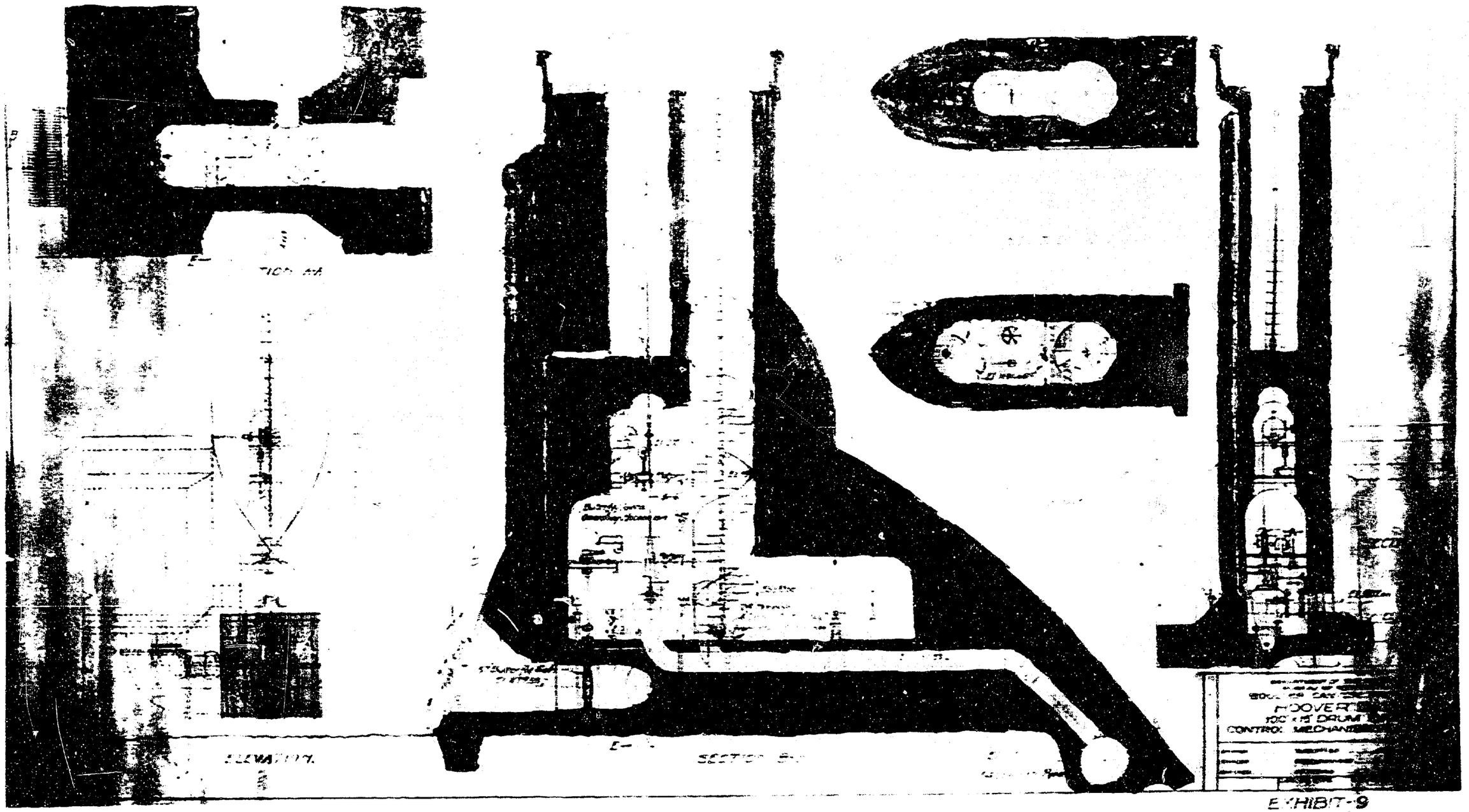


EXHIBIT-9

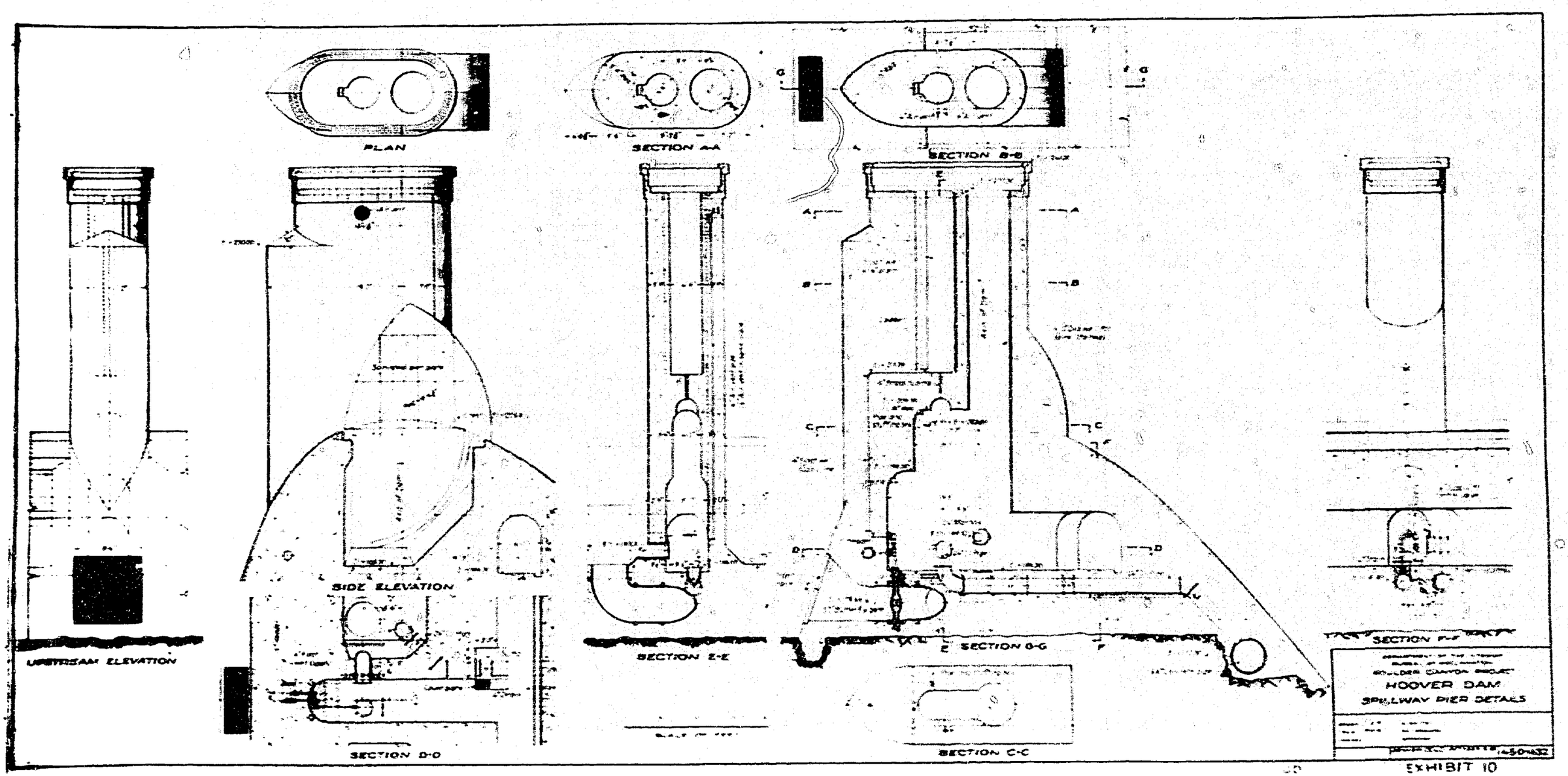

PLAN
SECTION A-A
SECTION B-B
UPSTREAM ELEVATION
SIDE ELEVATION
SECTION E-E
SECTION G-G
SECTION F-F
SECTION D-D
SECTION C-C
HOOVER DAM
SPILLWAY PIER DETAILS

EXHIBIT 10

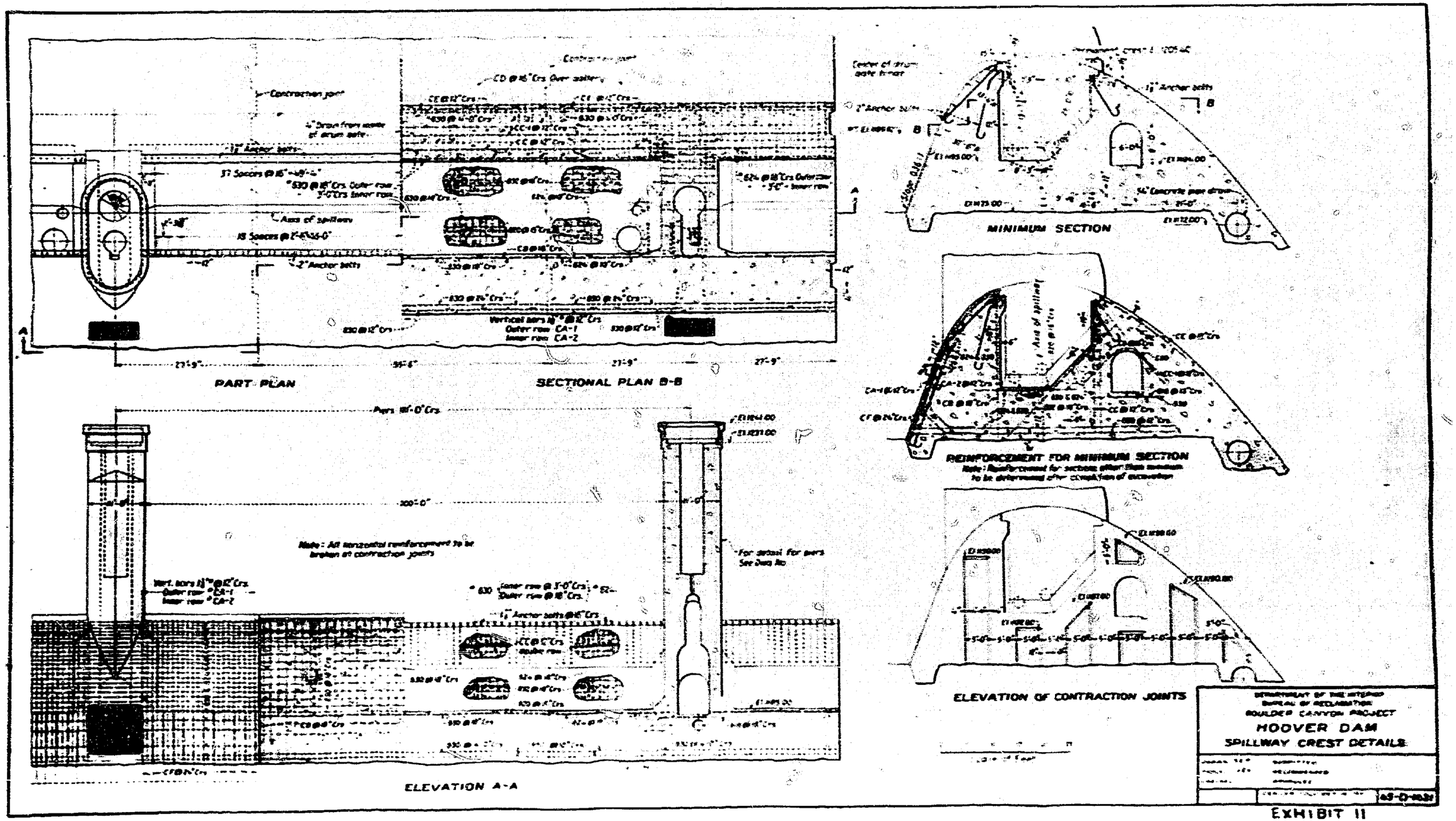

PART PLAN
SECTIONAL PLAN B-B
MINIMUM SECTION
REINFORCEMENT FOR MINIMUM SECTION
ELEVATION OF CONTRACTION JOINTS
ELEVATION A-A
Note: All horizontal reinforcement to be broken at contraction joints
HOOVER DAM
SPILLWAY CREST DETAILS

EXHIBIT II

27. However, with gates as now designed, there is no reason to expect such an occurrence. Accidental dropping has, in the past, been due either to failure of seals, failure of drain hose, or clogging of small pipes on which depended the operation of the balanced valve which controls automatic operation. The seals now used are made of metal, and are entirely different from those which failed. There is no more reason to anticipate failure of the present seals than of the gate itself. The design of the valves has been changed so that clogging is impossible. Past failure of drain hose was due to abrasion of the hose with the movement of the gate; this is now watched for when the gate is placed. While thought unnecessary, a check valve could easily be placed in the drain hose line to prevent water from entering the gate. Visible outlets will be provided for the drain hose, so that any increase in discharge can easily be seen.

