UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES FOR THE REDESIGN OF POWER CANAL DIVERSION DAM-- SALT RIVER PROJECT, ARIZONA

Hydraulic Laboratory Report No. Hyd.-218

RESEARCH AND GEOLOGY DIVISION

BRANCH OF DESIGN AND CONSTRUCTION
DENVER, COLORADO

NOVEMBER 29, 1948

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UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Branch of Design and Construction
Research and Geology Division
Denver, Colorado
Date: November 29, 1948

Laboratory Report No. 218
Hydraulic Laboratory
Compiled by: J. W. Ball and
J. M. Buswell
Reviewed by: J. E. Warnock

Subject: Hydraulic Model Studies for the Redesign of Power Canal Diversion Dam--Salt River Project, Arizona.

HISTORICAL BACKGROUND AND SUMMARY

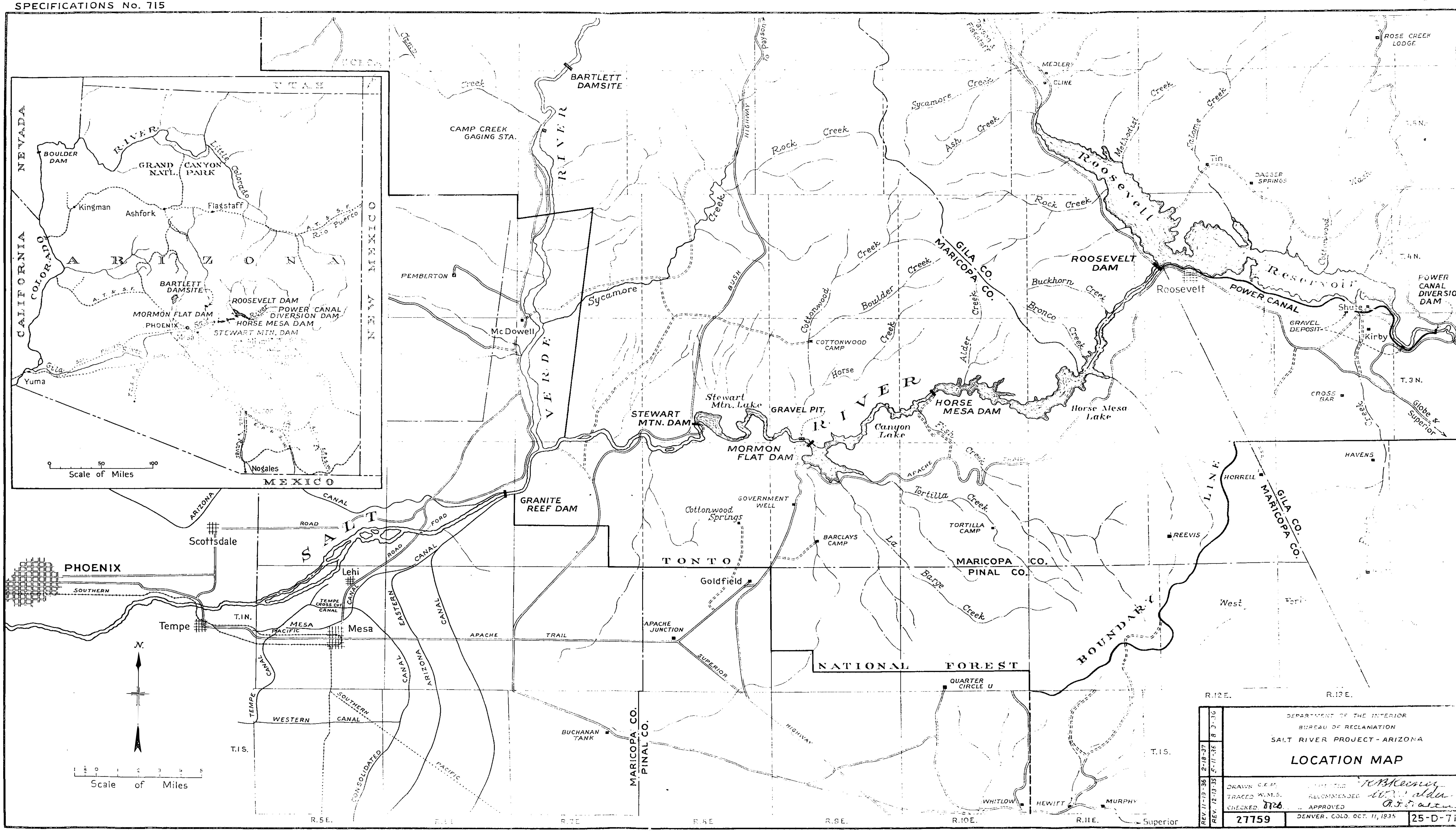
Redesign Before the January 1937 Flood

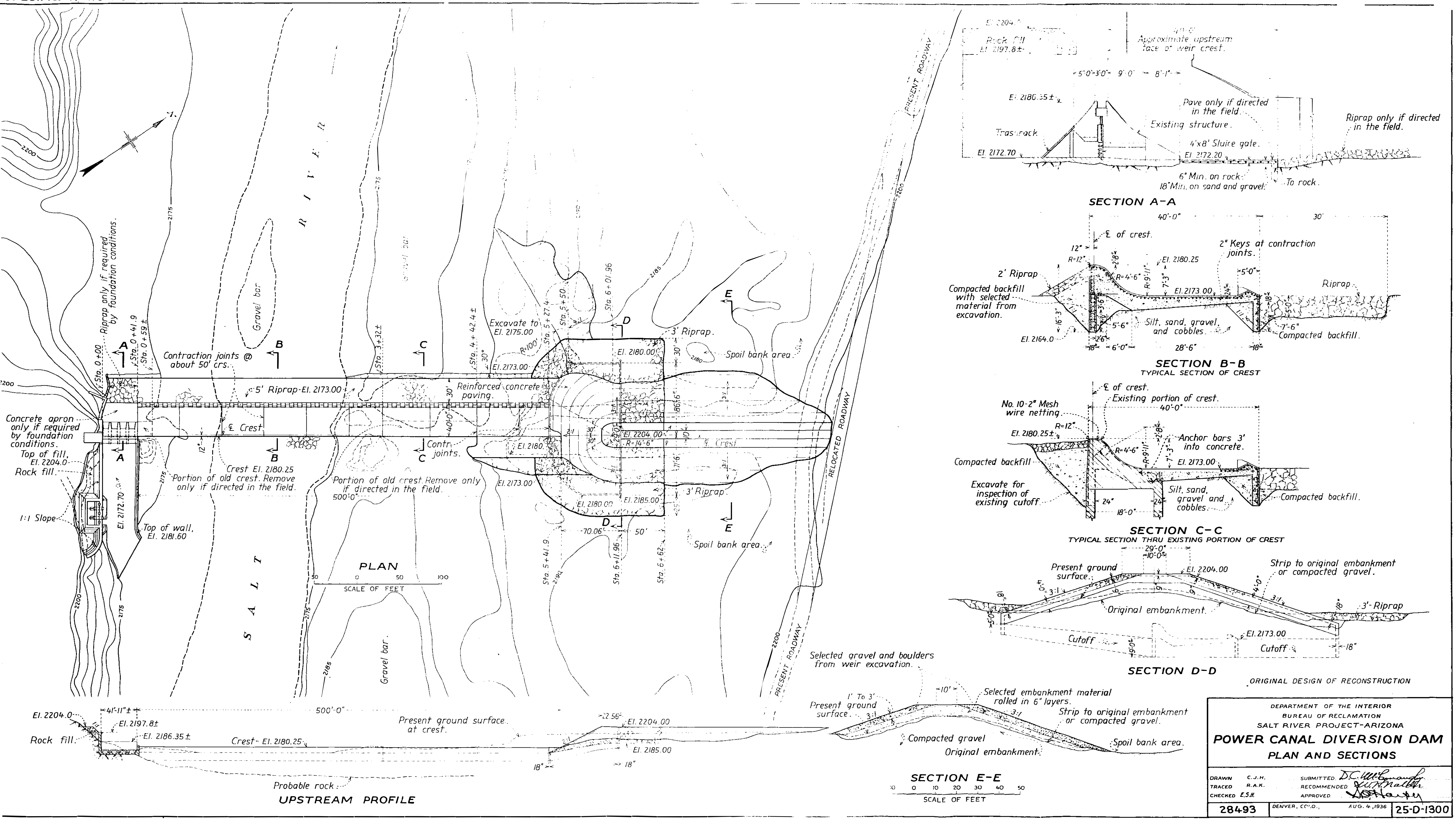
The Power Canal Diversion Dam in the Salt River about 100 miles east of Phoenix, Arizona, and 22 miles upstream from the Roosevelt Dam (Figure 1) was originally built during the period from 1903 to 1906 to divert water into a canal system for supplying water to turbines in a temporary plant at Roosevelt Dam where the electrical power generated was used to operate machinery during the construction of Roosevelt Dam. After the completion of the Roosevelt Dam, the system was adapted to maintain a constant head on certain power units in the permanent powerhouse during the seasonal fluctuation of the water surface in Roosevelt Reservoir.

In a flood in 1916, the dam was partially destroyed, and it was not until 1935 that plans were made for rebuilding it (Figure 2). A 1:24 model of the right abutment and the right end of the spillway was built in the Colorado Agricultural College Experiment Station Hydraulic Laboratory at Fort Collins, Colorado, in 1936 to obtain a design that would safely and efficiently handle a flood of 150,000 second-feet.

During the flood of 1916, one section of the spillway disappeared entirely, and no one could account for its location. During the 1937 flood, after the spillway had been originally reconstructed, this section was found downstream from the original alinement. Soundings below the reconstructed dam, after the 1937 flood, showed deep scour between the dam and the "lost section." Model studies, 1:24 scale, of this combination checked the field measurements and showed that continued flood flow would move the "lost section" not downstream but upstream. Undermining at the upstream side would disturb its equilibrium and cause it to roll toward the dam; in other words, an object too heavy to move downstream might roll upstream by undermining. To prevent such an incident at the prototype and its consequent endangerment of the reconstructed dam, the old section was removed.

FIGURE 1


UTAH
NEVADA
CALIFORNIA
NEW MEXICO
MEXICO
ARIZONA
BOULDER DAM
GRAND NAT'L. CANYON PARK
RIVER
Little Colorado
COLORADO
Kingman
Ashfork
Flagstaff
Rio Puerco
BARTLETT DAMSITE
ROOSEVELT DAM
POWER CANAL DIVERSION DAM
MORMON FLAT DAM
HORSE MESA DAM
PHOENIX
STEWART MTN. DAM
Yuma
Gila
Nogales
Scale of Miles
BARTLETT DAMSITE
CAMP CREEK GAGING STA.
VERDE RIVER
PEMBERTON
McDowell
Sycamore
Creek
Rock Creek
Sycamore Creek
Ash Creek
MEDLER
CLINE
ROSE CREEK LODGE
Tin
DAGGER SPRINGS
Roosevelt
Reservoir
Rock Creek
GILA CO.
MARICOPA CO.
ROOSEVELT DAM
Roosevelt
POWER CANAL
POWER CANAL DIVERSION DAM
Shute
Kirby
GRAVEL DEPOSIT
Buckhorn Creek
Bronco Creek
Cottonwood
Boulder Creek
COTTONWOOD CAMP
Horse
Alder Creek
Stewart Mtn. Lake
STEWART MTN. DAM
GRAVEL PIT
RIVER
HORSE MESA DAM
Horse Mesa Lake
Canyon Lake
MORMON FLAT DAM
CROSS BAR
Globe & Superior
T. 4 N.
T. 3 N.
HAVENS
HORRELL
GILA CO.
MARICOPA CO.
APACHE
Tortilla Creek
TORTILLA CAMP
GOVERNMENT WELL
Cottonwood Springs
BARCLAYS CAMP
La Barge Creek
REEVIS
MARICOPA CO.
PINAL CO.
TONTO
NATIONAL FOREST
BOUNDARY
LINE
West
GRANITE REEF DAM
SALT
CANAL
ROAD
ARIZONA
Scottsdale
PHOENIX
SOUTHERN
Lehi
TEMPE CROSS CUT CANAL
Tempe
T. 1 N.
MESA
PACIFIC
Mesa
EASTERN CANAL
ARIZONA CANAL
APACHE TRAIL
Goldfield
APACHE JUNCTION
SUPERIOR
HIGHWAY
BUSH HIGHWAY
To Payson
QUARTER CIRCLE U
N.
TEMPE CANAL
WESTERN CANAL
SOUTHERN PACIFIC
CONSOLIDATED
T. 1 S.
BUCHANAN TANK
MARICOPA CO.
PINAL CO.
Scale of Miles
WHITLOW
HEWITT
MURPHY
Superior
R. 5 E.
R. 8 E.
R. 9 E.
R. 10 E.
R. 11 E.
R. 12 E.
R. 13 E.
T. 1 S.
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SALT RIVER PROJECT - ARIZONA
LOCATION MAP
DRAWN
TRACED
CHECKED
RECOMMENDED
APPROVED
27759
DENVER, COLO. OCT. 11, 1935
25-D-711
REV. 11-19-36
REV. 12-13-35
2-18-37
2-11-36


SECTION A-A
SECTION B-B
TYPICAL SECTION OF CREST
SECTION C-C
TYPICAL SECTION THRU EXISTING PORTION OF CREST
SECTION D-D
SECTION E-E
PLAN
SCALE OF FEET
UPSTREAM PROFILE
SALT RIVER
PRESENT ROADWAY
RELOCATED ROADWAY
Spoil bank area.
Gravel bar
Contraction joints @ about 50' crs.
5' Riprap-El. 2173.00
Reinforced concrete paving.
Crest El. 2180.25
Portion of old crest. Remove only if directed in the field.
Riprap only if required by foundation conditions.
Concrete apron only if required by foundation conditions.
Top of fill, El. 2204.0
Rock fill.
1:1 Slope
Top of wall, El. 2181.60
Excavate to El. 2175.00
3' Riprap.
Approximate upstream face of weir crest.
Pave only if directed in the field.
Existing structure.
4'x8' Sluice gate.
Trashrack
Riprap only if directed in the field.
6" Min. on rock.
18" Min. on sand and gravel.
To rock.
2" Keys at contraction joints.
2' Riprap
Compacted backfill with selected material from excavation.
Silt, sand, gravel, and cobbles.
Compacted backfill.
No. 10-2" Mesh wire netting.
Existing portion of crest.
Anchor bars 3' into concrete.
Excavate for inspection of existing cutoff.
Present ground surface.
Strip to original embankment or compacted gravel.
Original embankment.
Cutoff
Selected gravel and boulders from weir excavation.
Selected embankment material rolled in 6" layers.
Compacted gravel
Present ground surface at crest.
Probable rock
ORIGINAL DESIGN OF RECONSTRUCTION
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SALT RIVER PROJECT-ARIZONA
POWER CANAL DIVERSION DAM
PLAN AND SECTIONS
DRAWN C.J.H.
TRACED R.A.K.
CHECKED E.S.H.
SUBMITTED
RECOMMENDED
APPROVED
28493
DENVER, COLO.
AUG. 4, 1936
25-D-1300

As an accurate tailwater rating curve prepared from observations in the field was not available, several curves were computed using various values of n and s in Manning's formula. The original tailwater curve referred to in Figure 3 as the minimum estimated, has n = 0.03 and s = 0.0019. The composition of the riverbed, however, indicated that retrogression might further decrease the tailwater elevation. For this reason, an arbitrary curve 2 feet below the minimum estimated was used to develop an apron and right abutment.