28. Even should accidental dropping occur, it will be noticed almost instantly at the power house. The inlets to the gate chamber are very large enough so that the gates can be raised if the seals are omitted entirely; in case of trouble with the balanced valve, it can be shut off and the emergency valve used; and in the case of failure of the hose, temporary repairs can be made in a very short time. The gate could, therefore, be raised again so quickly that the peak in discharge would almost have disappeared by the time a point where it could cause damage had been reached.

29. The crest capacity has been computed from Francis' formula, using a coefficient of 3.65 for gates lowered and 3.75 for gates raised. These coefficients check the model experiments. In order to reduce the drop in water surface over the weir, the channel has been designed so that a considerable part of the crest is submerged; this decreases the coefficient of discharge of the crest by .06 according to tests at Fort Collins. The models both at Fort Collins and at Montrose have shown the same small effect on discharge and in this they agree with experiments by Cox at the University of Wisconsin (Bulletin [illegible]).

30. The first drum gate design which was prepared on the basis of minimum cost is shown by Exhibit 12. In order to reduce the size of tunnel required, the transition curve was begun in the channel, resulting in the marked increase in slope just above the portal shown on the drawing.

31. Model tests. A complete description of all model tests [illegible] very lengthy; as most of the results were of indirect value, [illegible] only as stepping-stones, only the salient features will be given in the following brief discussion.

32. A model [illegible] scale of 1:20 of the design shown by Exhibit 12 was built at Montrose. The excessive drop from lake to channel water surface resulted in a large energy of cross flow, which the water prior in the channel, on account of its high velocity and consequent shallow depth, was too small to dissipate satis-

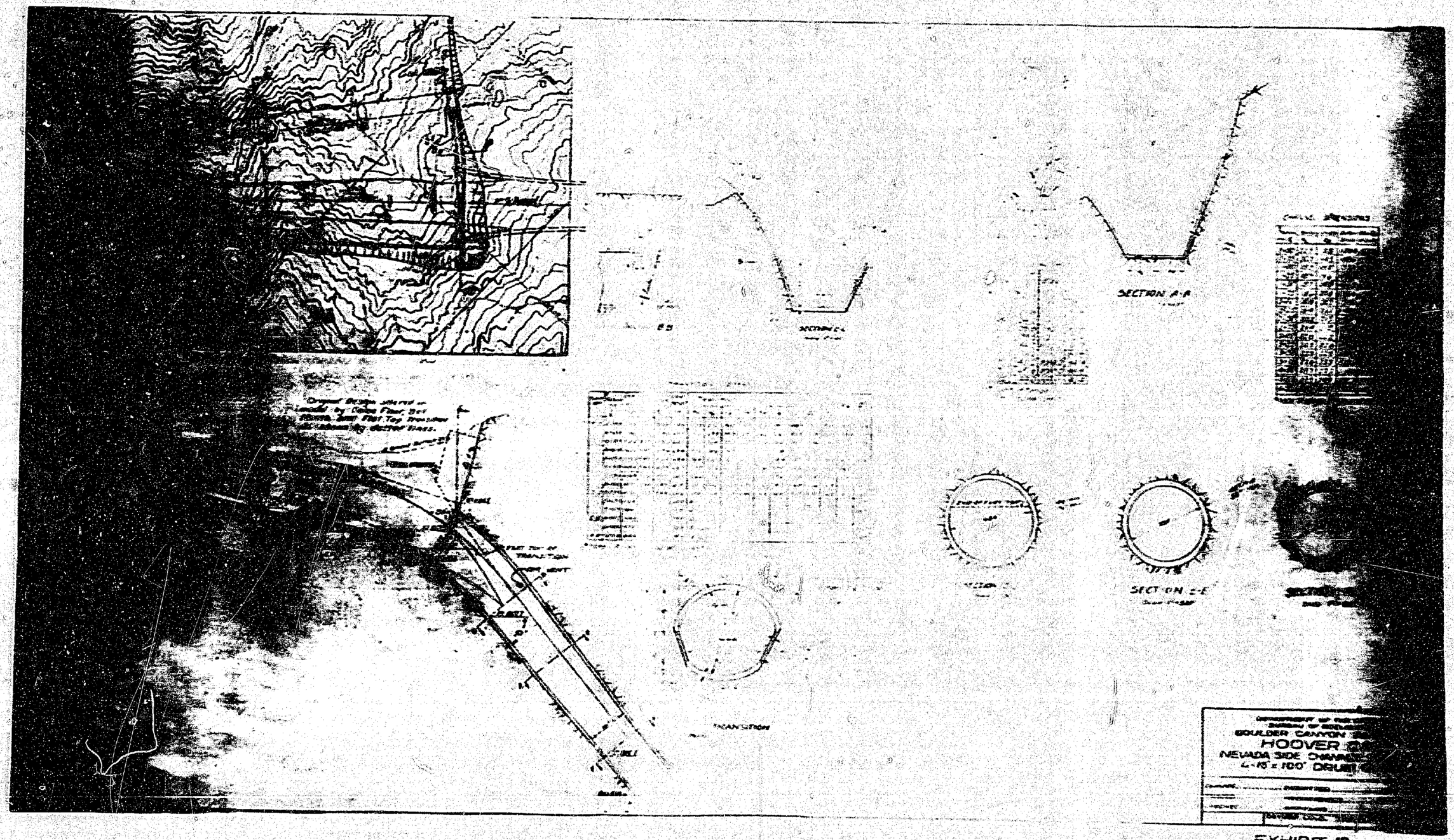

SECTION A-A
SECTION E-E
TRANSITION
BOULDER CANYON
HOOVER
NEVADA SIDE CHANNEL

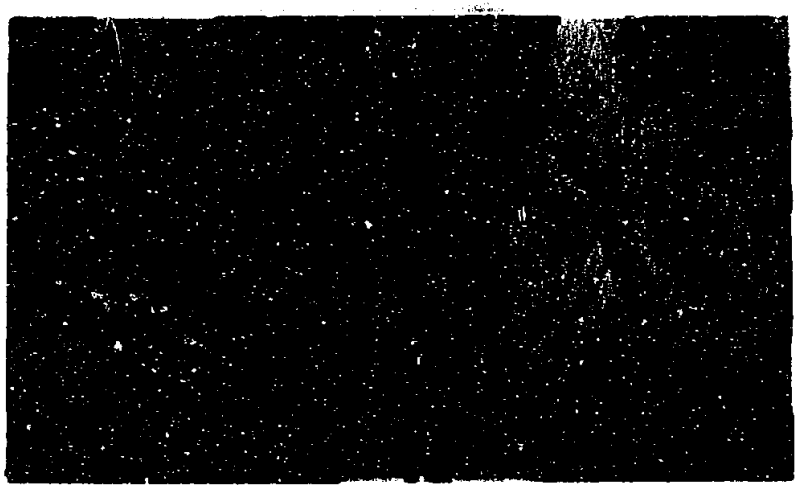
EXHIBIT 12

factorily. This resulted in a high wave on the right side of the channel. A [illegible] spiral motion was set up, causing considerable commotion in the transition section below the channel and also in the vertical bend where the shaft joins the tunnel. Severe splashing occurred at the vertical bend at 120,000 second-feet, and at 200,000 second-feet the flow was very disturbed through the whole tunnel, resulting, near the outlet end, in vibration which took the form of blows, or [illegible], repeated at about [illegible]-second intervals, and which, in the prototype, no doubt would be very objectionable. Plate 8 shows conditions in the channel.

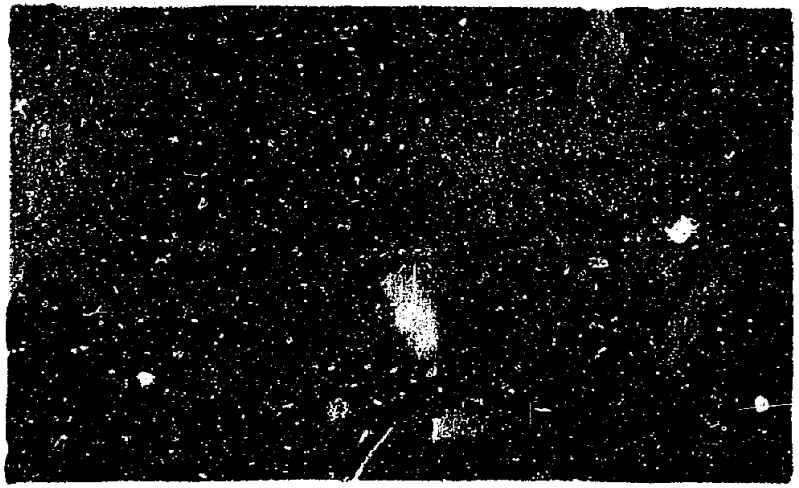
33. In order to increase the depth of water, so as better to dissipate the energy of cross flows, a false-floor was placed in the channel, as shown on the drawing (Exhibit 12), raising the floor 19 feet at station [illegible]. Some improvement was effected by this. A [illegible] in the form of a [illegible] was placed along the right side of the channel, to reduce the height of the wave along that side. This improved conditions considerably, both in the channel and in the shaft and tunnel.

34. Deeper water and a higher water surface in the channel [illegible]. These would be produced by a contraction at the lower end of the channel. Partly because the large size of the model made moving the part difficult, and partly because it was thought submerged flow would have a marked effect in smoothing out surface irregularities, the contraction was formed by blocking off a segmental section at the crown of the arch; a bell-mouth entrance was

12-AC-1. No discharge.



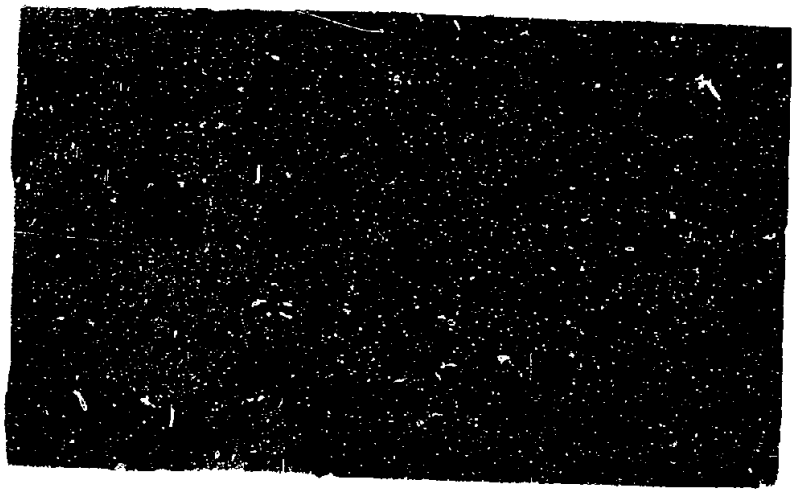
12-D-1. 50,000 Sec.-Ft. Discharge



12-A-1. 100,000 Sec.-Ft. Discharge.

12-C-1. 200,000 Sec.-Ft. Discharge.

First Drum Gate Model (Exhibit 12) (Montrose - Scale 1:20)
This design had too great a drop of the water between the reservoir and channel levels. PLATE 8.



X-63½. No Discharge.

9-C-1. 100,000 Sec.-Ft. Discharge.

X-99. 180,000 Sec.-Ft. Discharge.

9-A-1. 200,000 Sec.-Ft. Discharge.

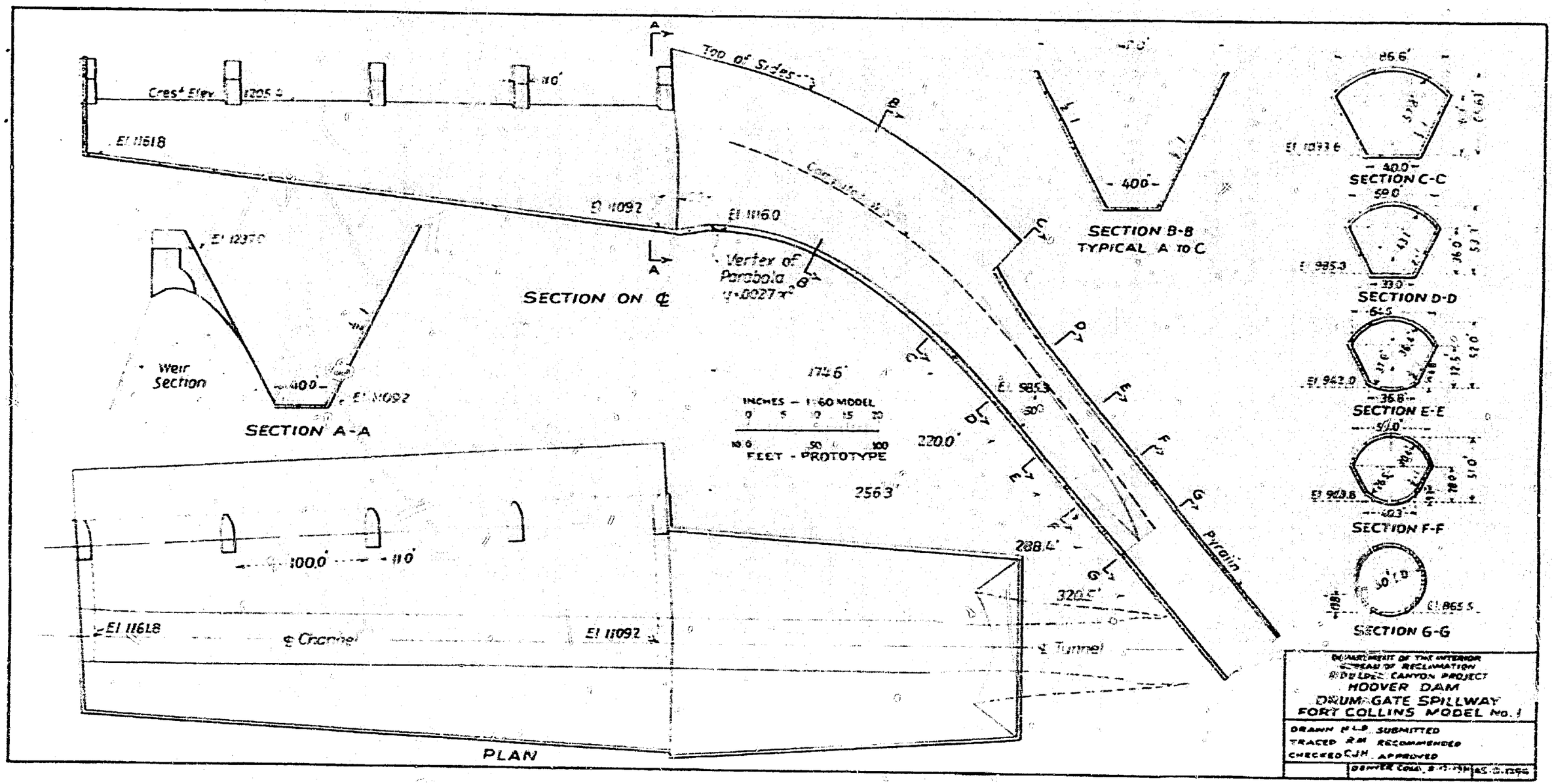
Model of Design Shown by Exhibit 12.
Modified by Raised Floor, Coping Baffle, Bell Mouth and Orifice.
The best results obtained with this design were with this set-up.

PLATE 9.

also provided. (See Exhibit 12.) A smaller contraction than that shown was tried but did not give as good results. At discharges over about 175,000 second-feet the contraction acted as a submerged orifice, producing deep water in the channel and starting the water down the shaft with a flat upper surface.

35. The results in the channel with the modifications described above, i. e., raised floor, baffle and segmental orifice with bell-mouth entrance, are shown on Plate 9. This combination produced, on the whole, the best results obtained. Several other modifications in which baffles of different forms were substituted for the coping were tried with varying degrees of success.

36. While the arrangement described produced satisfactory conditions at low and high stages, over the intermediate range the flow in the channel was very rough; and while this roughness seemed not to affect the flow in shaft and tunnel, it was desired to have as smooth flow as possible at all stages. As no further improvement seemed possible with this design, the model was changed to the design shown on Exhibits 1 and 2. On account of the large size of the model, considerable time is required to make the alterations, which require building a new transition and moving the shaft, as well as practically rebuilding the air channel. Hence, although work was begun about September 1, 1931, no tests of the new design except those at Fort Collins will be available prior to the board's visit to Montrose.



Crest Elev. 1205.4
El 1161.8
El 1109.2
El 1116.0
Top of Sides
Computed
Vertex of Parabola
SECTION ON ℄
SECTION B-B
TYPICAL A TO C
SECTION C-C
SECTION D-D
SECTION E-E
SECTION F-F
SECTION G-G
Weir Section
SECTION A-A
INCHES — 1:60 MODEL
FEET - PROTOTYPE
174.6'
220.0'
256.3'
288.4'
320.5'
Pyralin
100.0'
11.0'
℄ Channel
℄ Tunnel
PLAN
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM
DRUM-GATE SPILLWAY
FORT COLLINS MODEL No. 1
DRAWN H.L.B. SUBMITTED
TRACED R.M. RECOMMENDED
CHECKED C.J.H. APPROVED

EXHIBIT 13

37. A new design was prepared for study and is shown in Exhibit 12. For better observation of conditions in the vertical transition bend and shaft, and for ease of making alterations, the section was made open around the transition bend.

38. In the new design, the transition curve was omitted from the channel itself and was begun 40 feet below the lower end of the spillway crest; the bottom of the channel was lowered at the upper end, resulting in a somewhat flatter grade.

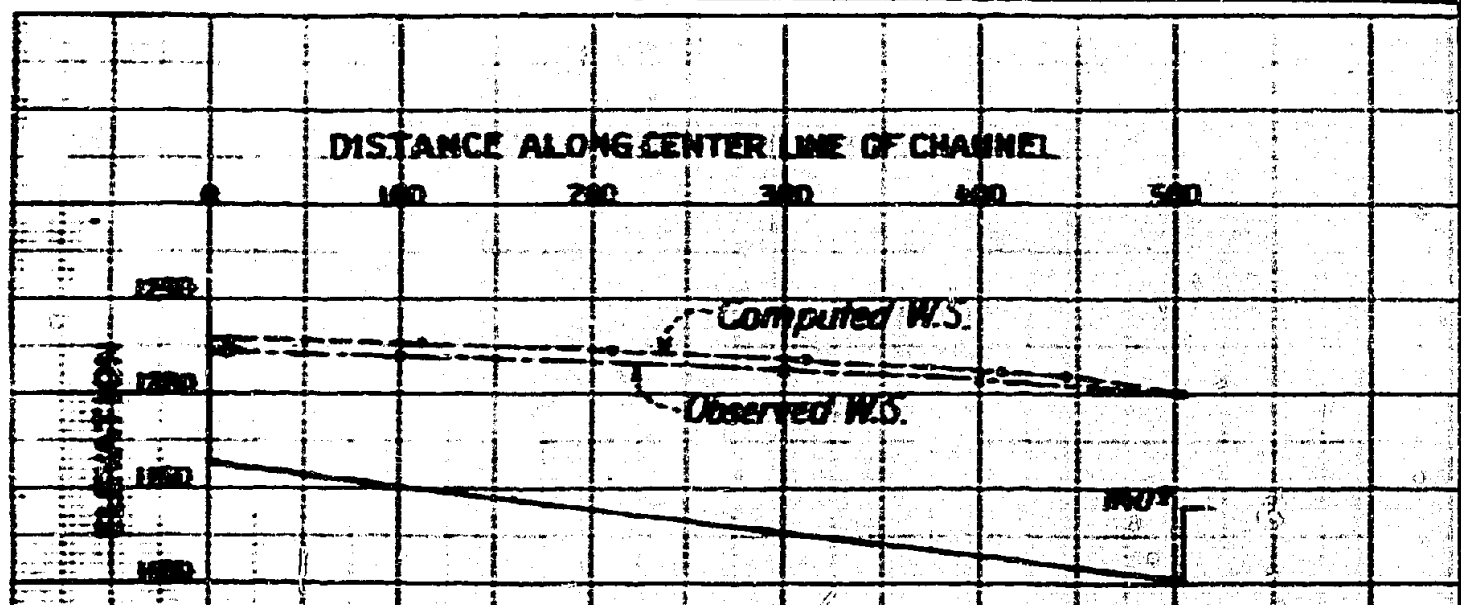
39. A model (scale 1:60) was made at Fort Collins. This design showed marked improvement over the previous one, but still was not thoroughly satisfactory. Views of this model and of its performance are shown by Plate 10. Experiments were made on the effect of changing the distance between the end of the spillway crest and the beginning of the transition bend; also with deepening the water in the channel by means of contractions; formed by raising the bottom and contracting the sides below the end of the spillway crest in different amounts.