An interesting and rather startling incident, impossible to observe on the prototype because of the turbid condition of the flood water, was witnessed in the 1:24 model. The river above the dam carries a heavy bed load, and during a flood bars form across the dam completely covering it for short intervals. In the clear water of the model it could be seen that holes were scoured to a depth of 12 feet along the upstream face of the spillway crest. The velocity of approach was high due to the shallow channel, and as the water passed over the crest an eddy formed below the upstream edge. This eddy picked up bed material near the upstream face of the dam and carried it downstream. The pocket increased in size until the intensity of the eddy was decreased, and it could no longer pick up material. The hole then gradually filled again from material being moved along by the stream, but while a particular hole was filling, another would be forming elsewhere. As a hole became filled, the cycle would be repeated. Examination of portions of the original dam remaining in place disclosed scour to a depth sufficient to confirm the observations in the model. Based on these facts, there is reason to believe that one of the major factors of the 1916 failure was piping under the dam due to the reduction of percolation length by the formation of the holes upstream. Only one section of the dam, the "lost section," was moved any distance from its original position. Assuming that the major cause was piping, that one section was undermined and literally skidded downstream where it came to rest tilted upstream. In the original 1937 redesign, the riverbed was heavily riprapped upstream from the dam to stabilize it against a recurrence of this failure. Immediately after completion of the original reconstruction of the prototype, a flood of approximately 68,000 second-feet passed over the dam in January 1937. This flood was equal to or greater in magnitude than the one which had caused the 1916 failure. Subsequent examination of the riprapping immediately upstream of the dam face showed no disturbance.

Redesign After the January 1937 Flood

The meandering of the river upstream caused a concentration of flow near the intake section producing an extremely high headwater elevation in this region during the January 1937 flood. The riprap downstream from this structure was washed away, and the structure was considered to have inadequate protection. A 1:48 model of the complete

1:48

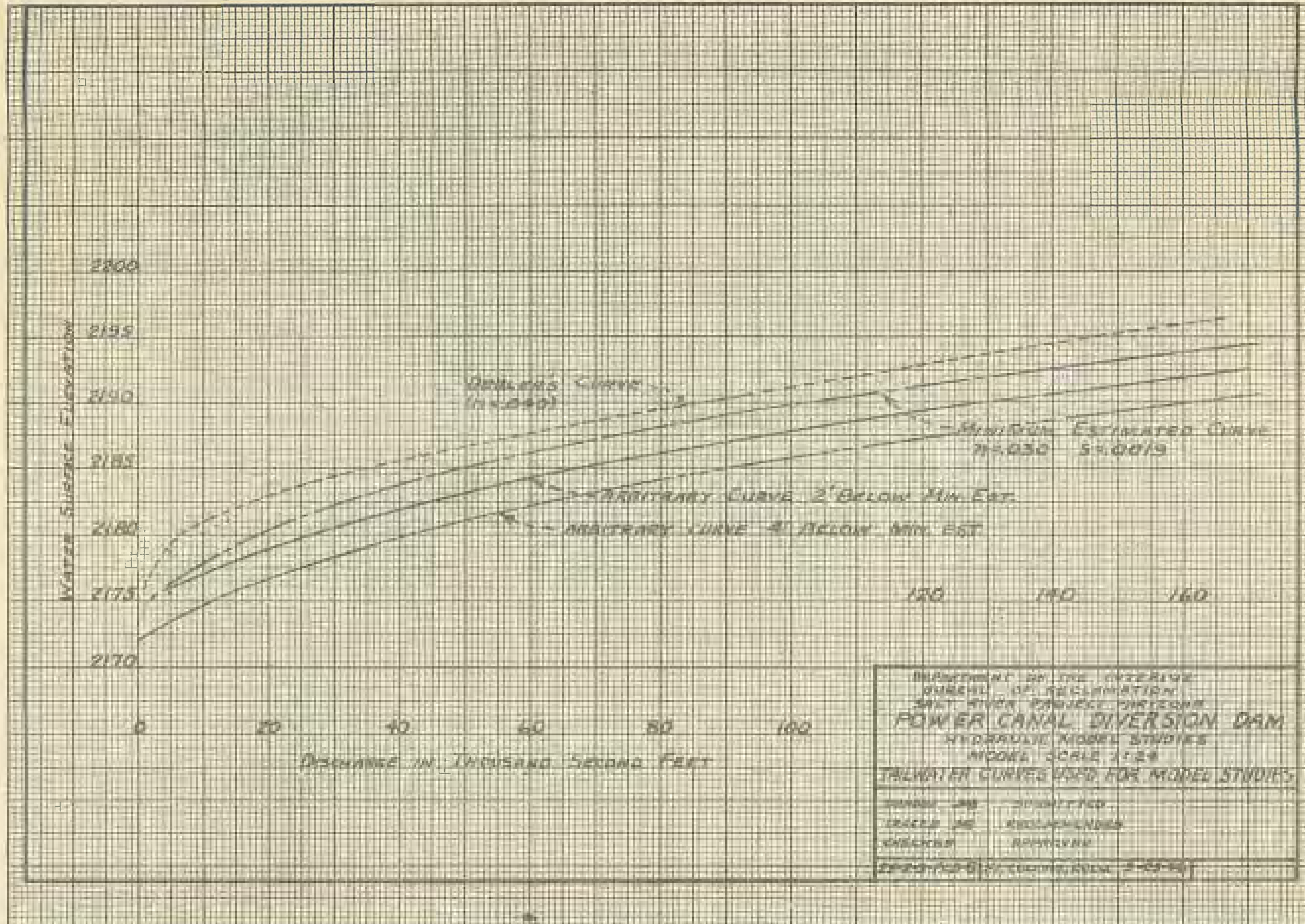

2200
2195
2190
2185
2180
2175
2170
Water Surface Elevation
0
20
40
60
80
100
120
140
160
Discharge in Thousand Second Feet
Minimum Estimated Curve
n=.030 S=.0019
Arbitrary Curve 2' Below Min. Est.
Arbitrary Curve 4' Below Min. Est.
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SALT RIVER PROJECT ARIZONA
POWER CANAL DIVERSION DAM
HYDRAULIC MODEL STUDIES
MODEL SCALE 1:24
TAILWATER CURVES USED FOR MODEL STUDIES

FIGURE 3

structure was built in March 1937, to ascertain the alterations necessary to give adequate protection against a flood of 150,000 second-feet. A tailwater curve approximately 4 feet lower than the minimum estimated (Figure 3) was used because of the uncertain and continually changing conditions in the field. 150,00

In the redesign subsequent to the flood of 1937 and subsequent to the model tests on the 1:48 model, additional protection was provided in the form of a sloping apron 25 feet long downstream from the original redesign of apron which extended to a point 30 feet downstream from the upstream face of the crest. Each of these sections had a Rehbock sill at its downstream end. To further minimize scour downstream from the right abutment, a solid sill was designed as shown in Figure 30. Reh Sill Fig 30

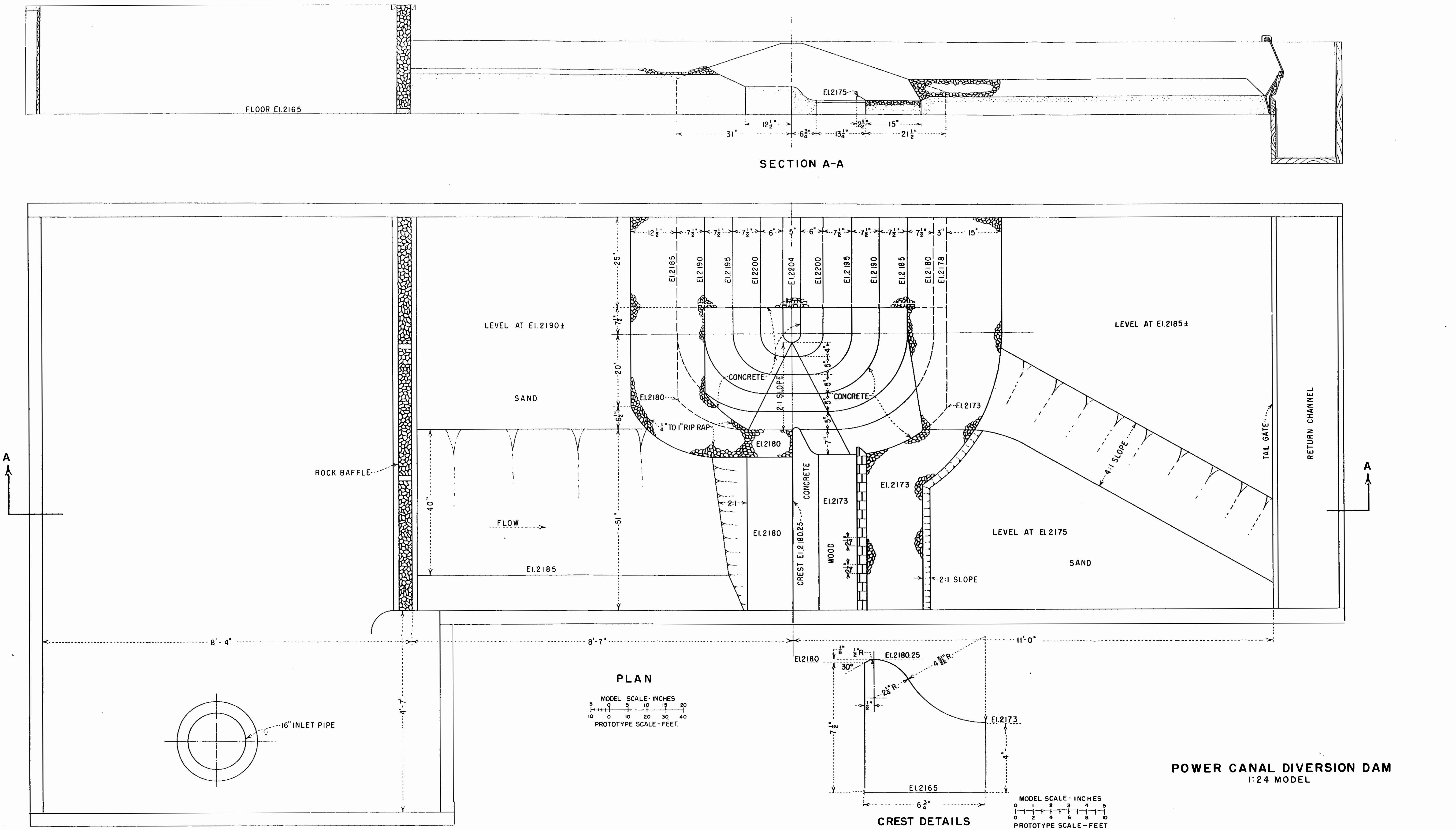
It was determined in the 1:48 model that an intermediate wall between the intake structure and the spillway would considerably minimize the scour downstream from that portion of the structure. In fact, it was considered essential to the safety of the structure if the riverbed were gravel and boulders, and it would substantially improve conditions should the foundation below the intake prove to be solid rock.

During the course of the 1:48 model studies, an articulated apron was studied in the hope that it would conform to the future retrogression. It was found, however, that the articulated apron had no advantages over a solid one when both were protected by sills at the downstream ends. So far as known, the present construction of the dam is as shown on Figure 30.

THE INVESTIGATIONS

Study of Upstream Protection in Sectional 1:24 Model of Spillway (1936)

The limited floor space in the Hydraulic Laboratory at the time the model tests were begun on the Power Canal Diversion Dam prevented the construction of a complete model of the structure. The model represented the right abutment and a short section of the spillway crest and was constructed in opposite hand as a matter of convenience in location in the laboratory. The scale ratio of 1:24 was governed by the capacity of the laboratory pump and by the length of crest considered necessary to give representative flow conditions. The model, constructed of metal bents and concrete, was built in a metal-lined flume (Figure 4). Provision was made for installation of an intermediate wall to create a sectional model of the spillway crest. The riverbed upstream and downstream was represented by coarse sand. Water was measured over a 2-foot Cipolletti weir supplied to the model through a 16-inch outlet from the laboratory



POWER CANAL DIVERSION DAM
1:24 MODEL

supply system. The flow passed over the spillway and tailwater regulator into a channel where it was returned to the supply system.

Tests 1 to 13, inclusive, concerned protection upstream from the spillway crest and were conducted on the 1:24 sectional model of a typical cross-section of the dam. The model represented about 40 feet of the prototype crest length and approximately 1/10 of a mile of the streambed. The purpose of the tests on this model was to study the flow conditions and to determine the alterations necessary to give adequate protection against scour adjacent and upstream to the structure.

Test 1 (Figure 5) was made on the original crest with no riprap upstream. The need for protection was clearly indicated when at 100,000 second-feet, with a tailwater 2 feet above the minimum estimated, holes varying from 5 to 10 feet in depth (prototype) were continually being dug and filled. This same phenomenon occurred during preliminary studies on the 1:48 model (Figure 6). In Test 2, with the high tailwater, a 2-foot blanket of riprap extended 12 feet upstream at elevation 2180, practically eliminating the movement of the upstream material.