40. Cross weirs, similar to that shown on Exhibits 1 and 2, in heights ranging from 1 to 40 feet, and effects up to 20 feet [illegible] in different combinations. The effect of different weirs is shown in Plate 11. While the same water surface elevation [illegible] at the lower end of the spillway crest [illegible] the weir was more effect in smoothing the flow. A flat slope to the face of the weir impaired

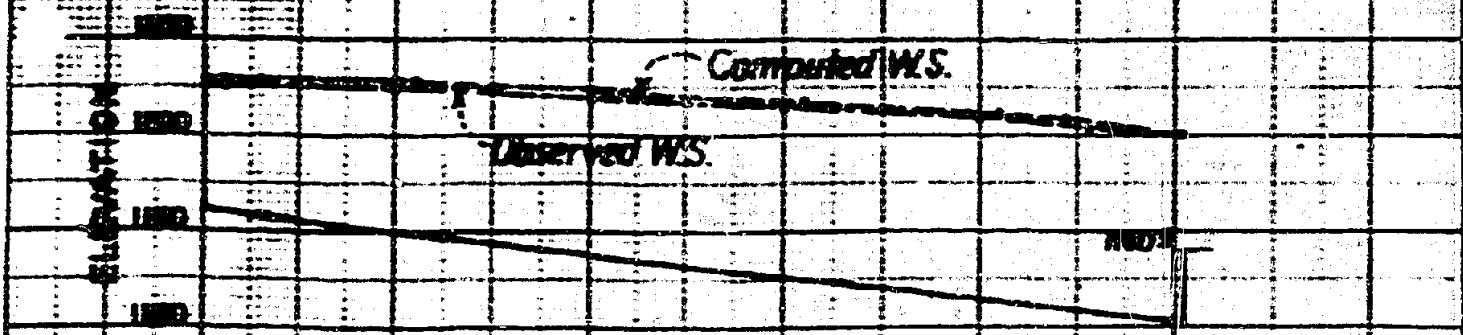
its effect. The offset, or shoulder, shown on the drawing at the lower end of the spillway crest is very desirable, because without it there exists a "hole" in the water surface at that point, which is obliterated by the offset. By means of these studies, the design presented in Exhibits 1 and 2 was arrived at. Plates 12 and 13 show a design with low weir and large contraction. In this case, the baffles are much more effective than with the higher weir. This design, while quite effective and somewhat lower in cost, was not considered quite as good as that shown in Exhibits 1 and 2.

41. It is, perhaps, natural to inquire how it is possible to introduce a weir some 30 feet high, and a contraction in addition, across a channel designed for a given flow, without any necessity of enlarging the channel. There are two reasons: first, because it is practicable to submerge the weir to a greater depth than was assumed; and, second, because the loss in the channel decreases as the depth is increased. In addition, it appears that the computed losses are somewhat too great. Exhibit 14 shows observed and computed water surfaces for the design shown by Exhibits 1 and 2.

42. Model tests. It is thought inadvisable to burden this memorandum with a complete description of all experiments made. The photographs show a few of the more noteworthy features. It is unfortunate that correct impressions can not be obtained from these photographs. The action of the water in the shaft, rather than in the channel itself, is the chief means by which the relative merit of different designs was judged; and it has not been found possible to



For design shown on exhibits 1 and 2



For design shown on exhibits 1 and 2 Modified by addition of baffle shown on exhibit 4.

Observed water surfaces from Fort Collins model - Scale 1:80

HOOVER DAM

COMPARISON OF COMPUTED AND OBSERVED WATER SURFACES FORT COLLINS SPILLWAY MODEL

C.J.H.
10-15-38

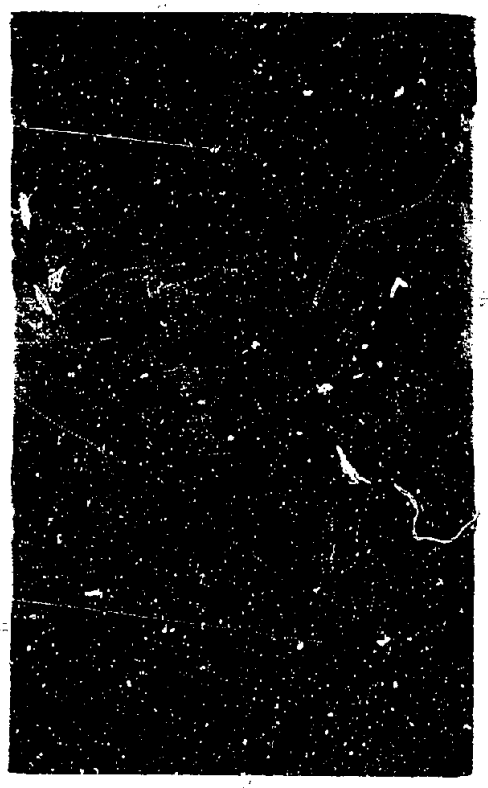
45-D-1469

EXHIBIT 14

12-A-0. Set-up.

12-A-1. Flow 40,000 c.f.s.

12-A-3. Flow 120,000 c.f.s.

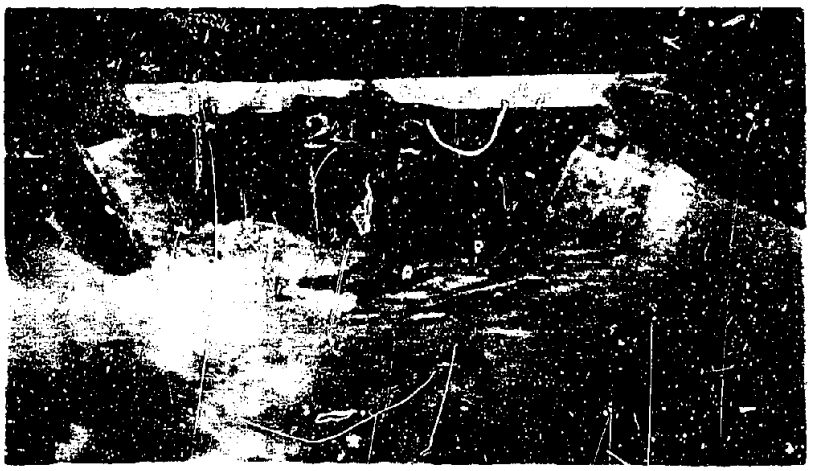


12-A-5. Flow 200,000 c.f.s.

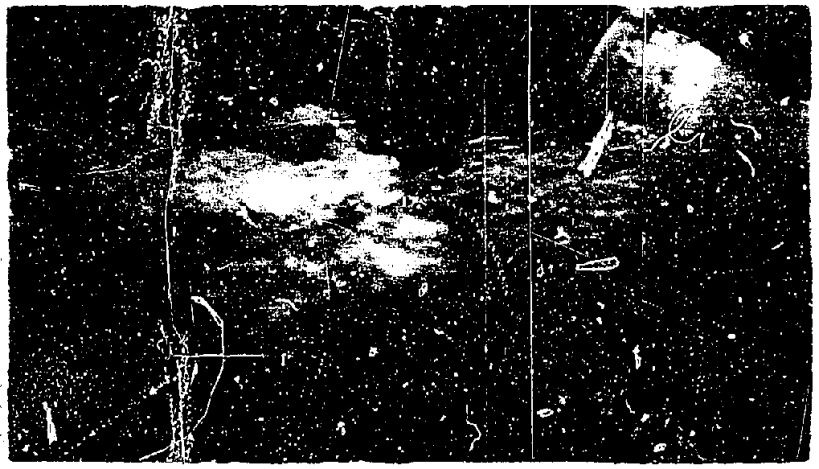
Model of Design Shown by Exhibit 13.
(Fort Collins - Scale 1:60)

PLATE

24-A-1. Flow 200,000 c.f.s. Weir 40' High. Crest at El. 1145.8.



24-A-2. Flow 200,000 c.f.s. Weir 20' High. Crest at El. 1133.8.



24-A-3. Flow 200,000 c.f.s. Weir 15' High. Crest at El. 1128.8.

Effect of Different Heights of Cross Weir in Channel of Exhibit 13.
Weir 60 Ft. Below End of Spillway Crest.
(Baffle Shown on Exhibit 4 in Channel)
(Fort Collins - Scale 1:60) PLATE 11.

52-A-0. Set-up.

52-A-3. Flow 100,000 c.f.s.

52-A-2. Flow 150,000 c.f.s.

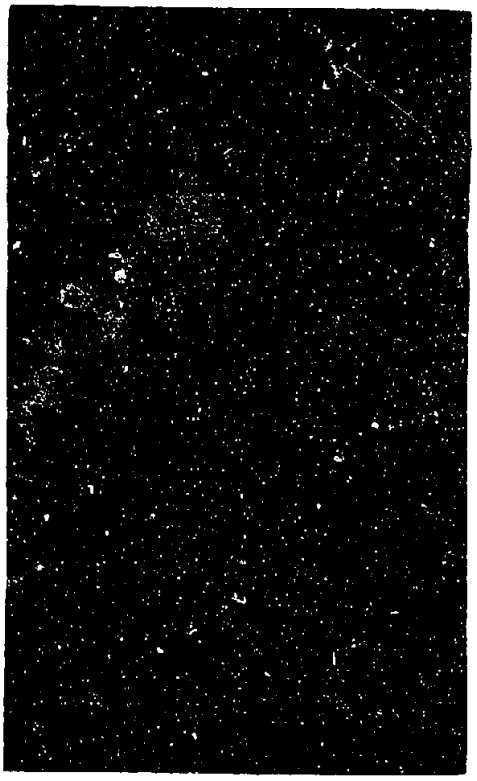
52-A-1. Flow 200,000 c.f.s.

Low Cross Weir with Large Lateral Contraction.
(Weir Crest at El. 1125 - 25-ft. shoulder)
(Fort Collins - Scale 1:60) PLATE 12.

53-A-0. Set-up.

53-A-3. Flow 100,000 c.f.s.

53-A-2. Flow 150,000 c.f.s.



53-A-1. Flow 200,000 c.f.s.

Same as Plate 12 except for Addition of **Baffles.**

PLATE 13.

reproduce this adequately, by either still or moving-pictures. Study of the photographs will show differences in the channel water surfaces, but the impression thus gained is not in all cases a reliable means of forming a judgment. The only accurate means by which the performance of a model can be judged is by actual observation.

43. Transitions. Four different transitions were designed, and models made. The curve used in the design for both the Fort Collins and Montrose final models (Exhibits 1 and 2) gave better results than a steeper curve; one considerably flatter was tried at Fort Collins, with no visible improvement. In all designs, an excess area equal to at least 35% of the computed wetted area was provided. In the design now proposed (Exhibits 1 and 2) there is considerably more than this at the upper end of the transition, on account of a desire to keep the water surface below the springing line of the crown arch until the vertical bend was passed. This results in an increase in cost which may be unnecessary.

44. Bend losses. No satisfactory theory of flow in bends is known; estimation of losses must be based on observations and experiments on small pipe, mostly on 90-degree bends of small radius running full; and on unsatisfactory empirical formulae. It appears probable, however, that bend losses decrease with an increase in size of pipe; and that assumptions based on losses in small pipe are therefore conservative. Exhibit 15 shows computed depths and velocities in shaft and tunnel. Computations were based on a value of "n" of .013

in Kutter's formulae for friction losses and on bend losses of 0.18 and 0.15 of the velocity head for the vertical bends and for the horizontal bend in the Nevada tunnel respectively. For the bend and transition at the top of the shaft, a loss of 0.10 the difference in velocity heads was assumed; this may seem small compared to the other assumptions, but conditions here should be favorable to low bend losses. These figures have been used by the bureau for a number of years, and no lack of capacity has ever been reported.

45. Vertical bends in models. Only rough checks are possible from the models; results agree very well with the computations. At Fort Collins, the vertical bend and tunnel below are made of transparent material, so that the flow can be observed; the water glides smoothly around the bend and no disturbance is evident; but measurements are difficult if not impossible. At Montrose, removable panels or "windows" were set in the 30-inch wood-stave pipe which serves as a model, but observations through these windows are unsatisfactory.

46. Horizontal bends. In order to observe the flow in the horizontal bend of the Nevada tunnel a model (scale 1:60) was made at Fort Collins of 10-inch wrought steel pipe, with the wall cut away as much as possible without permitting the water to escape. When the water strikes the curve, the surface is folded and a part of the water follows the top of the pipe for a short distance and then falls into the stream. This action remains practically the same for all flows. The radius of the bend was that shown in the specifications, 819 feet.