In Test 3, the depth of the riprap area was increased to 4 feet (Figure 5), and the minimum estimated tailwater used. Two identical runs on this arrangement gave different results. In the first, most of the riprap was washed quickly over the crest between flows of 60,000 and 100,000 second-feet; in the second, less rock went over the crest and that remaining on the upstream side settled on approximately a 1-1/2:1 slope away from the crest.

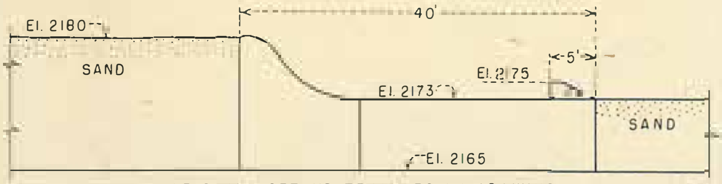
In Test 4, a 2-foot thickness of riprap placed on a 2:1 slope from elevation 2180 to 2175 and extended 15 feet upstream, gave considerable improvement (Figure 5). The material was disturbed by the digging action, but only that adjacent to the crest was washed downstream.

In Test 6, the riprap contiguous to the crest was lowered 2 feet and extended to 25 feet upstream (Figure 5). This arrangement was considered satisfactory when only one or two small rocks were moved over the crest.

In Test 8, the riprap width was reduced to 20 feet to ascertain if this reduction were feasible. Less desirable conditions resulted, and the 25-foot width was used in all subsequent tests. A more complete investigation on a wider sectional model with a deeper sand bed and the rock thickness increased to 3 feet (Test 33, Figure 7) showed that excellent conditions existed on the upstream side of the dam for all flows up to and including 150,000 second-feet with tailwater elevation 2 feet below the minimum estimated. In later tests the stability of the material upstream with a tailwater 4 feet below the minimum estimated also

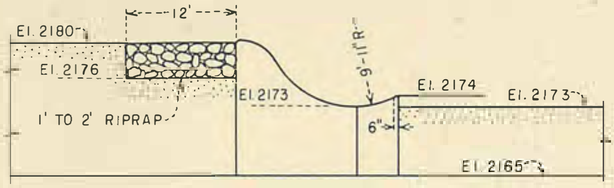
DS scour

FIGURE 5

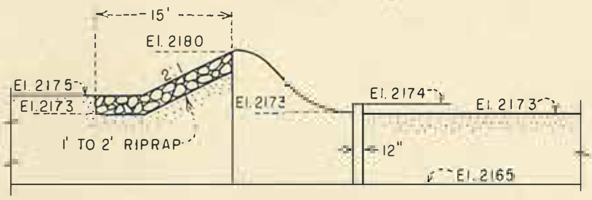


ORIGINAL SET-UP FROM DESIGN DRAWING

TEST 1

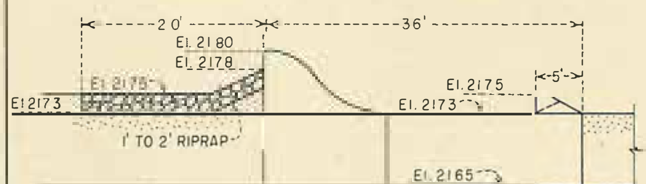


TEST 3

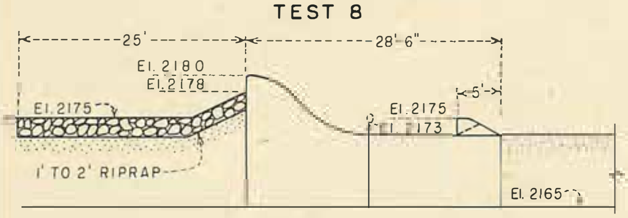


TEST 4

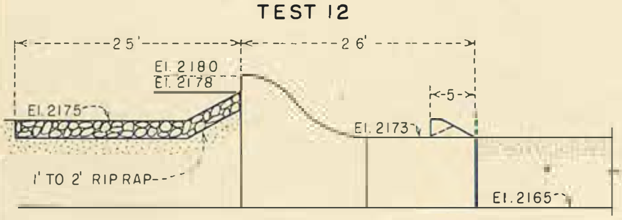
TEST 6



TEST 8



TEST 12



TEST 13

TEST 1

Observations were made at various discharges from 5,000 to 150,000 second-feet. The tailwater curve used was two feet higher than the minimum estimated. From 5,000 to 30,000 second-feet the flow over the crest was below critical with no erosion downstream of sill and very little upstream of the crest. Above 30,000 the jump was submerged and sand was deposited downstream of the crest and on the apron. Severe erosion occurred upstream of crest. This erosion appeared the most severe at a discharge of 100,000 second-feet. Holes varying from five to ten feet deep were being continuously dug and filled upstream of the crest.

NOTE

FOR ALL SUBSEQUENT TESTS OBSERVATIONS WERE MADE AT DISCHARGES OF 5,000, 15,000, 30,000, 50,000, 100,000 AND 150,000 SECOND-FEET AND THE MINIMUM TAILWATER CURVE USED. EACH DISCHARGE WAS RUN APPROXIMATELY FIVE MINUTES.

TEST 3

Two identical runs were made. In one, at approximately 60,000 second-feet, the riprap on the right half of the test section went over the crest leaving a hole 12 feet deep which soon filled with sand. By the time 100,000 second-feet was reached, most of the riprap had gone over the crest. Holes varying from 5 to 10 feet deep were continuously dug and filled upstream of the crest. No serious erosion occurred downstream of the sill.

In the other run the erosion upstream was more gradual causing the riprap to settle on approximately $1\frac{1}{2}$:1 slope upstream from the crest. Less riprap went over the crest and the erosion downstream became severe at 60,000 second-feet. The floor at elevation 2165 was swept clean at a point 24 feet downstream from the bucket lip. In the first run the severe erosion upstream caused less erosion downstream. In the second the moderate erosion upstream caused more severe erosion downstream.

TEST 4

The upstream erosion was improved over test 3. Some riprap was still being carried over, particularly that immediately upstream from the crest. Erosion downstream was very severe. At 50,000 second-feet the floor at elevation 2165 was swept clean from 12 to 24 feet downstream from the sill.

TEST 5

Set-up same as test 4 except the square sill replaced by Rehbock sill. The results were the same as test 4 indicating that apron was too short.

TEST 6

The upstream conditions were very satisfactory. Only two pieces of riprap went over the crest. At the completion of the test the riprap had settled approximately 2 feet. There was no serious erosion downstream from the sill at any flow. The worst scour conditions occurred at a discharge of 50,000 second-feet. The apron appeared to be longer than necessary.

TEST 7

Same as test 6 except upstream topography raised to elevation 2180. The results were the same as in test 6.

TEST 8

Apron shortened 4 feet.

The conditions downstream were as good or better than test 6. There was some undermining at the upstream edge of the riprap. It appears that the 25 feet of riprap upstream would be safer, also that more scour below the apron would result when no sand was carried over the crest.

TEST 9

Same as test 8 except sand was removed from the upstream side of crest. More severe scour resulted below the apron.

TEST 10

Apron shortened 6 feet

Riprap extended 25 feet upstream and dropped to elevation 2178 immediately upstream from crest. No apparent change in erosion due to shortening of the apron.

TEST 11

Apron shortened $8\frac{1}{2}$ feet.

Same as test 10.

TEST 12

Apron shortened $11\frac{1}{2}$ feet.

The surface of the jump at a discharge of 30,000 second-feet was very rough, otherwise the erosion and flow conditions were satisfactory at all discharges.

TEST 13

Apron shortened 14 feet.

Much erosion occurred downstream from the sill at 30,000 second-feet. This become worse at 50,000. The apron was evidently too short.

[SECTION WAS 20" (MODEL) WIDE]

UNITED STATES
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SALT RIVER PROJECT — ARIZONA

POWER CANAL DIVERSION DAM

HYDRAULIC MODEL STUDIES
MODEL SCALE 1:24
SUMMARY OF TESTS 1 TO 13 INCLUSIVE

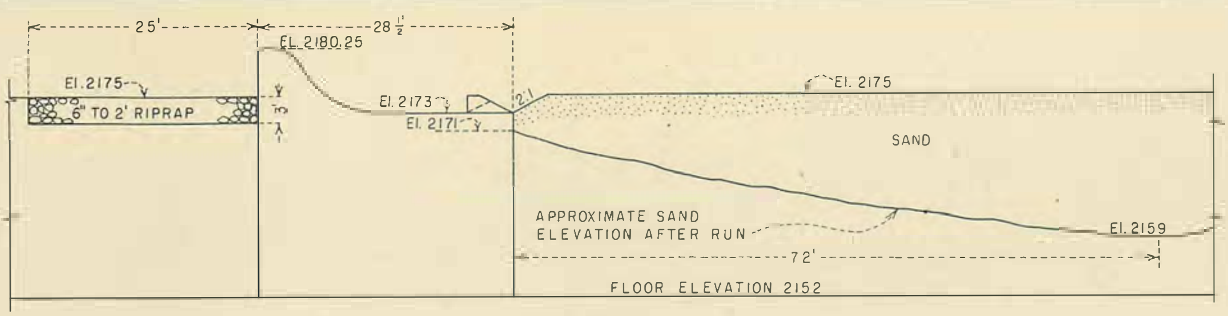
DRAWN ... SUBMITTED ...
TRACED ... RECOMMENDED ...
CHECKED ... APPROVED ...

25-2-3-PCD-3 FORT COLLINS, COLO. — AUG. 20, 1936

PITS ADJACENT TO UPSTREAM FACE OF DAM NEAR THE LEFT END.

PITS AT LEFT END OF SPILLWAY AND ADJACENT TO THE INTAKE WORKS.

PITS WHICH WERE CONTINUALLY BEING DUG AND FILLED
THE UPSTREAM FACE OF SPILLWAY DURING A DISCHARG
130,000 SECOND-FEET WITHOUT RIPRAP IN PLACE.



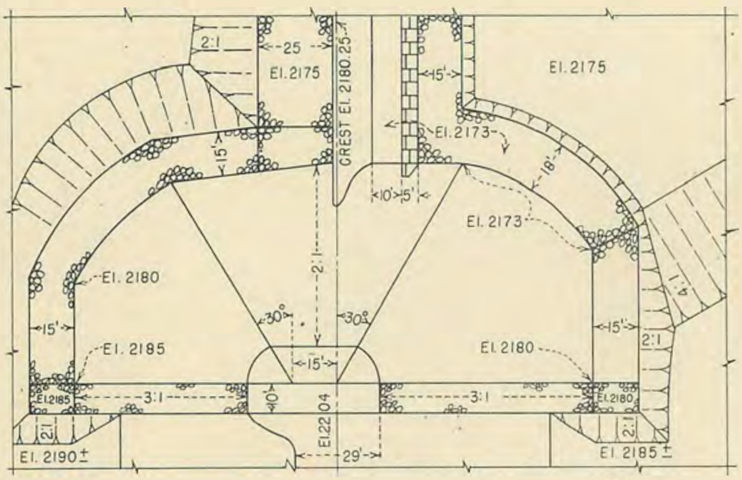
TEST 33

TEST 33

Set-up as shown in the sketch. A test section 48 feet wide was used. Discharges of 10,000, 25,000, 50,000, 100,000 and 150,000 second-feet were run. Three different tailwater curves were used: Debler's curve, minimum estimated curve and a curve two feet less than the minimum estimated (as per drawing "Tailwater Estimates", by C.J.H. dated 9-9-35). Each discharge and tailwater was run until a fairly constant condition was reached. The results were satisfactory for all tailwater elevations and discharges except for a discharge of 150,000 second-feet with a tailwater two feet below the minimum estimated. At that tailwater the roll formed 8 feet downstream from the sill. The sand adjacent to the sill was in constant movement to a depth of approximately five feet below the apron (elevation 2173). When the run was completed the sand was at elevation 2171 just downstream of the sill. The bed had scoured to an elevation of 2159 at a distance of 72 feet downstream from the sill.

TEST 34

Same set-up as test 33 except three feet of riprap was placed for 30 feet down-stream of the sill. The riprap varied from six inches to two feet in size. Runs were made similar to those of test 33. There was no appreciable difference from the results obtained in test 33. At 150,000 second-feet the riprap was carried to elevation 2160 from a point 28 feet downstream of the sill to a point 76 feet downstream of the sill.



TEST 35

TEST 35

Set-up as shown in sketch. This abutment was the same as that on the specification drawing #25-D-1300 except the upstream part of the abutment was moved 15 feet upstream. The same discharges and tailwater curves were used as in test 33. The conditions on the model were satisfactory up to and including 100,000 second-feet. At 150,000 second-feet satisfactory conditions existed with the minimum estimated tailwater and with the tailwater elevation according to Debler's curve. With a tailwater one foot below the minimum estimated, the riprap at the downstream corner of the abutment started to wash out. With a tailwater two feet below the minimum estimated, a hole was scoured to elevation 2154 in 50 minutes. The hole was approximately 20 feet wide and 48 feet long with the upstream end about eight feet downstream from the end of the abutment.

TEST 36

Same set-up as test 35 except the cut off wall around the abutmet was raised vertically to elevation 2204. The same procedure was followed as in test 35. At high discharges with the tailwater two feet below the minimum estimated, the standing wave formed several feet downstream from the end of the sill. Apparenly the apron was too short for the low tailwater. Good results were prevalent at the lower tailwater elevotions up to and including 100,000 second-feet. A discharge of 150,000 second-feet with tailwater at minimum estimated and above, for 25 minutes, gave very good results. An additional 35 minutes with the tailwater lowered two feet gave considerably more erosion. The sand and riprap settled two feet to four feet below the apron (elevation 2173) just abutment was slightly greater than at any other point.
At the end of the run the scour was to elevation 2155 at a point 42 feet downstream from the sill.

TEST 37

Set-up same as test 36 except the apron was lengthened $11\frac{1}{2}$ feet, making its total length $26\frac{1}{2}$ feet. The model discharge was gradually increased to 150,000 second-feet. This discharge, with the tailwater two feet below the minimum estimated, was maintained for 30 minutes. The scour was much less than in test 36. The sand and riprap settled to elevation 2172 immediately downstream from the sill. The scour was to elevation 2160 at a point 28 feet downstream from sill.

TEST 38

Same as test 35 except the 2:1 paved abutment slope was changed to a $1\frac{1}{2}:1$ slope. The same procedure was followed as in test 35. The steeper slope caused more erosion downstream from the abutment. The riprap near the downstream corner of the abutment commenced to roll at 100,000 second-feet, with the tailwater elevation according to Debler's curve. The lower tailwater elevations increased the erosion. A flow of 150,000 second-feet for a period of 40 minutes scoured a hole 35 feet wide by 72 feet long to the floor at elevation 2154. The upstream end at this elevation was about 50 feet from the end of the apron. The sand and riprap immediately downstream from the apron was washed out and settled to elevation 2167.

UNITED STATES
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SALT RIVER PROJECT – ARIZONA

POWER CANAL DIVERSION DAM

HYDRAULIC MODEL STUDIES

MODEL SCALE 1:24

SUMMARY OF TESTS 33 TO 38 INCLUSIVE

25-2-3-PCD-7 FT. COLLINS, COLO. SEPT-15-36 indicated ample protection. The upstream arrangement, as shown in Test 6, except with 3 feet of riprap was used in the final design of the prototype (Section C-C, Figure 30).

Study of Protection Downstream from Spillway in 1:24 Sectional Model (1936)

Preliminary visual tests concerning the protection downstream from the spillway indicated that the original design of cross-section of the dam (as built in 1906) would serve for all discharges if sufficient tailwater depth were maintained. Since there was no accurate tailwater rating curve, the structure was at first studied using the minimum estimated tailwater curve (Figure 3).

In Test 1, with a 2-foot Rehbock sill at the end of the apron 40 feet downstream from the upstream face of the dam (Section B-B, Figure 2, and Test 1, Figure 5), there was practically no scouring of the riverbed for any discharge, indicating that the apron was too long.

In Test 2, the apron was removed and a sill placed immediately downstream from the original crest section with a 10-foot strip of riprap 4 feet in thickness immediately downstream from it. This arrangement gave practically the same results. Subsequently, the tailwater elevations used in the first two tests were found to be 2 feet above the minimum estimated, and the satisfactory results were attributed to this fact.

An upward sloping curve added to the downstream end of the crest section to form a bucket (Test 3, Figure 5) was tested with the minimum estimated tailwater, and no riprap downstream from the dam. Two identical runs on this arrangement gave different results. In one the rapid movement of the material from the upstream side of the dam replaced that which was washed away downstream with the result that there was very little scouring indicated. In the other, a more gradual transportation of material from upstream allowed the scour to reach the floor of the model, elevation 2165, 24 feet downstream from the edge of the bucket in a very short time. The conditions in both cases were obtained between flows of 60,000 and 100,000 second-feet.

A 1-foot square sill placed at the downstream edge of the crest (Test 4, Figure 5) was very unsatisfactory as was a 2-foot Rehbock sill placed in the same position. At 50,000 second-feet severe erosion occurred, which exposed a large area of the model floor.

As the original design of apron arrangement with an overall length of 40 feet had not been tested with the minimum estimated tailwater, it was reinstalled (Test 6, Figure 5). Very little erosion occurred at any flow up to and including 150,000 second-feet. Thus, the apron appeared longer than necessary. The apron was shortened by increments to 36, 34, 31-1/2, 28-1/2, and 26 feet (Tests 8 to 13, inclusive, Figure 5). The scour increased as the apron length decreased, but did not appear serious until the 26-foot dimension was reached. The 28-1/2-foot length (Test 12) was considered satisfactory, although considerable erosion occurred when no material was passing over the crest. To be on the safe side, an overall length of 30 feet was recommended (Section B-B, Figure 27).

Pressures and Water Surface Profiles

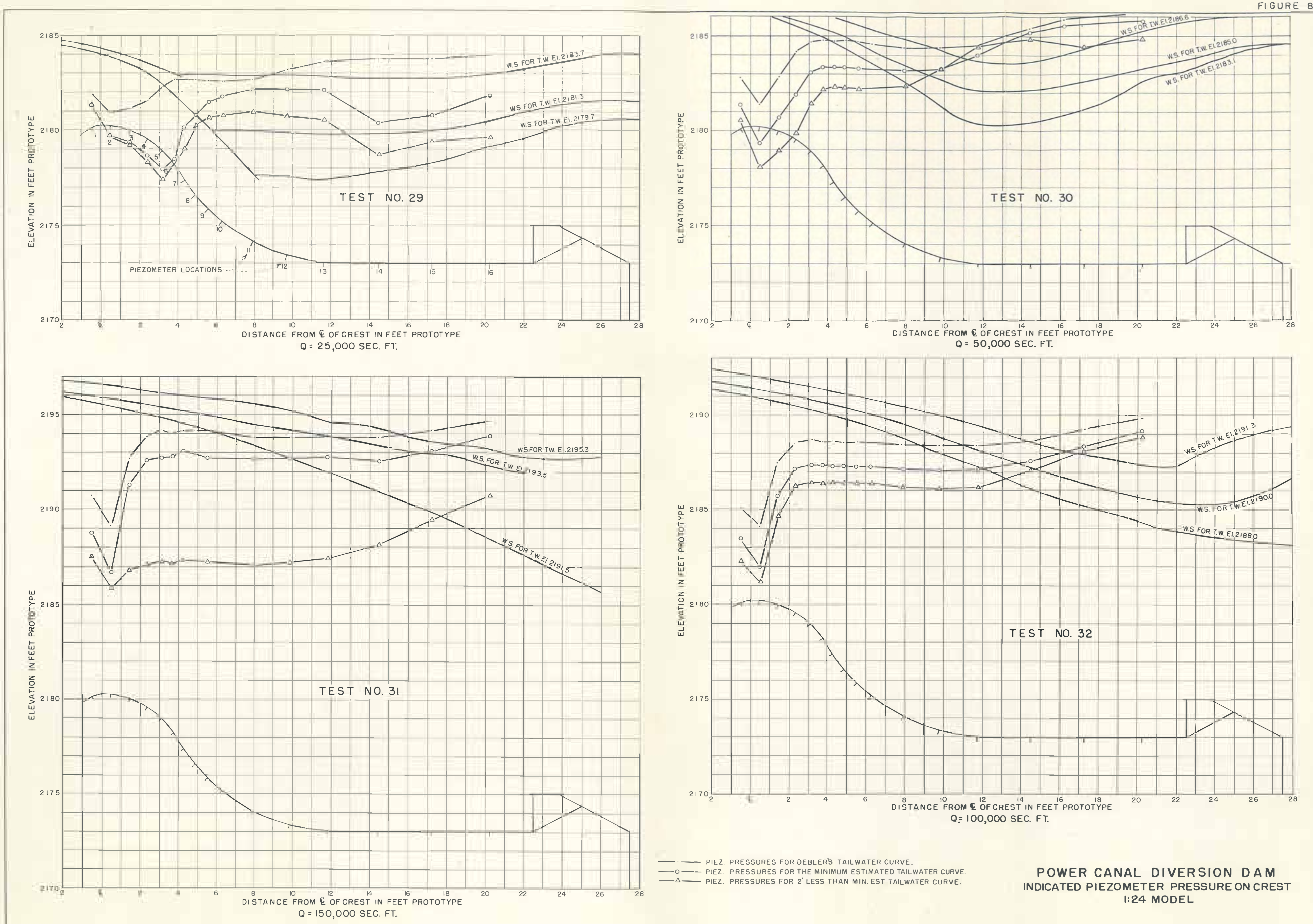
Water surface profiles and pressures over the crest (Tests 29 to 32, inclusive) were taken for different tailwater elevations and discharges, using the 28-1/2-foot length of apron. Slight negative pressures (Figure 8) occurred on the crest at discharges of 25,000 and 50,000 second-feet, but became positive for all higher discharges.

Model Study of Right Abutment to Scale of 1:24 (1936)

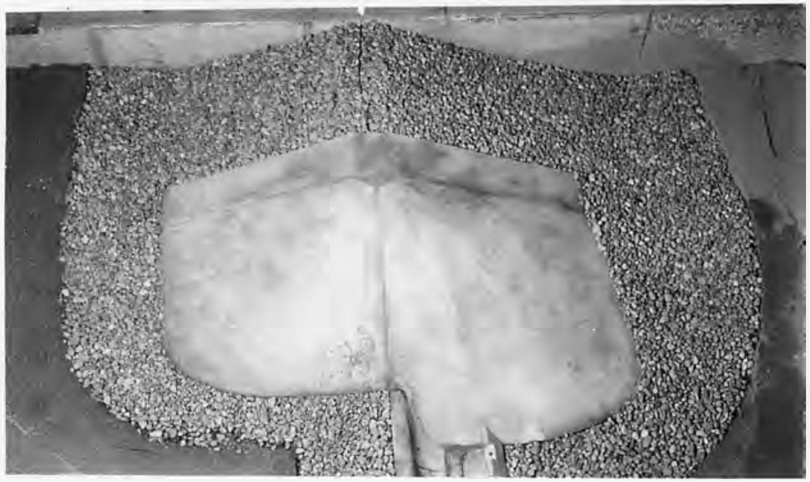
The intermediate wall in the model was removed, giving an arrangement representing the right abutment according to the original design and about 60 feet of the adjacent prototype crest. Tests 14 to 28 were made to study the flow conditions in the vicinity of the right abutment to determine necessary alterations to prevent serious erosion. The minimum estimated tailwater curve was used. Severe scour occurred downstream (Test 14) from the end of the apron and along the cutoff wall of the original abutment with a discharge of 150,000 second-feet (Figure 9-A and B). Visual tests indicated that improvement might be obtained by extending the upstream portion of the abutment. It was extended upstream in increments of 10, 10, and 5 feet (Figure 10). No appreciable change resulted after the first extension. The second, however, gave more acceptable conditions, while the third proved entirely satisfactory (Figure 9-C and D).

During these investigations, all the riprap covering the sloping concrete surface of the right abutment was washed away except that on the downstream corner. The surface of the broken rock was therefore lowered to the elevation of the top of the cutoff wall. The strips of paving on the 3:1 slopes on the upstream and downstream sides of the abutment next to the bank were not considered necessary so were replaced by riprap which proved very satisfactory. These slopes were therefore riprapped for all subsequent tests.

Shortly after obtaining a reasonable design of the right abutment capable of withstanding a flow of 150,000 second-feet with the minimum estimated tailwater curve, additional information on the characteristics of the riverbed indicated the tailwater might be lower than used in previous tests. The model was then tested (Test 19) using an arbitrary tailwater curve 2 feet below the minimum estimated, Figure 3. Downstream material, including riprap, was carried away, exposing the downstream cutoff wall to a considerable depth. Improvement occurred when the



POWER CANAL DIVERSION DAM
INDICATED PIEZOMETER PRESSURE ON CREST
1:24 MODEL



A. RIVER BED BEFORE RUN.

B. RIVER BED AFTER RUN.

ORIGINAL DESIGN.

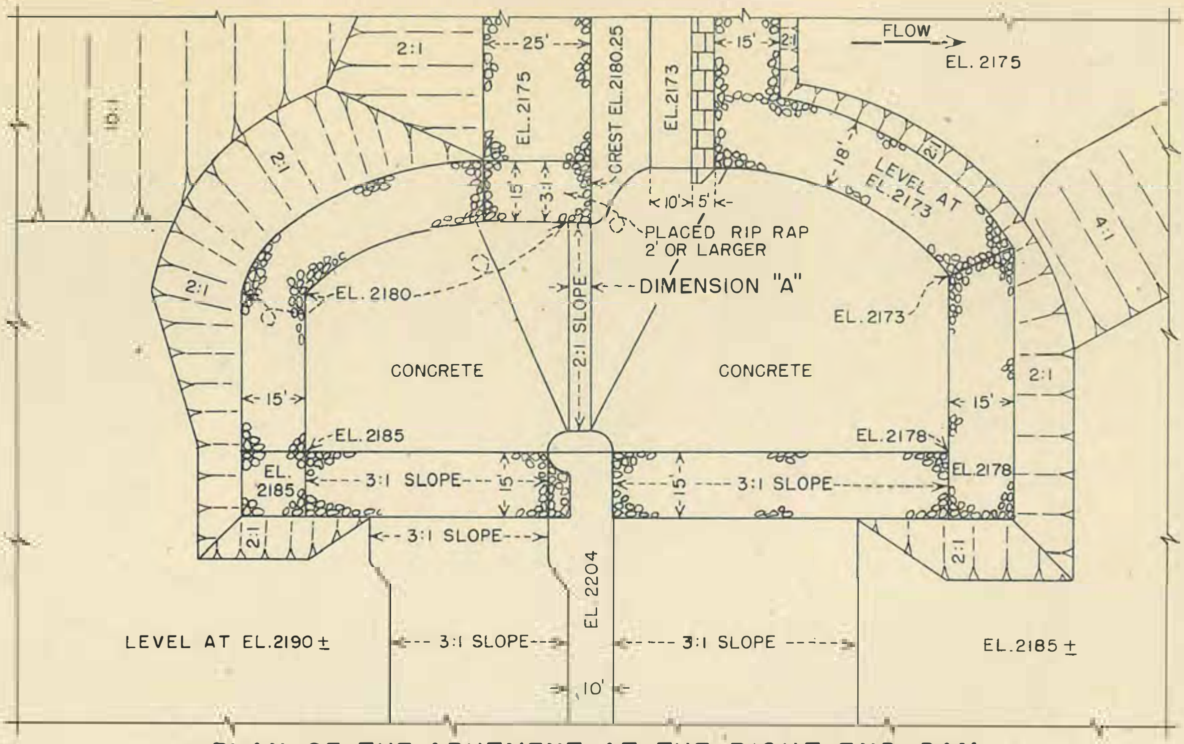
C. RIVER BED BEFORE RUN.

D. RIVER BED AFTER RUN.

ORIGINAL DESIGN WITH UPSTREAM SECTION OF ABUTMENT MOVED 25 FEET UPSTREAM.

EFFECT OF LENGTHENING UPSTREAM SECTION OF RIGHT ABUTMENT
DISCHARGE 150,000 SECOND-FEET - MINIMUM ESTIMATED TAILWATER.