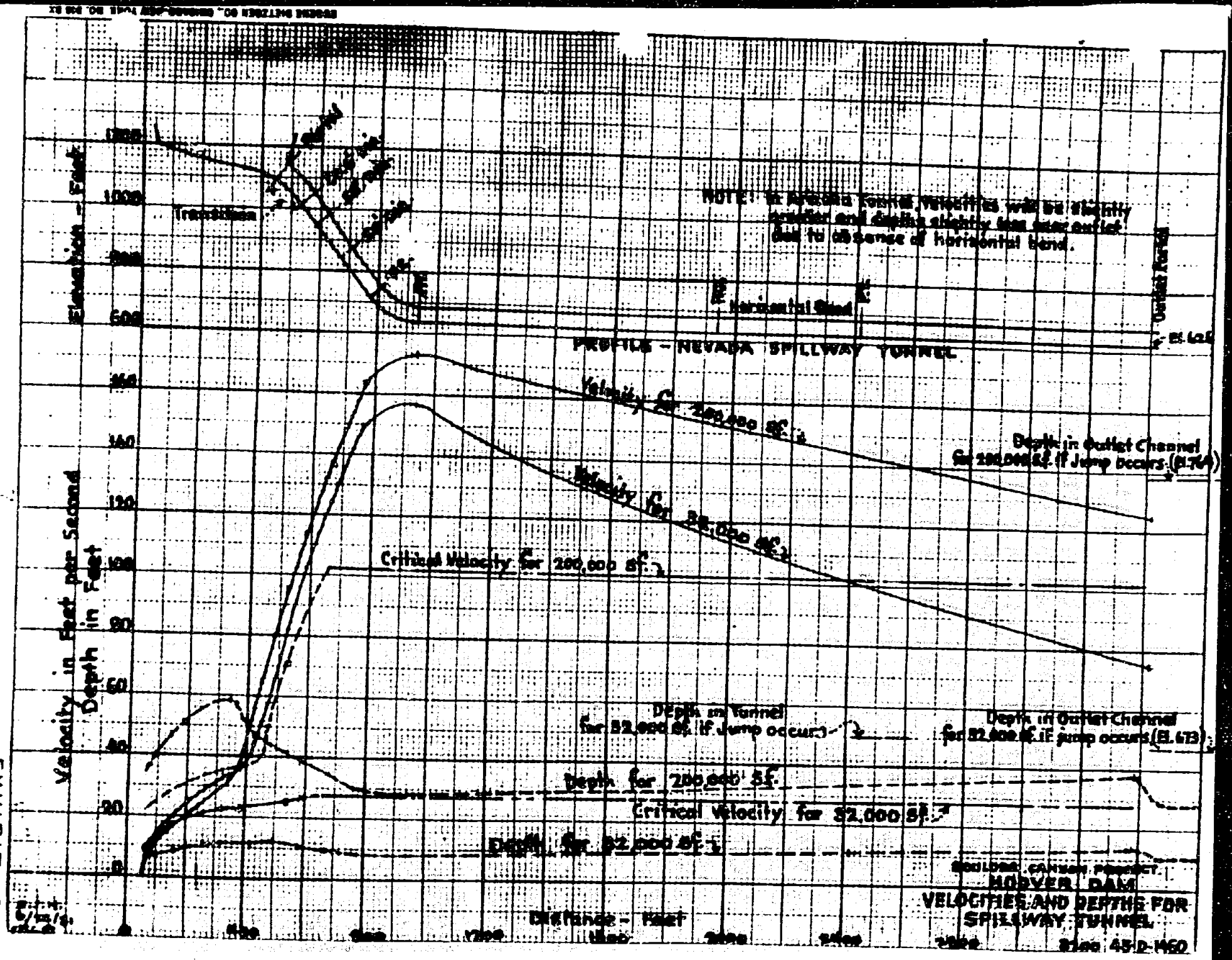

Elevation - Feet
Velocity in Feet per Second
Depth in Feet
Transition
Horizontal Bend
Outlet Portal
PROFILE - NEVADA SPILLWAY TUNNEL
Velocity for 200,000 S.F.
Depth in Outlet Channel for 200,000 S.F. if Jump Occurs (El. 704)
Velocity for 52,000 S.F.
Critical Velocity for 200,000 S.F.
Depth in Tunnel for 52,000 S.F. if Jump occurs
Depth in Outlet Channel for 52,000 S.F. if jump occurs (El. 673)
Depth for 200,000 S.F.
Critical Velocity for 52,000 S.F.
Depth for 52,000 S.F.
Distance - Feet
BOULDER CANYON PROJECT
HOOVER DAM
VELOCITIES AND DEPTHS FOR SPILLWAY TUNNEL
Plan 45-D-1460

EXHIBIT 15

A model bend with 1000-foot radius was built to the same scale, but no improvement could be detected. Another model was built with a compound curve with no improvement. Flow in these bends is shown by Plate 14.

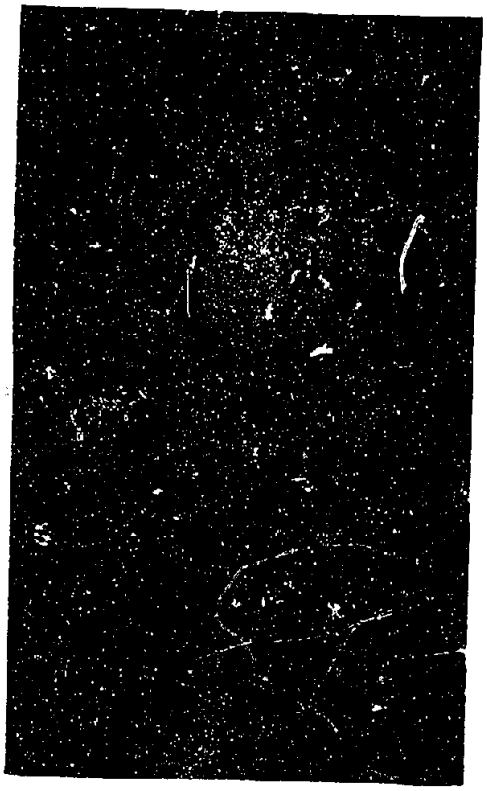
47. As the above-described experiments at Fort Collins showed no advantage gained from altering the radius of the horizontal bend, the alignment shown in the specifications has been retained.

48. The Montrose model, scale 1:20, was built to this radius. Small panes of glass were placed in the pipe for purposes of observation. Spray makes observation difficult, but apparently the condition is the same as that in the smaller models at Fort Collins. At the tunnel outlet, the water surface is level and the motion of the water surface set up at the horizontal bend seems to have disappeared entirely. The depth at the outlet agrees well with the computations.

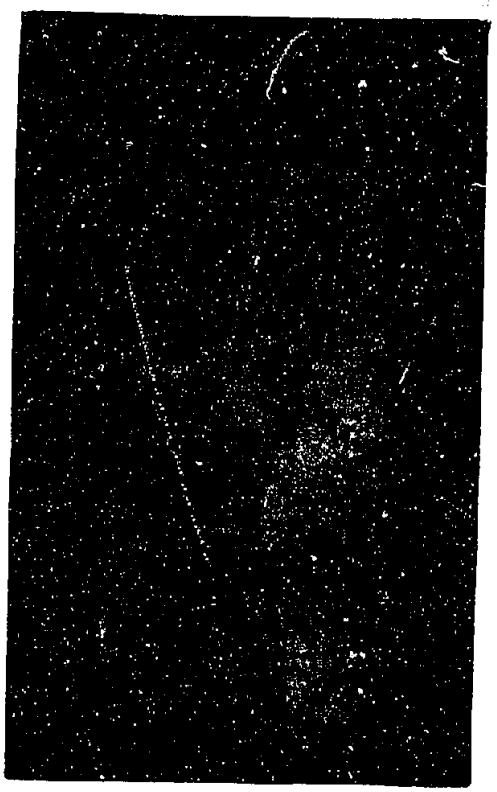
49. It is believed, in view of the above, that no uneasiness need be felt concerning this bend. It would be possible to allow the Arizona spillway gates to be lowered completely before the Nevada gates begin to move, thus keeping the discharge in the Nevada tunnel at a low figure (about 32,000 second-feet) for all except most extraordinary floods. This would cause an unbalanced condition at the tunnel outlets which might be objectionable, and is believed unnecessary.

Discharge 60,000 sec.-ft.

Discharge 200,000 sec.-ft.



1600-ft. Radius Bend.



Compound Bend.

Flow in Horizontal Bends
(Fort Collins - Scale 1:60)

PLATE 14.

50. Entrained air. Test on side channel models in 1923 showed a swell of from -3 to +20 per cent with an average of about 5 or 6 per cent; tests on the Arrowrock spillway with a discharge of 10,000 second-feet showed a swell of about 45 per cent. (Julian Hinds, Side Channel Spillways, Trans. A. S. C. E., Vol. 89.) These percentages were derived from observations of water surfaces and velocities in the channel. Even in the models, such measurements can not be of a high order of accuracy; the measurements at Arrowrock were made under such adverse conditions that they are believed of no value. Even if correct, they would be too high for maximum discharges, since the spillway gates were lowered only slightly and the water fell about 40 feet through the air. This is also the conclusion reached by Mr. Hinds.

51. Experiments on a model at Fort Collins in 1930 indicated only about 2 per cent of air; these measurements were volumetric and are believed accurate. In view of the conditions at maximum discharge, it is believed that ample provision for any possible amount of air has been made by the excess areas present in the transition, shaft and tunnel.

52. Frequency of use of spillways. Graphs of the reservoir operation under conditions as of 1938 for the years 1897 to 1929 inclusive, prepared under the direction of Mr. E. B. Debler, show that in only nine of these 33 years would the water have reached the fixed crest of the drum gate spillways. The maximum elevation reached

during each year (by months), with the distance above or below the fixed crest (elevation 1205.4), is shown by the following table. The highest elevation reached is still 10 feet below the crest of the drum gates.

TABLE IV

Maximum Reservoir Water Surface Elevations.

Year	Max. Elev.	Dist. above Fixed Crest	Year	Max. Elev.	Distance above Fixed Crest
1897	1210	5	1915	1197	-8
98	1172	-33	16	1192	-13
99	1185	-20	17	1211	6
1900	1171	-34	18	1197	-8
01	1174	-31	19	1170	-35
02	1154	-51	1920	1211	6
03	1131	-74	21	1211	6
04	1098	-109	22	1206	0
1905	1135	-70	23	1211	6
06	1169	-36	24	1195	-10
07	1211	6	1925	1160	-45
08	1183	-22	26	1180	-25
09	1211	6	27	1188	-17
1910	1196	-9	28	1201	-4
11	1184	-21	29	1201	-4
12	1200	-5	1930		-? (Not included)
13	1181	-24	31		-? (in computations
14	1211	6			

33. Flood capacity. It is of interest to estimate the maximum flood which can be handled at Hoover Dam. For this purpose it is necessary to assume a flood. Data for the assumptions made are taken from the report, "Hydrology of the Boulder Canyon Reservoir," (January 31, 1930) by E. B. Debler, (Ex. E, p. 6, and Ex. A, p. 8). The estimated flood volumes discharged by the river during the time when the rate of discharge exceeded a certain specified figure are as follows:

TABLE V

Colorado River at Black Canyon

Run-Off Volumes - Ten-Thousand-Year Flood

Runoff Million Acre-Feet	Discharged While Rate Exceeded
22	40,000 Sec.-Ft.
17	50,000
12	60,000
10	75,000
6	100,000
*2.6	200,000
* .8	400,000

* Extrapolated values

The above figures are for undepleted flow; that is, actual figures of record are corrected for depletion to give what the volume of discharge would have been if no development had taken place. They are, therefore, larger than would actually occur, even with the present upstream developments.