Figure 9



PLAN OF THE ABUTMENT AT THE RIGHT END DAM

TEST 16
DIMENSION "A" EQUALS 10 FEET

The discharge was brought up to 150,000 second-feet, stopping momentarily at 50,000 and 100,000 second-feet. The riprap (elevation 2173), adjacent to the concrete abutment and approximately 50 feet downstream from the sill, started to wash out In less than two minutes a hole was dug, in this area, to the model floor at elevation 2165. It was evident that the upstream abutment should be moved further upstream. Except as noted, the flow about the rest of the model was entirely satisfactory.

TEST 17
DIMENSION "A" EQUALS 20 FEET

The discharge was brought up to 150,000 second-feet, stopping momentarily at 50,000 and 100,000 second-feet. Some of the riprap (elevation 2173), at the downstream corner of the abutment, washed out; the remainder settled until the top of it was approximately at 2170. The model was run for 45 minutes with no further change noted. The riprap in this area was between 6 inches and 2 feet in size. It seemed probable that if only 2-foot riprap had been used no erosion would have occurred.

TEST 18
DIMENSION "A" EQUALS 25 FEET

The discharge was brought up to 150,000 second-feet, stopping momentarily at 50,000 and 100,000 second-feet. Condition were very satisfactory. There was no tendency for the riprap (elevation 2173) along the edge of the downstream abutment to move. The model was run for sixty minutes with no change in condition.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SALT RIVER PROJECT-ARIZONA

POWER CANAL DIVERSION DAM
HYDRAULIC MODEL STUDIES
MODEL SCALE 1:24
SUMMARY OF TESTS 16 TO 18 INCL.

DRAWN J.M.B.	SUBMITTED
TRACED J.E.V. - V.G.M.	RECOMMENDED
CHECKED	APPROVED
25-2-3-PCD-1	FORT COLLINS, COLO. 8-22-36

downstream edge of the abutment was extended level at elevation 2173 (Figure 11), but prolonged operation produced severe erosion. The streamward side of the downstream abutment section was made vertical by removing the portion of the 2:1 paved slope below the crest (Figure 11), but this gave no improvement in scour conditions. Several minor alterations to the apron, including variable length sections in this vicinity gave slight improvement. However, the shallow sand bed below the model and the short approach to it were believed to contribute to the severe conditions, so the model was reconstructed before additional tests were conducted. This design, which had proven satisfactory with the minimum estimated tailwater was then considered inadequate when severe scour (Test 35) occurred along the downstream edge of the abutment (Figure 12-A and B).

Practically the same results were obtained with a vertical abutment (Figure 12-E and F). Slopes of 1-1/2:1 and 3:1 on the streamward side of the abutment gave more erosion than the original 2:1 slope, Figure 12-A and B and Figure 14-A and B. The 2:1 slope was used in all subsequent tests.

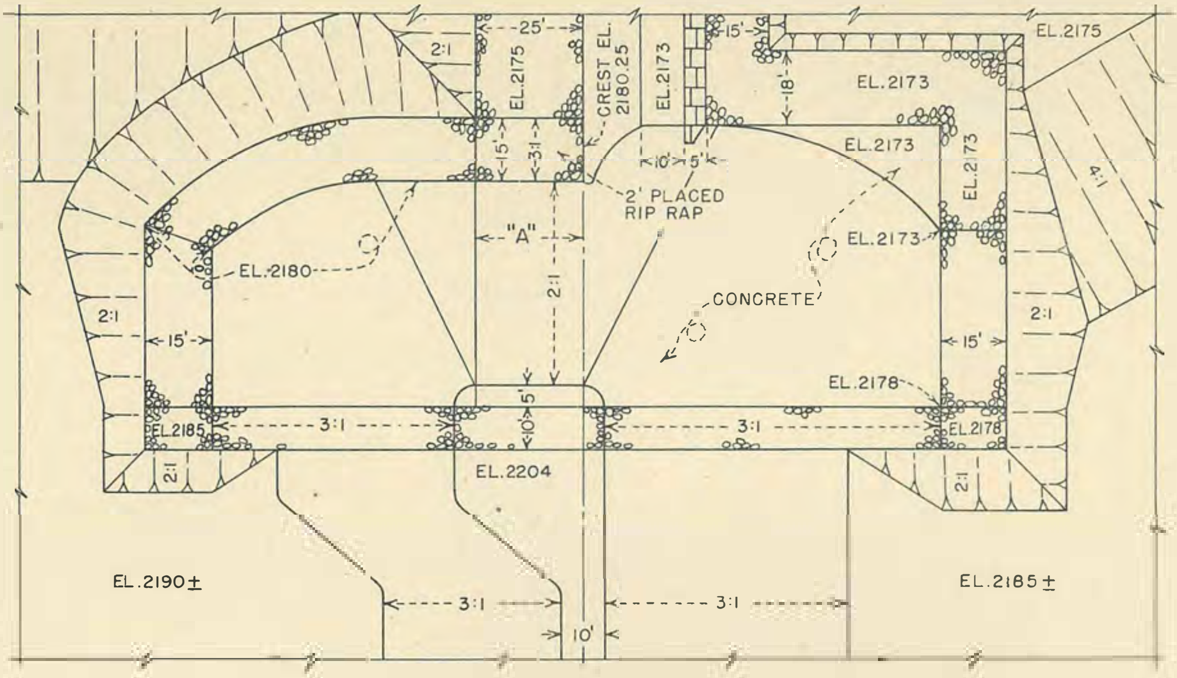
The investigation of numerous variable length aprons adjacent to the abutment (Figure 13) resulted in a design (Test 42) which would, without too much erosion, handle a flood of 150,000 second-feet (Figure 14-C, D, E, and F). There was very little scour with the minimum estimated and higher tailwater elevations. Lowering the tailwater 1 foot increased the erosion slightly, while dropping it an additional foot materially increased it. The results were not considered critical because the cutoff walls were not completely exposed and very little of the riprap adjacent to the structure was washed away, most of it settling as the fine sand was washed away from between the particles. This design (Test 42, Figure 13) was recommended and was embodied in the construction of the prototype during the Winter of 1936-37.

Field Data on February 1937 Flood

On February 7 and 8, 1937, a few days after completion of the reconstruction of the prototype, a flood with a peak capacity of 68,000 second-feet passed over the dam. The meandering of the river upstream caused the flood to be concentrated near the intake structure (Figure 15), thus distorting the relation between tailwater and headwater to such extent that the riprap protection downstream from the apron in this area was washed away. With the possibility of larger floods, these conditions were critical. A laboratory reinvestigation was necessary. A model was built to a scale of 1:48 to represent the entire spillway and the approach and tailwater conditions (Figure 16). This model was also constructed and studied in the Colorado Agricultural Experiment Station at Fort Collins, Colorado. As additional area was then available

TEST 19

Same set-up as test 18. A tailwater curve two feet lower than the minimum estimated (as per drawing "Tailwater Estimates", dated 9-9-35, C. J. H.), was used. No serious erosion occurred from 0 to 100,000 second-feet. At 150,000 second-feet, as in test 16, the region around the down-stream edge of the abutment was eroded to the model floor (elevation 2165). It required 15 minutes to erode to the floor as against two minutes in test 16. The magnitude of the erosion was about the same in both tests. The scour downstream of the sill was comparable with that of test 16 and was not considered serious.



PLAN OF ABUTMENT

TEST 20
DIMENSION "A" EQUALS 25 FEET

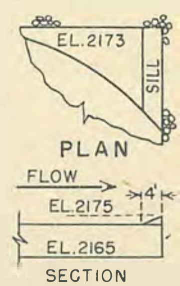
Set-up as shown in sketch. Same tailwater as Test 19. No serious erosion occurred from 0 to 100,000 second-feet. At 150,000 second-feet the sand downstream of the abutment started to erode. After 50 minutes at this discharge a hole, commencing 28 feet downstream of the abutment and 28 feet long, was scoured to the floor at elevation 2165.

TEST 21
DIMENSION "A" EQUALS 30 FEET

Set-up as shown in sketch. Same tailwater as test 19. Same results as test 20. No sand coming over the dam in tests 20 and 21.

TEST 22
DIMENSION "A" EQUALS 30 FEET

Set-up as shown in sketch except triangular sill two feet high was placed at the downstream edge of the level part of the abutment (elevation 2173). Same results as test 20 except the scour was more rapid.



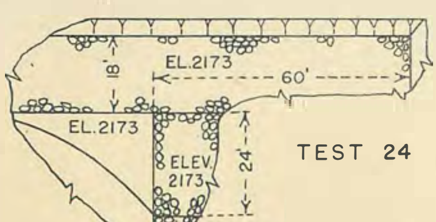
TEST 22

TEST 23

Same as test 21 except there was no riprap downstream of the crest. Erosion was more severe than in test 21.

TEST 24

Same as test 21 except an area 42' x 60' downstream from the abutment was riprapped three feet in depth, level at el. 2173. A discharge of 150,000 sec.-ft. for 90 minutes proved satisfactory. Riprap 20 feet downstream from edge of abutment settled to elevation 2171 and the floor was exposed beyond the end of the riprapped area.



TEST 24

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SALT RIVER PROJECT-ARIZONA

POWER CANAL DIVERSION DAM
HYDRAULIC MODEL STUDIES
MODEL SCALE 1:24
SUMMARY OF TESTS 19 TO 24 INCL.

DRAWN. J.M.B. .... SUBMITTED ....
TRACED. J.E.V.-V.G.M. ... RECOMMENDED ....
CHECKED .... APPROVED ....

25-2-3-PCD-4 FORT COLLINS, COLO. 8-27-36

Figure 12

A. RIVER BED BEFORE RUN.

B. RIVER BED AFTER RUN.

STREAMWARD SIDE OF ABUTMENT ON 3:1 SLOPE.

C. RIVER BED BEFORE RUN.

D. RIVER BED AFTER RUN.

STREAMWARD SIDE OF ABUTMENT ON 1½:1 SLOPE.

E. RIVER BED BEFORE RUN.

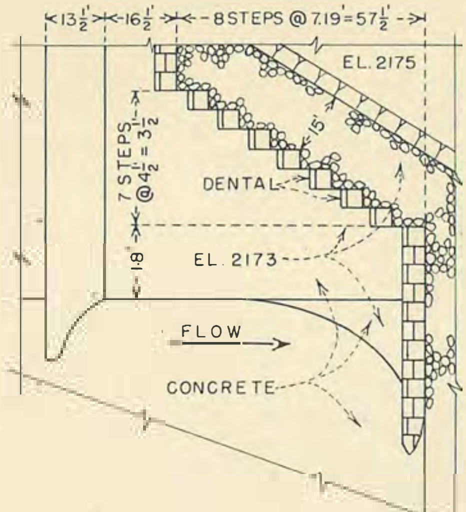
F. RIVER BED AFTER RUN.

STREAMWARD SIDE OF ABUTMENT VERTICAL

COMPARISON OF DIFFERENT ABUTMENT SLOPES - DISCHARGE 150,000 SECOND-FEET
WITH TAILWATER TWO FEET BELOW THE MINIMUM ESTIMATED.

TEST 39

Same setup as test 35, except the 2:1 paved abutmen slope was replaced by a 3:1 slope. The same procedure was followed as in test 35. Some erosion occurred 30 feet downstream from the end of the apron for 100,000 second-feet with the tailwater according to Debler's curve. The riprap along the downstream portion of the abutment rolled downstream and a hole 3 feet deep was formed. As the tailwater was lowered the hole became deeper but always reached a point of stability and never eroded deep enough to expose the bottom of the cutoff wall. With a discharge of 150,000 second-feet and the tailwater elevation according to Debler's curve a hole was scoured to the floor (elev. 2154) in 15 minutes. Ten minutes at the minimum estimated tailwater and ten minutes at two feet below minimum estimated increased this scour. At the end of the run an area 38 feet wide by 60 feet long with the upstream end about 70 feet downstream from the sill was scoured to the floor (elevation 2154).



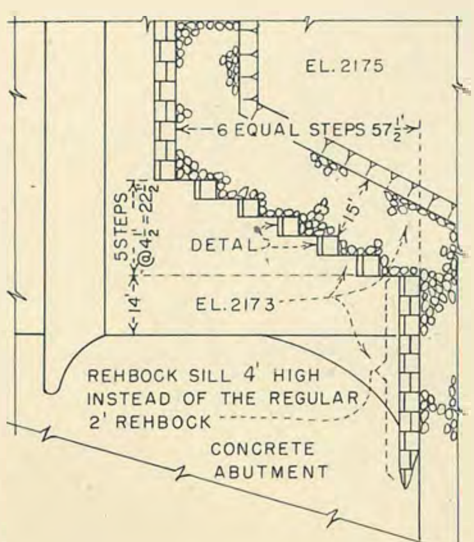
TEST 40

TEST 40

Same as test 35 except variable length apron was used (see sketch). The same procedure was followed as in test 35. Conditions were satisfactory for all discharges up to and including 150,000 second-feet with the minimum estimated tailwater. At 100,000 second-feet, with the tailwater two feet below the minimum estimated, some of the riprap was rolled away from below the sill at a point directly downstream from the intersection of the crest and the 2:1 slope. Erosion in the above mentioned area became severe at 150,000 second-feet with the tailwater elevation two feet below the minimum estimated. These flow conditions were maintained for 35 minutes. At the end of the run the scour was to elev. 2160 along the cut-off wall directly downstream from the intersection of the crest and the 2:1 slope. No erosion occured elsewhere.

TEST 41

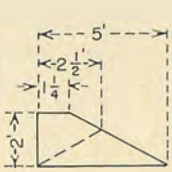
Same as test 40 except variable length apron altered and a 4-foot Rehbock sill placed at the downstream edge of the abutment only (see sketch). The same procedure was followed as in test 35. Satisfactory results were obtained for all discharges up to and including 150,000 second-feet with the minimum estimated tailwater. With the minimum estimated tailwater very slight erosion occurred immediately downstream from the large sill. This discharge with the tailwater two feet below minimum estimated eroded this area to elevation 2163. The sand and riprap immediately downstream from the rest of the apron settled to elevation 2167.



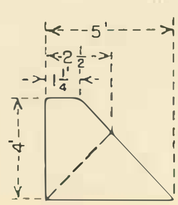
TEST 41

TEST 42

Same as test 40 except a 4-foot Rehbock sill was used on the portion of the apron farthest downstream. Also, 2-foot placed riprap five feet in depth, was extended 20 feet downstream from the 4-foot sill. Twelve runs of 20 minutes each were made. The sand bed was not disturbed between runs, thus giving very severe operating conditions during the latter part of the test. Runs were made at discharges of 25,000, 50,000, 100,000 and 150,000 second-feet with tailwater elevations according to three curves (fig. 25-2-3-PCD-8). Very satisfactory results were obtained for all discharges with tailwater elevation according to Debler's curve and the minimum estimated curve. Conditions with tailwater according to the arbitrary curve were more severe but the scour adjacent to the cutoff wall was not below elevation 2166. The sand and riprap immediately downstream from the dentals settled to elevation 2168.



2' REHBOCK SILL



4' REHBOCK SILL

UNITED STATES
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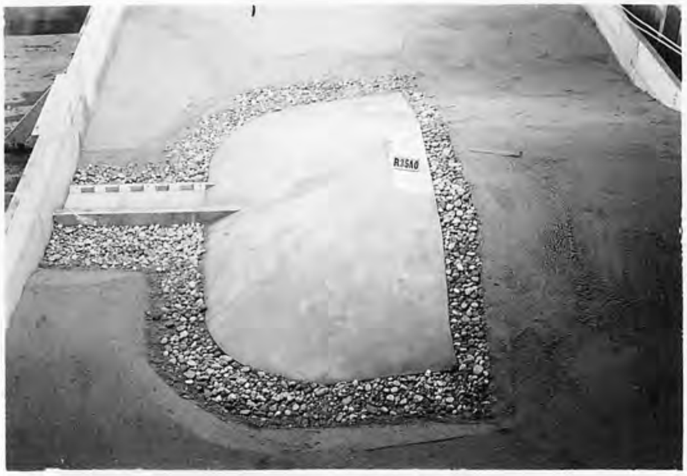
SALT RIVER PROJECT-ARIZONA

POWER CANAL DIVERSION DAM
HYDRAULIC MODEL STUDIES
MODEL SCALE 1:24
SUMMARY OF TESTS 39 TO 42 INCL.

DRAWN. J.M.B.	SUBMITTED.
TRACED. J.E.V.–H.M.	RECOMMENDED.
CHECKED.	APPROVED.

REV. 10-8-36 J.M.B.

25-2-3-PCD-9 FT. COLLINS, COLO. —— 9-23-36



A. RIVER BED BEFORE RUN.

B. RIVER BED AFTER RUN.

SHORT APRON - TAILWATER TWO FEET BELOW MINIMUM ESTIMATED.

C. SCOUR AT END OF APRON.

D. CONDITIONS UPSTREAM.

VARIABLE LENGTH APRON - MINIMUM ESTIMATED TAILWATER.

E. SCOUR AT END OF APRON.

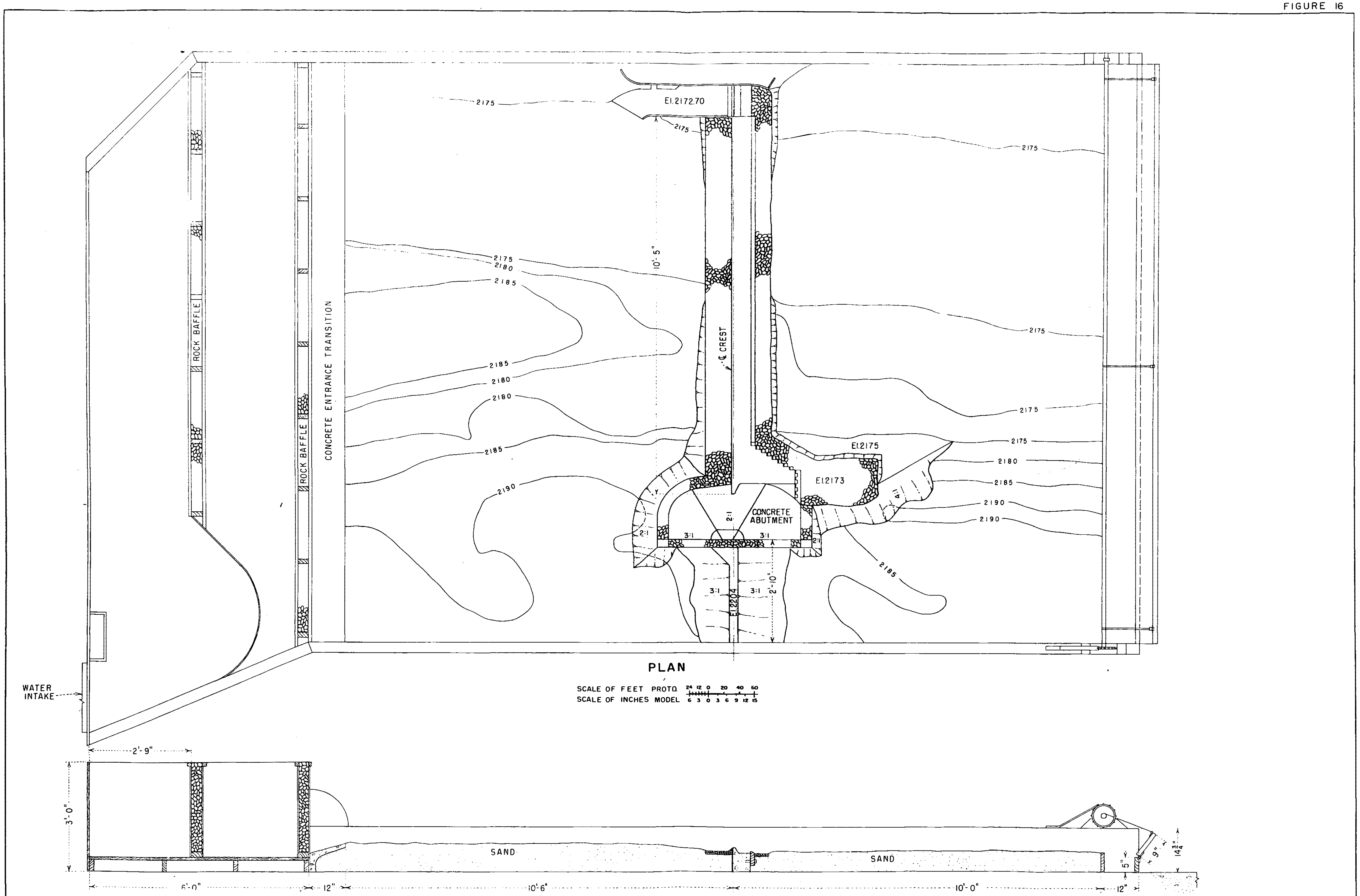
F. CONDITIONS UPSTREAM

VARIABLE LENGTH APRON - TAILWATER TWO FEET BELOW MINIMUM ESTIMATED.

CONDITIONS FOR RECOMMENDED ABUTMENT - WITH AND WITHOUT VARIABLE LENGTH APRON - DISCHARGE 150,000 SECOND-FEET.

VIEW OF PROTOTYPE FROM LEFT BANK SHOWING CONCENTRATION OF FLOW NEAR LEFT END OF SPILLWAY FOR A DISCHARGE OF 38,000 SECOND-FEET.

Figure 15



POWER CANAL DIVERSION DAM
1:48 MODEL

in the laboratory, the restrictions of space prevalent at the time of the construction of the 1:24 model of a section of the structure did not then exist.

The model was constructed of concrete placed between sheet metal templates which served as guides for screeds in shaping the surface of the model dam. The dentated sill was made of redwood and was fastened to the spillway apron by bolts embedded in the concrete. The model was placed directly on the floor of one of the laboratory tanks, and the area representing possibly 2/10 of a mile of the full width of the riverbed was enclosed by low wooden walls made watertight with asphalt emulsion. Brass rods 1/4 inch in diameter were fastened to strips of galvanized metal placed on the tank floor. The tops of these rods served as guides for placing the sand and gravel which represented the topography.

A maximum model discharge of 9.39 second-feet, corresponding to a prototype discharge of 150,000 second-feet was measured by a 4-foot Cipolletti weir. Rock baffles quieted the flow before it entered the streambed above the dam. An adjustable weir at the downstream end of the model was used to regulate the tailwater.

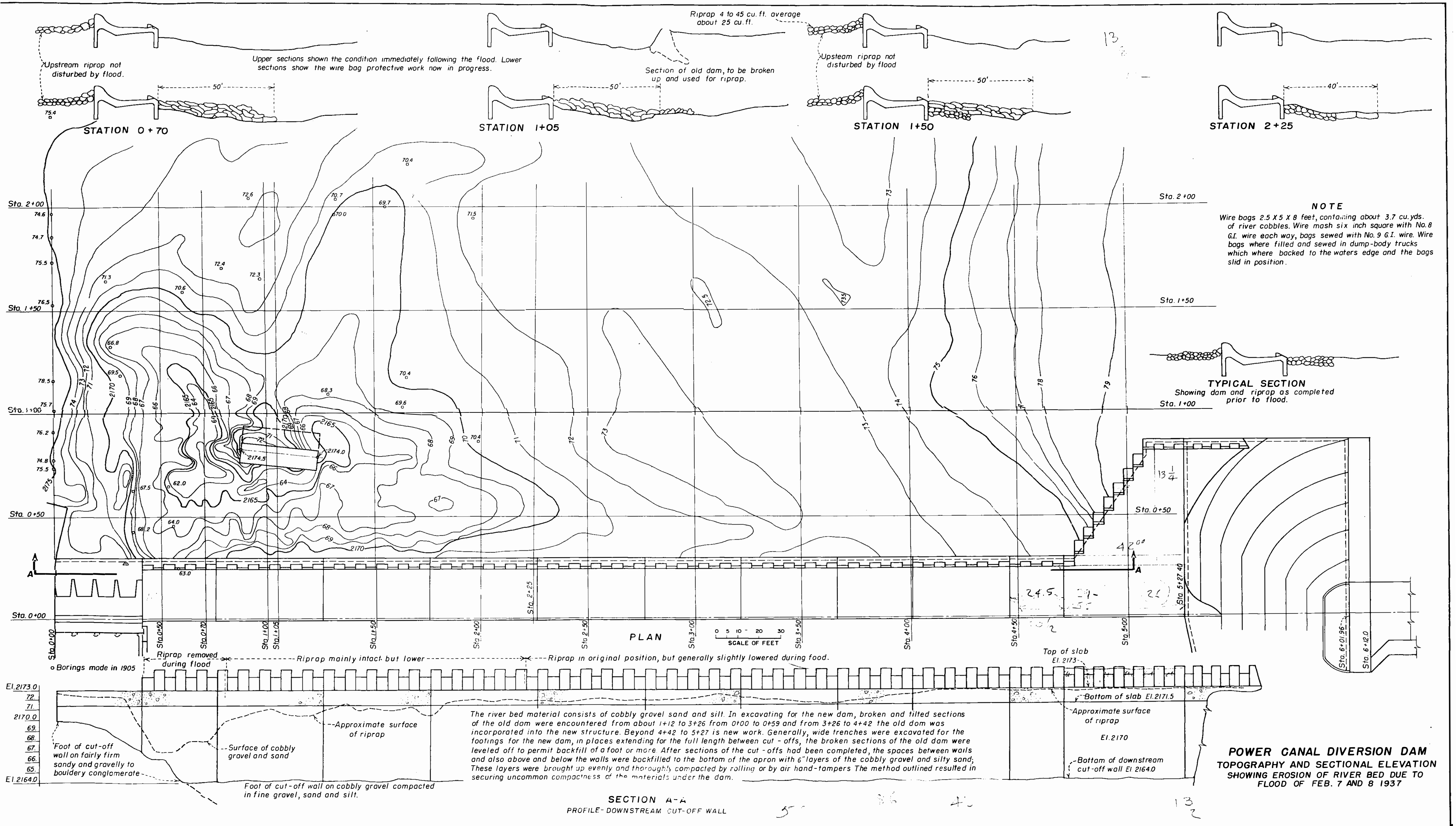
The field data shown on Figures 17 and 18 and on a number of photographs received after the flood of February 7 and 8, 1937, were used in adjusting the performance of the 1:48 model so that it would represent the prototype conditions as far as feasible.

Studies on 1:48 Model of Power Canal Diversion Dam (1937)

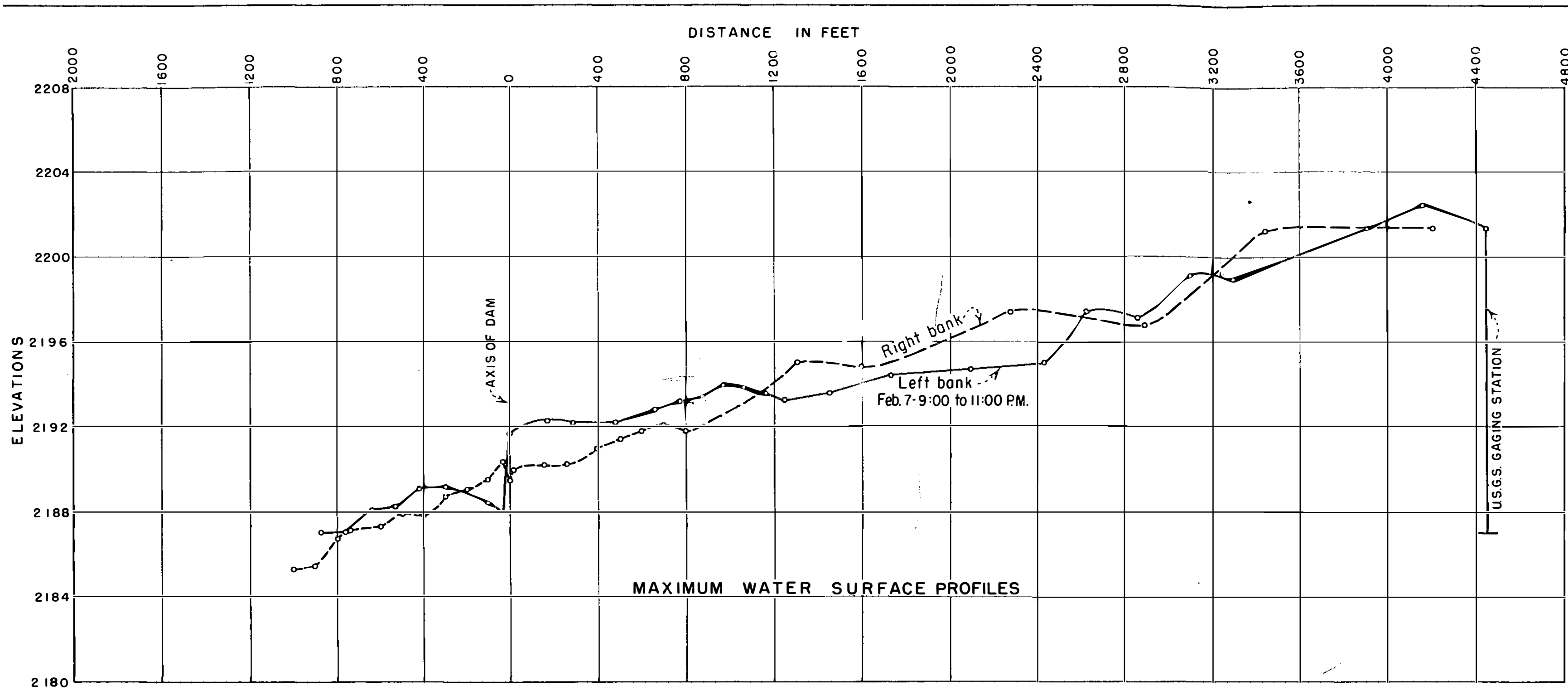
In the preliminary tests the flow upstream from the dam was concentrated to approximate the prototype conditions shown on the photographs taken during the flood (Figure 15). The concentration of flow near the intake structure could not be maintained in the model with the riverbed built of fine sand. The sand upstream from the model spillway crest was replaced by a coarse gravel. Tests indicated inadequate scour protection for the left end of the dam with discharges above 60,000 second-feet and the tailwater 2 feet below the minimum estimated. Because of the close agreement up to 40,000 second-feet between the tailwater in the field during the first part of the 1937 flood and the arbitrary curve used for testing the 1:24 model, that curve was used for these investigations.