54. From the above figures, the volume discharged during the time the rate of flow exceeded any given amount is known.

55. The peak discharge of the flood which probably would be equaled or exceeded once in ten thousand years is estimated at 450,000 second-feet. For the purpose of constructing a hydrograph, this peak flow is not entirely consistent with the estimates of volume. A characteristic of high Colorado River floods is the short duration of the peak discharge, and since assumption of a sharp peak is unfavorable, it has been made. This assumption

results in a peak of 800,000 second-feet, or about 1.8 times the estimated probable peak flow; the flow in the river would then exceed 100,000 second-feet for only 30 days. This is about the same time as for floods of record, which are, of course, much smaller than that assumed; during the 1884 flood, the rate of discharge probably exceeded 100,000 second-feet for 20 days. The assumptions, therefore, seem very conservative.

56. It has been assumed that no flood storage was available at the beginning of the flood; that the drum gates operated automatically; that outlet valves were opened when the water surface reached elevation 1232, and that the power house discharge was 12,000 second-feet. As shown graphically on Exhibit 16, the water surface would have risen to elevation 1233.6. If the outlets had remained closed, the water surface would have risen to elevation 1235, still 27 inches below the top of the parapet wall.

57. On the same assumptions, (outlet valves remaining closed) for the design shown in the specifications, the water would have flowed about an inch deep over the parapet.

58. Ample warning of a flood of such magnitude would be given by the amount of snow in the mountains; the river stage at Hoover Dam can be predicted accurately 3 to 5 days in advance from discharges of the tributaries. The assumptions made are, therefore, all conservative. It would seem probable that if advantage of all available information were taken by those vitally interested, a flood with a peak flow in the neighborhood of one million second-feet could safely be passed through the reservoir.

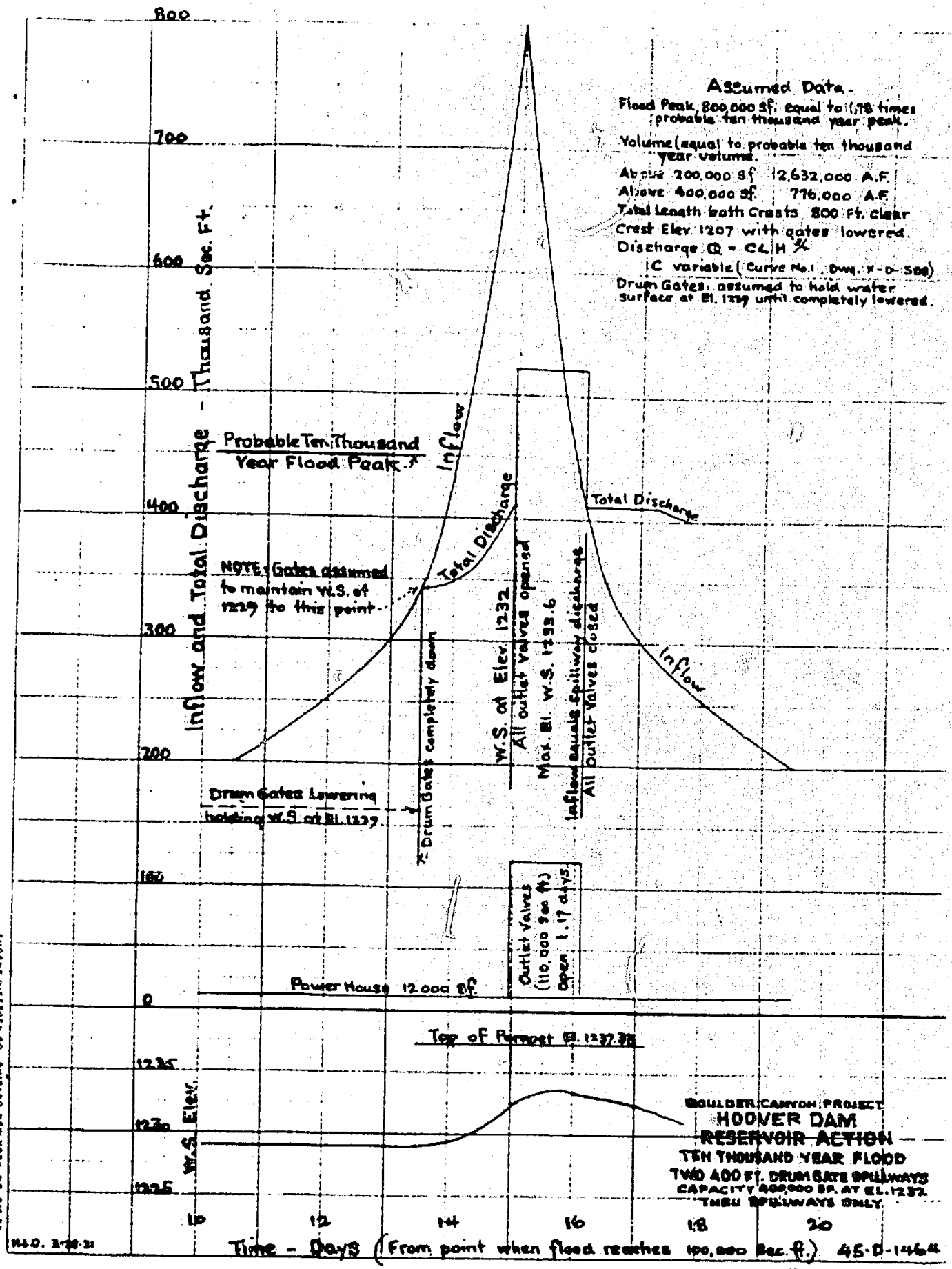

Assumed Data.
Flood Peak, 800,000 sf. equal to 1.78 times probable ten thousand year peak.
Volume (equal to probable ten thousand year volume.
Above 200,000 sf 12,632,000 A.F.
Above 400,000 sf. 776,000 A.F.
Total length both Crests 800 Ft. clear
Crest Elev. 1207 with gates lowered.
Discharge Q = CL H 3/2
C variable (Curve No. 1 Dwg. X-D-588)
Drum Gates: assumed to hold water surface at El. 1239 until completely lowered.
Inflow and Total Discharge - Thousand Sec. Ft.
800
700
600
500
400
300
200
100
0
Probable Ten Thousand Year Flood Peak
Inflow
Total Discharge
Total Discharge
NOTE: Gates assumed to maintain W.S. of 1239 to this point
Drum Gates Lowering holding W.S. at El. 1239
Drum Gates completely down
W.S. at Elev. 1232
All outlet valves opened
Max. El. W.S. 1233.6
Inflow equals Spillway discharge
All outlet valves closed
Inflow
Outlet Valves (110,000 Sec. Ft.) open 1.17 days.
Power House 12,000 sf.
Top of Parapet El. 1237.38
W.S. Elev.
1235
1230
1225
10
12
14
16
18
20
Time - Days (From point when flood reaches 100,000 Sec. Ft.)
BOULDER CANYON PROJECT
HOOVER DAM
RESERVOIR ACTION
TEN THOUSAND YEAR FLOOD
TWO 400 FT. DRUM GATE SPILLWAYS
CAPACITY 400,000 SF. AT EL. 1232.
THRU SPILLWAYS ONLY
45-D-1464

EXHIBIT 16

59. In addition to the spillway types already discussed, three other types have been suggested; i. e.,

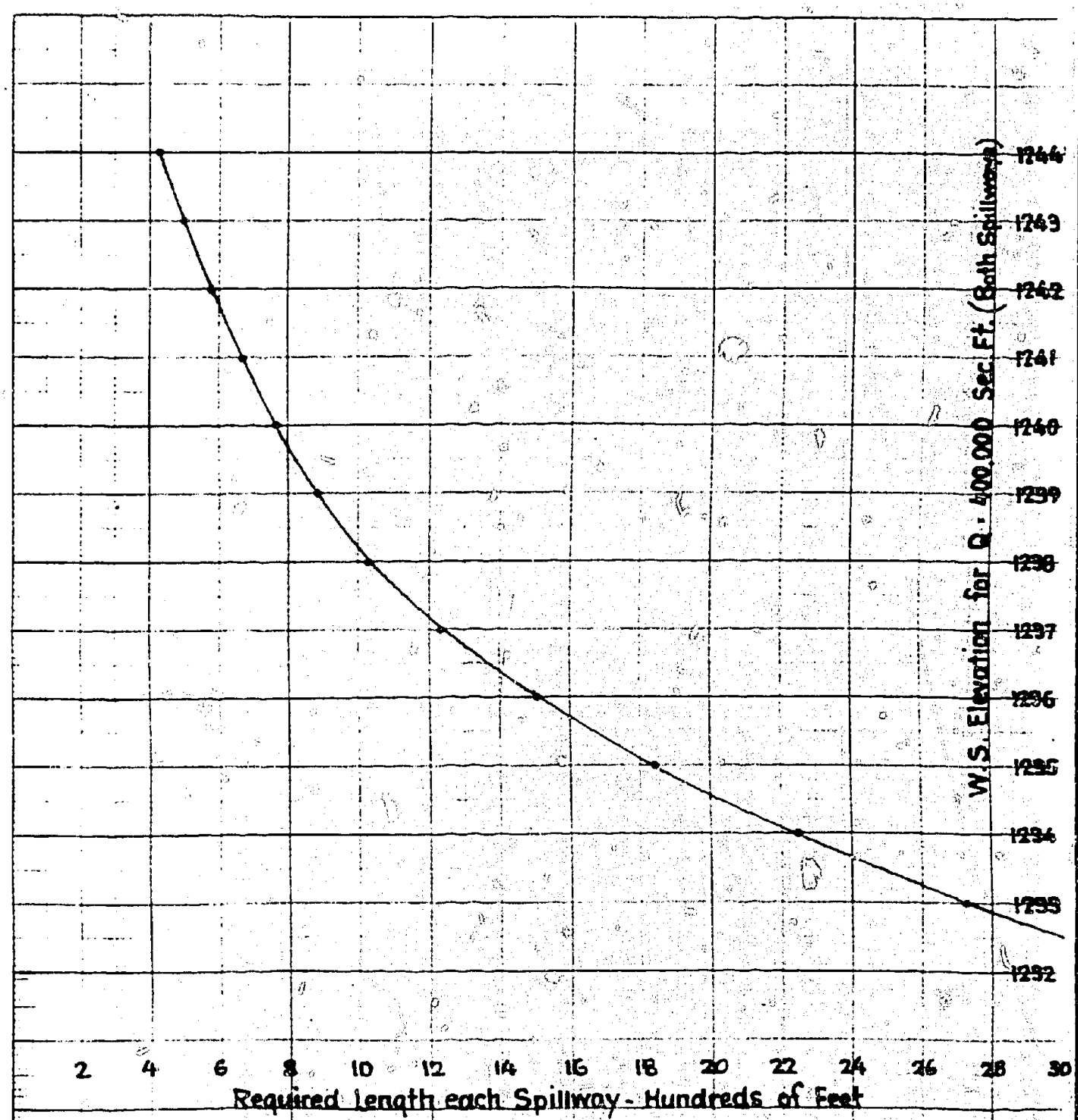
(a) uncontrolled crest;

(b) siphon spillways;

(c) glory hole.

60. Uncontrolled crest. An uncontrolled crest is impracticable without allowing a much greater rise than 3 feet for the great increase in discharge required (168,500 second-feet) between elevation 1229 and elevation 1232. This would require either that the dam be raised or that the flood control storage, or the power head and power and irrigation storage, be decreased. Exhibit 17 shows the length of crest necessary for such a spillway for a total discharge of 63,000 second-feet at elevation 1229 and 400,000 second-feet at different elevations. For 400,000 second-feet at elevation 1232.5 such crest would need to be 3,000 feet long; or for two 400-foot crests, a maximum water surface of about 1234.5 would be required. The cost of such spillways probably would be somewhat less than that of the drop inlet spillways proposed.

61. Siphon spillways. Most normal spillways show "efficiencies" of 70 per cent or less although much higher efficiencies have been obtained with models. If an efficiency of 60 per cent be assumed on account of the large throat, and the drop in head be taken equal to the height of the water barometer, 24 siphons with throats 15 feet high by 2[illegible] feet wide, or the equivalent, would be needed for each spillway. It is felt that the present knowledge of siphon spill-



HOOVER DAM
SIDE CHANNEL SPILLWAY - FREE CREST
REQUIRED LENGTH - EACH SPILLWAY

Discharge over both spillways
63,000 Sec. Ft. at Elev. 1229
400,000 Sec. Ft. at given Elev.

C.J.H. 1-20-38

45-D-1463

EXHIBIT 17

ways is not sufficient to warrant such sizes; moreover, the cost would probably exceed that of the design proposed.

62. Glory hole. This was the type originally proposed; but for a much smaller capacity. For the capacities now fixed, gates on the crest would be necessary. The vertical shaft was the principal objection; and while it might be possible to design a spillway of this type with an inclined shaft, a long experimental program would have been required, with no assurance of reaching a satisfactory design. As it was desired to fix in the specifications, as nearly as possible, the general type of spillway to be used, and as no time was available for experiments, the glory hole was abandoned.

63. Spillways with low outlet velocities. At its latest meeting, the board expressed a wish for studies of spillway outlets using a velocity not exceeding 100 feet per second.

64. It is not practicable to destroy sufficient energy between the reservoir water surface and the diversion tunnels to reduce the velocity to 100 feet per second. At the junction of the shaft with the diversion tunnel, the velocity is about 175 feet per second and the velocity head is 475 feet. The velocity head for 100 feet per second is only 155 feet; to destroy the difference would, at maximum capacity, entail the absorption of more than 700,000 horsepower in each shaft. Any attempt to increase friction to such an extent would result in a greater menace than the velocity which it is desired to avoid.

65. Three possibilities present themselves; viz.,

(a) tunnels along the canyon walls at a higher elevation than the diversion tunnels;

(b) risers at the outlet ends of the diversion tunnels which would force them to run full, reducing the velocity to about 102 feet per second;

(c) spillways on the Arizona side only, with tunnels leading to Spillway Wash.

66. Since tunnels along the canyon walls must have a fixed length, the minimum cost will be given by the highest velocity. Estimates were made using a velocity of 100 feet per second. The use of separate tunnels would make it possible to use all four diversion tunnels for penstocks, eliminating the 30-foot penstock tunnels, and to place outlet valves in the two outside tunnels, reducing somewhat the cost of the canyon wall outlet works. A slight shortening of the outer diversion tunnels would also result from this plan. A plan of this scheme is shown by Exhibit 18.

67. The outlets of these tunnels would be at about elevation 900, some 250 feet above the river into which the water must find its way. Two treatments of the outlet are possible. The first is to construct a shaft from the elevation of the tunnels to the river bed; the velocity in this shaft would still reach about 175 feet per second. The second is to allow the water to drop along the face of the cliff, or to build a lip so that the trajectory is entirely in air.

68. The use of a riser at the lower end of the diversion tunnels would force them to run full, with a velocity of 102 feet per second. However, the mouth of the riser would have to be at a definite elevation, which would be fixed by the estimated losses in the tunnels at maximum capacity. These losses are difficult to estimate accurately, and decrease rapidly as the discharge decreases. It was not found possible to design an inlet which would avoid a hydraulic jump to a full tunnel at some stage. A hydraulic jump in a tunnel of this size would cause serious disturbances.

69. The use of spillway wash as an outlet would require that the spillway be placed entirely on the Arizona side. Since the elevation of the tunnel would be lower for higher velocities, requiring an increased length of tunnel, preliminary estimates were made for different velocities. These schemes are shown on Exhibits 19 to 21. The cost estimates include nothing for preparation of the channel between the ends of the tunnel outlet cuts and the river. Large amounts of money would be required to prevent erosion of this wash. While the erosion of itself might not be a serious matter, the deposition of the eroded material in the river would form a dam, backing water around the power house, probably with disastrous results in case of a large flow through the spillways.

70. The cost of the various plans just discussed are given in the following table. The increase in cost over and above the cost of the drum gate spillways takes account of savings made possible by the rearrangements discussed in paragraph 66. Costs include 25% for engineering, superintendence and contingencies.

TABLE VI

COST OF SPILLWAYS USING OUTLETS OTHER THAN DIVERSION TUNNELS

DRUM GATE CRESTS

Gates	Tunnels	Location	Velocity	Total Cost	*Increase
(1) 4 - 16 x 100	1 - 58.6 feet	Along river. Mouth at El. 900.	100	$11,400,000	$ 4,700,000
(1) 4 - 16 x 100	1 - 58.6 feet	Along river. Shaft to El. 626.	100	12,400,000	5,700,000
5 - 20 x 100	1 - 75 feet	To Spillway Wash (2)	100	13,500,000	6,700,000
5 - 20 x 100	2 - 75 feet	To Spillway Wash (2)	50	18,900,000	12,000,000
5 - 20 x 100	5 - 47.2 feet	To Spillway Wash (2)	50	24,000,000	17,300,000

* Above cost of 4 - 16 x 100-foot drum gates shown by Exhibits 1 and 2.
(1) Each of two spillways.
(2) Cost of work on channel of wash below end of outlet cuts not included.

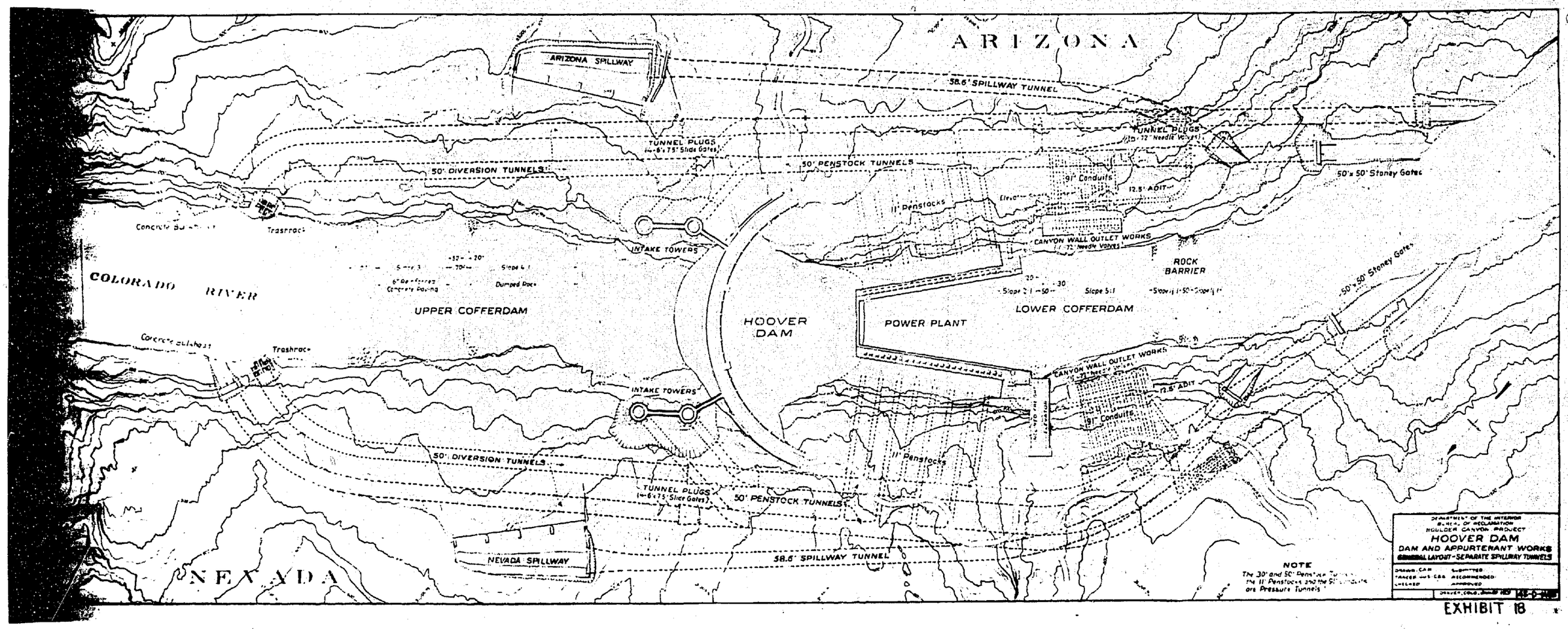
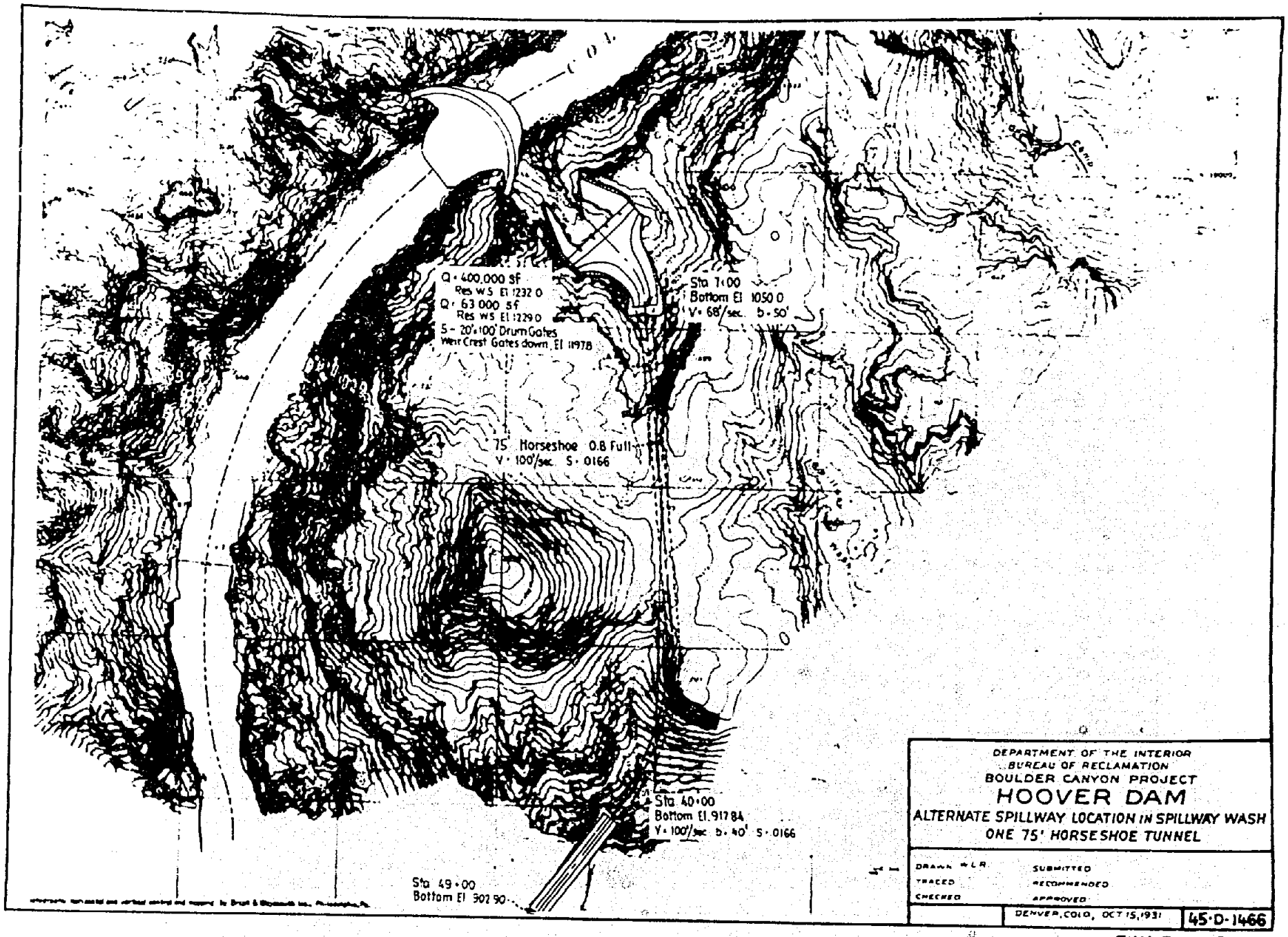