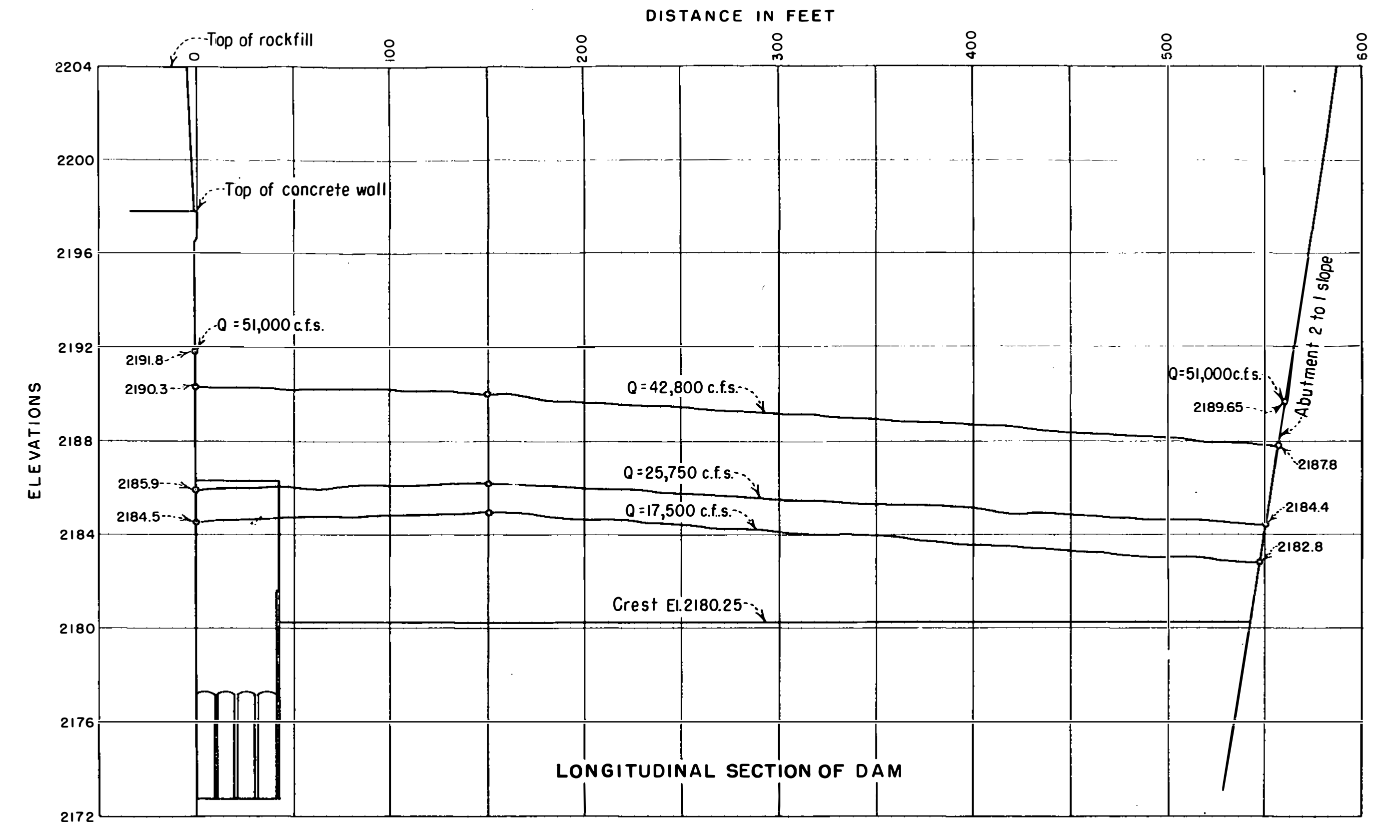
Scour contours for the February 1937 flood, with the section of the old dam downstream from the left side and using 51,000 second-feet for the peak flow, checked the prototype remarkably well (Figure 17). The results would have checked even better had a peak flood of 67,300 second-feet been used on the model, as it was later determined that the peak flow was of that order. After a discharge curve prepared from current meter measurements of the February flood failed to agree with a previous United States Geological Survey rating curve, the peak flow on the model

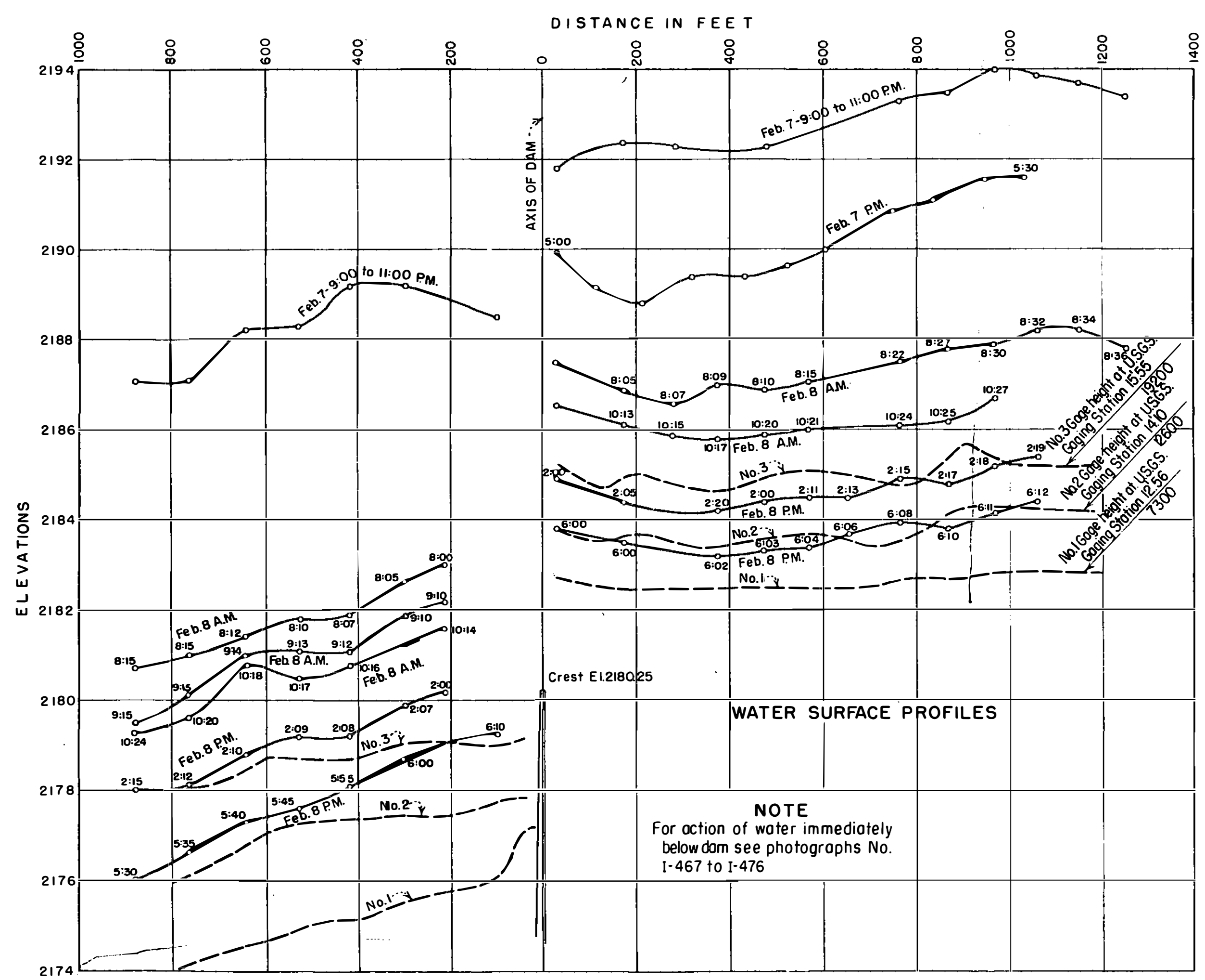
FIGURE 17



POWER CANAL DIVERSION DAM
TOPOGRAPHY AND SECTIONAL ELEVATION
SHOWING EROSION OF RIVER BED DUE TO FLOOD OF FEB. 7 AND 8 1937







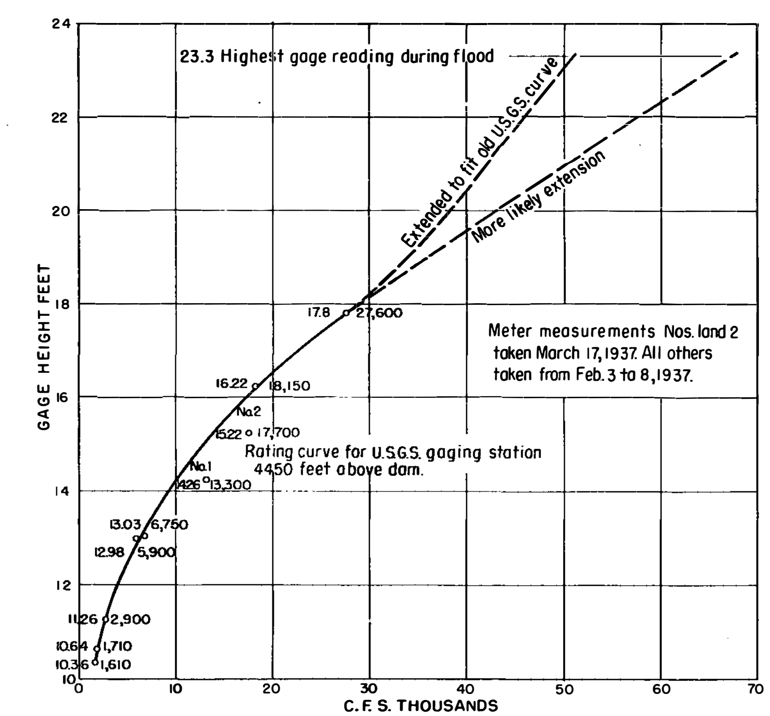
U.S.G.S. GAGE READINGS					
TIME	READING	TIME	READING	TIME	READING
FEB.6-10:00P.M.	5.01				
FEB.6-11:00P.M.	5.02				
FEB.6-12:00P.M.	5.09				
FEB.7- 1:00A.M.	5.20	FEB.8- 1:00A.M.	22.20	FEB.9-1:00 A.M.	13.90
FEB.7- 2:00A.M.	5.34	FEB.8-2:00A.M.	21.90	FEB.9-2:00 A.M.	13.80
FEB.7- 3:00 A.M.	5.56	FEB.8-3:00A.M.	21.60	FEB. 9-3:00 A.M.	13.70
FEB.7- 4:00A.M.	6.10	FEB.8- 4:00A.M.	21.40	FEB.9- 4:00 A.M.	13.55
FEB.7- 5:00A.M.	6.65	FEB.8-5:00A.M.	21:00	FEB.9-5:00 A.M.	13.45
FEB.7- 6:00A.M.	7.25	FEB.8-6:00A.M.	20.50	FEB.9-6:00A.M.	13.30
FEB.7- 7:00A.M.	7.95	FEB.8-7:00A.M.	20.00	FEB.9-7:00 A.M.	13.25
FEB.7- 8:00A.M.	9.10	FEB.8-8:00A.M.	19.30	FEB.9-8:00 A.M.	13.20
FEB.7- 9:00A.M.	10.60	FEB.8-9:00A.M.	18.70	FEB.9-9:00A.M.	13.13
FEB.7 10:00A.M.	12.90	FEB.8-10:00A.M.	18.10	FEB.9-10:00A.M.	13.80
FEB.7-11:00A.M.	15.00	FEB.8-11:00A.M.	17.40	FEB.9-11:00A.M.	13.30
FEB.7-12:00 Noon	16.20	FEB.8-12:00Noon	17.20	FEB.9-12:00Noon	13.00
FEB.7- 1:00P.M.	16.80	FEB.8- 1:00P.M.	16.80	FEB.9-1:00 P.M.	12.95
FEB.7- 2:00P.M.	17.20	FEB.8- 2:00P.M.	16.40	FEB.9-2:00 P.M.	12.85
FEB.7- 3:00P.M.	19.00	FEB.8-3:00P.M.	16.10	FEB.9-3:00 P.M.	12.78
FEB.7- 4:00P.M.	20.00	FEB.8-4:00P.M.	15.75	FEB.9-4:00 P.M.	12.68
FEB.7-5:00P.M.	20.50	FEB.8-5:00P.M.	15.50	FEB.9-5:00P.M.	12.58
FEB.7- 6:00P.M.	21.20	FEB.8-6:00P.M.	15.20	FEB.9-6:00P.M.	12.48
FEB.7-7:00P.M.	22.00	FEB.8-7:00P.M.	15.00	FEB.9-7:00 P.M.	12.35
FEB.7-8:00P.M.	22.40	FEB.8-8:00P.M.	14.85	FEB.9-8:00 P.M.	12.25
FEB.7- 9:00P.M.	23.30	FEB.8-9:00P.M.	14.55	FEB.9-9:00 P.M.	12.13
FEB.7-10:00P.M.	22.90	FEB.8-10:00P.M.	14.40	FEB.9-10:00P.M.	12.02
FEB.7-11:00P.M.	23.30	FEB.8-11:00P.M.	14.25	FEB.9-11:00P.M.	11.93
FEB.7-12:00P.M.	22.20	FEB.8-12:00 P.M.	14.10	FEB.9-12:00P.M.	11.85

NOTES

Depth on crest determined from rod readings below wall of left abutment 30 feet upstream from crest. 0=El.2180.25.