ARIZONA
ARIZONA SPILLWAY
38.6' SPILLWAY TUNNEL
TUNNEL PLUGS
(in 6 x 75' Slide Gates)
50' DIVERSION TUNNELS
50' PENSTOCK TUNNELS
TUNNEL PLUGS
(in 72" Needle Valves)
91" Conduits
12.5' ADIT
50' x 50' Stoney Gates
11' Penstocks
Concrete Bulkhead
Trashrack
INTAKE TOWERS
CANYON WALL OUTLET WORKS
(in 72" Needle Valves)
ROCK BARRIER
COLORADO RIVER
Slope 4:1
Dumped Rock
UPPER COFFERDAM
HOOVER DAM
POWER PLANT
LOWER COFFERDAM
Slope 2:1
Slope 5:1
50' x 50' Stoney Gates
NEVADA SPILLWAY
38.6' SPILLWAY TUNNEL
NEVADA
NOTE
The 30' and 50' Penstock Tunnels, the 11' Penstocks and the 91" Conduits are Pressure Tunnels
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM
DAM AND APPURTENANT WORKS
GENERAL LAYOUT - SEPARATE SPILLWAY TUNNELS

EXHIBIT 18



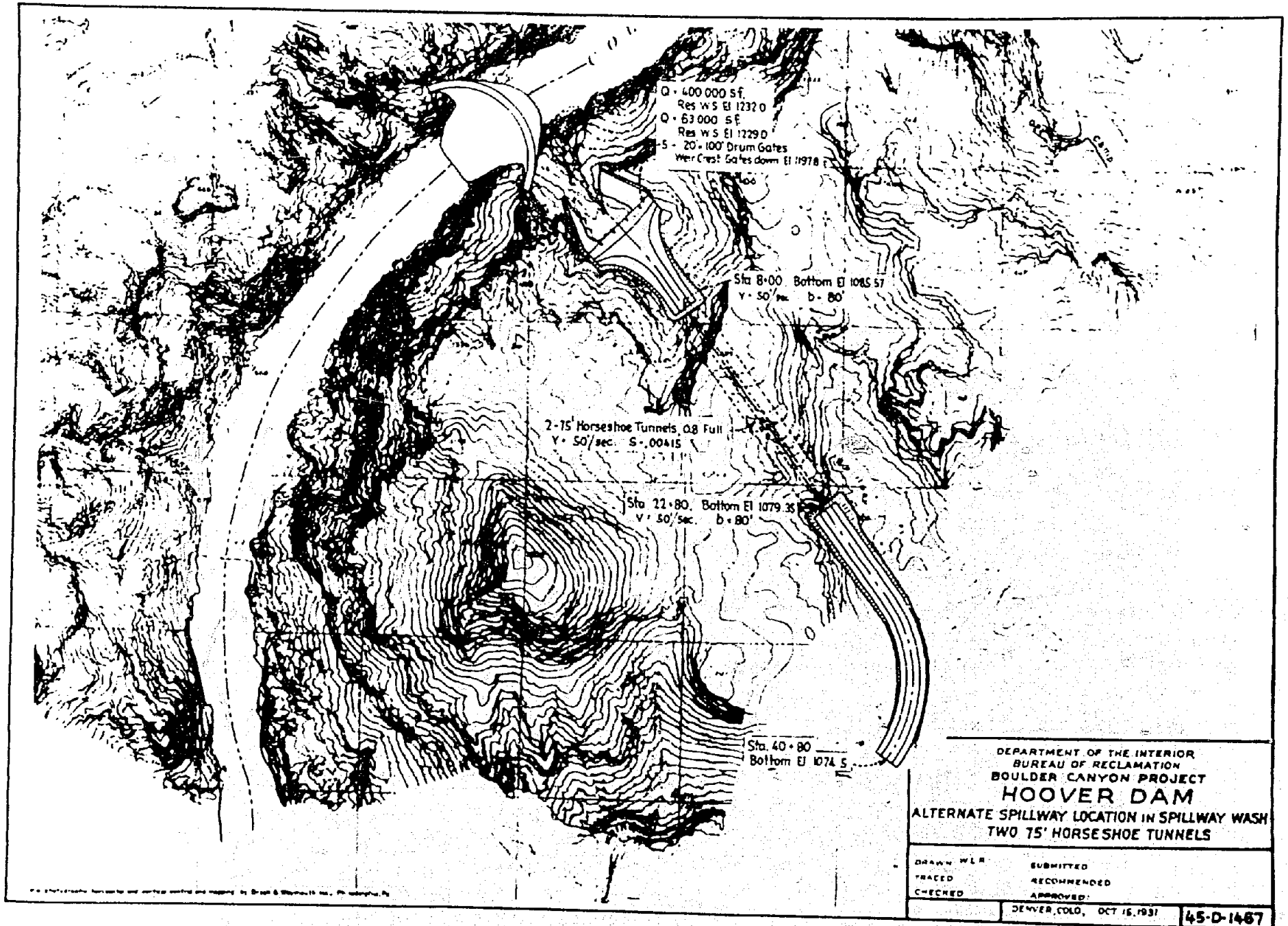
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT

HOOVER DAM

ALTERNATE SPILLWAY LOCATION IN SPILLWAY WASH
ONE 75' HORSESHOE TUNNEL

DRAWN W.L.R.	SUBMITTED	
TRACED	RECOMMENDED	
CHECKED	APPROVED	
	DENVER, COLO., OCT. 15, 1931	45-D-1466

EXHIBIT 19



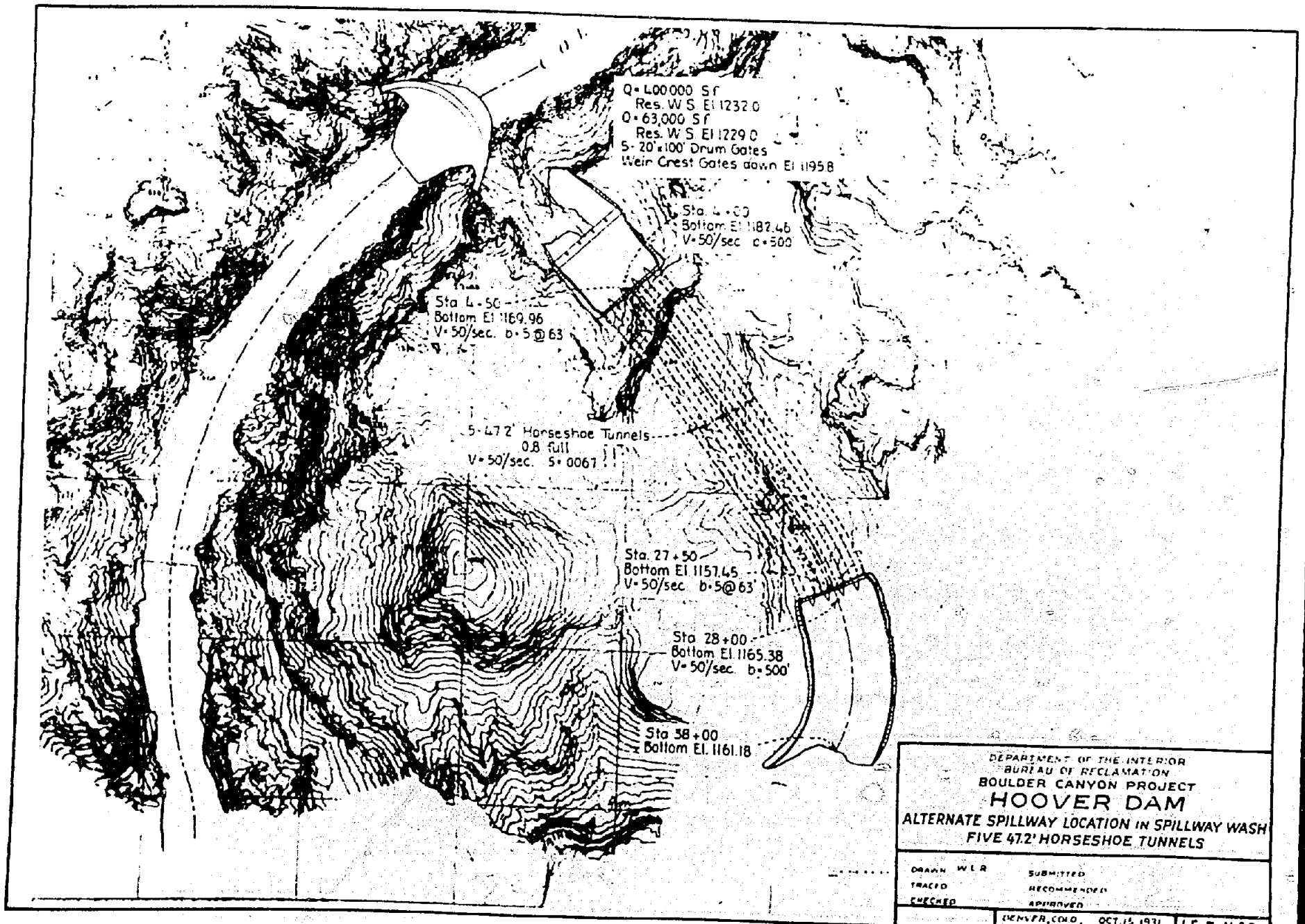
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT

HOOVER DAM

ALTERNATE SPILLWAY LOCATION IN SPILLWAY WASH
TWO 75' HORSESHOE TUNNELS

DRAWN W.L.R.	SUBMITTED	
TRACED	RECOMMENDED	
CHECKED	APPROVED:	
	DENVER, COLO. OCT 15, 1931	45-D-1487

EXHIBIT 20



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
HOOVER DAM
ALTERNATE SPILLWAY LOCATION IN SPILLWAY WASH
FIVE 47.2' HORSESHOE TUNNELS

DRAWN W.L.R. SUBMITTED
TRACED RECOMMENDED
CHECKED APPROVED
DENVER, COLO. OCT. 15, 1931 45-D-1468

EXHIBIT 21