Q determined from U.S.G.S. gaging station 4450 feet upstream from dam, readings being taken at the same time as the readings for depth on crest 0=El.2178.04.

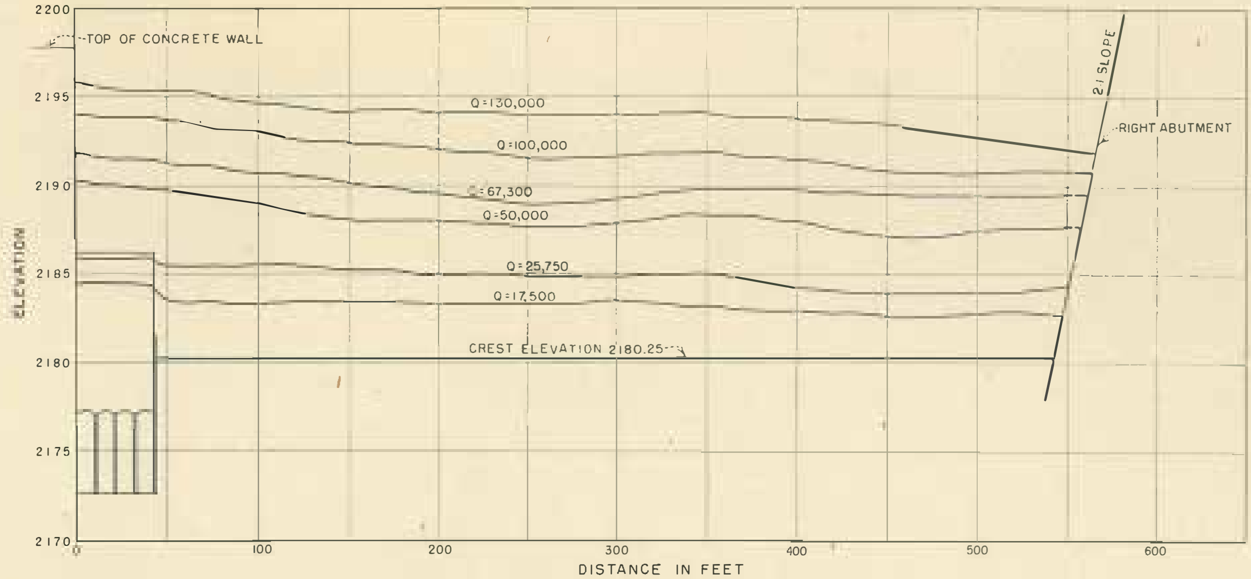
U.S.G.S. GAGE	DEPTH ON CREST	Q c.f.s.	TIME
14.7	3.05'	11200	FEB.8-8:30P.M.
15.2	3.55'	13500	FEB.8-6:00P.M.
16.1	4.25'	17500	FEB.8-3:00P.M.
16.4	4.65'	19350	FEB.8-2:00 P.M.
16.8	4.95'	21750	FEB.8-1:00 P.M.
17.2	5.30'	24700	FEB.8-12:00Noon
17.4	5.95'	26000	FEB.8-11:00 A.M.
18.1	6.25'	30400	FEB.8-10:00A.M.
18.7	7.05'	33200	FEB.8-9:00A.M.
19.3	7.25'	35500	FEB.8-8:00 A.M.
20.0	9.05'	38200	FEB.7-4:00P.M.
20.5	9.45'	40000	FEB.7-5:00P.M.
21.2	10.05'	42800	FEB.7-6:00P.M.
23.3	11.55'	51000	FEB.7-9:00P.M.



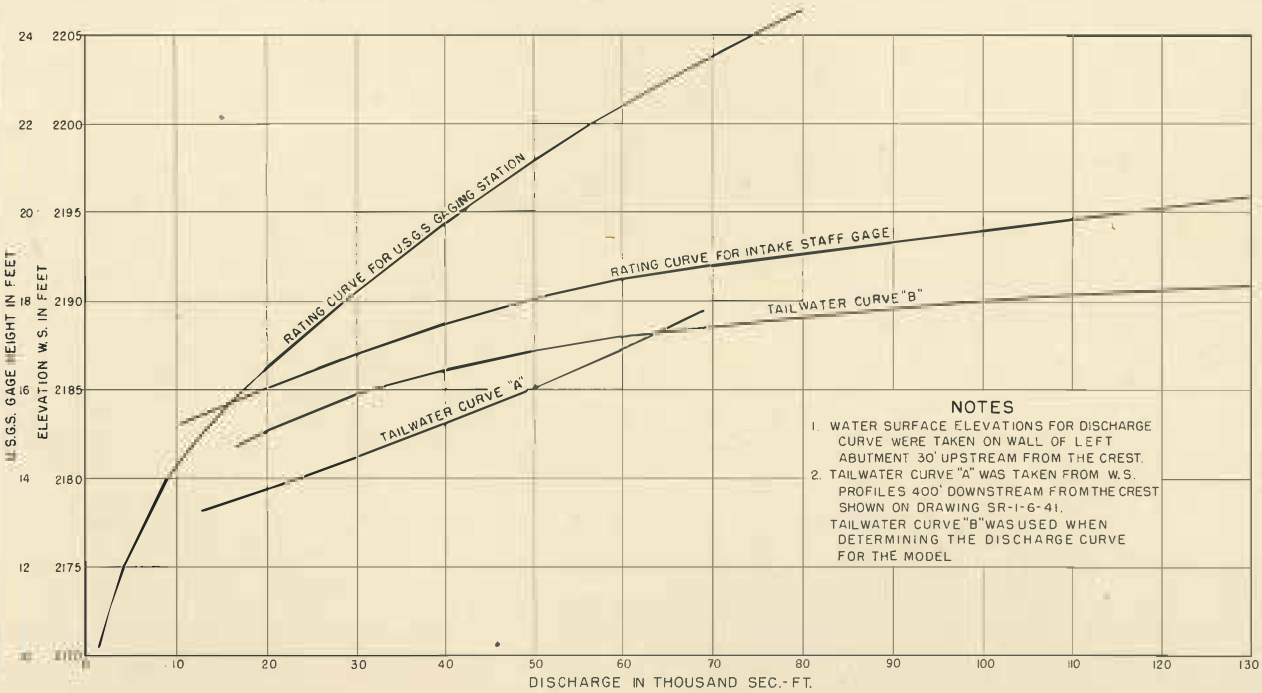
POWER CANAL DIVERSION DAM
FLOOD LEVELS FEB. 7 AND 8, 1937

was checked to determine if a more likely extension of the rating curve (Figure 18) were correct. Two staff gages were installed in the model, one 30 feet (prototype) upstream from the crest axis on the left intake wall, and the other on the 2:1 slope of the right abutment at the axis of the dam. Prototype flows of 17,500, 25,750, 42,800, and 51,000 second-feet for which field data have been obtained were used for the test. The staff gages were observed for each flow, and the depth on the crest was obtained over the entire length of the dam. For the first two flows the surface on the wall of the intake section checked the prototype elevations, while those at the right abutment were too low. These lower elevations were attributed to a relatively smooth and differently arranged streambed below the model dam. The tailwater was raised until the proper elevation was obtained at the abutment. It was necessary to raise the tailwater TW to elevation 2182 for a discharge of 17,500 and elevation 2184 for 25,750 second-feet. The staff reading at the intake was not affected by the increase in tailwater depth, so this method was used for subsequent runs for which prototype data were available. When an attempt was made to check the water surfaces for 42,800 and 51,000 second-feet, an unreasonably deep tailwater was required to give the proper reading on the intake staff gage, and the water surface at the right abutment was considerably above the recorded prototype elevation when the intake gage read correctly. This condition was corrected by maintaining the desired water surface elevation at the right abutment with the tailwater regulating device, while increasing the flow until the desired reading was obtained on the intake staff gage. A discharge of 50,000 second-feet was required to obtain water surface elevations comparable to the prototype for the estimated 42,800 second-feet, while 67,300 second-feet were necessary to give the proper elevation for the estimated 51,000 second-feet. From these studies it was concluded that the maximum discharge for the flood of February 7 and 8 was nearer the 68,000 second-feet obtained by a smooth extension of the rating curve (Figure 18) than the 51,000 second-feet obtained by extension to fit the old United States Geological Survey rating curve. Discharges of 100,000 and 130,000 second-feet were also studied. The absence of prototype data for these discharges made it impossible to extend the rating curve accurately. However, the prototype curve should not vary much from the approximate extension obtained on the model. The water surfaces across the dam for the various flows were plotted, and a new rating curve for the staff gage on the intake wall obtained. A new rating curve for the United States Geological Survey station, including discharges up to the peak flood, was also prepared from these data (Figure 19).

As soon as the prototype flood data had been checked, studies to obtain a design capable of handling 130,000 second-feet with a tailwater 2 feet below the minimum estimated, were begun. A solution for this discharge was sought first because it was the largest discharge that the laboratory pump could conveniently supply for a 2-hour run.



LONGITUDINAL SECTION OF DAM
SHOWING W.S. AT CREST



POWER CANAL DIVERSION DAM
HYDRAULIC MODEL STUDIES
MODEL SCALE 1:48
SUMMARY OF FLOW DATA

The height of the dentated sill was increased to 3 feet (Test 14, Figure 20), as the first step in preventing scour below the apron, but no appreciable improvement was noted.

A solid 20-foot strip of pavement without a sill placed downstream from the apron, sloping from elevation 2173 to 2170 (Test 13) improved conditions but did not eliminate erosion near the left end of the dam. A solid 25-foot extension sloping from elevation 2173 to 2169 with a 3-foot dentated sill at the downstream end (Test 16, Figure 20) proved very satisfactory except at the extreme left end below the intake section.

An articulated extension of 20 feet sloping from elevation 2173 to 2169 with a 3-foot Rehbock sill at the end (Test 17, Figure 21) also gave ample downstream protection to the central portion of the spillway but failed to protect that portion of the riverbed downstream from the intake.

Extensions and additions of various types were used to prevent scour downstream from the left end of the dam and intake (Figure 17). None proved entirely satisfactory when the model was operated for an appreciable length of time until the left intake wall below the dam was straightened and used in conjunction with the articulated apron. For all arrangements except this the eddy which formed immediately downstream from the intake next to the bank whirled the sand and riprap from behind the sill and exposed the cutoff wall to a depth of at least 6 feet. Apparently the straightened wall assisted in improving the conditions for the relation of head and tailwater used.

About the time the articulated apron was being seriously considered as a solution, additional field data were obtained which indicated that a lowering of the tailwater had taken place after the peak flood in February 1937. To be on the safe side a tailwater curve 4 feet below the minimum estimated (Figure 3) was used in further studies. When this tailwater relationship was used on the previous arrangement of downstream protection, severe scour resulted reaching elevation 2159 near the downstream edge of the right abutment and elevation 2155 below the intake section in 2-1/2 hours (model). The scour downstream from the intake section was not considered serious as a rock channel was indicated in that area.

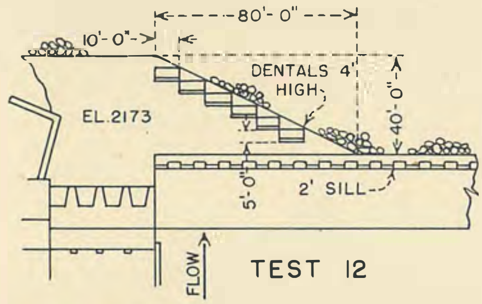
This rock channel was installed on the model (Test 20, Figure 21), and different arrangements made around the right abutment using variations of the articulated apron to prevent scour downstream. None of the designs worked satisfactorily because the short apron sections became undermined and dropped with the result that the material washed away from the cutoff wall of the existing apron and the scour eventually reached the same depth in each case.

POWER CANAL DIVERSION DAM

SUMMARY OF TESTS 12 TO 16 INCL.

NOTE FOR ALL TESTS

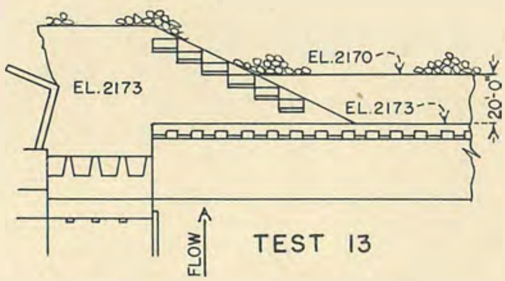
The model discharge representing 130,000 second-feet was maintained for $2\frac{1}{2}$ hours with the tailwater 2-feet below the minimum estimted. Flow was concentrated on left end of spillway to represent conditions anticipated on the prototype. Grovel was used to represent the river bed upstream from the dam and practically no sand passed over the spillway during the tests. The material used for riprap passed through a $\frac{1}{2}$-inch mesh and was retained on a $\frac{1}{4}$-inch mesh screen.



TEST 12

TEST 12

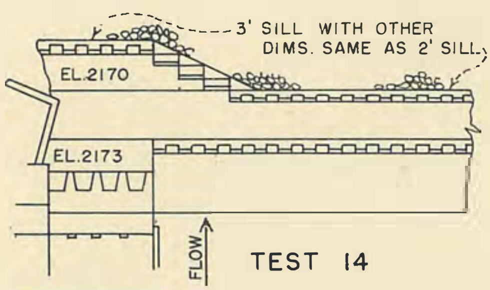
The model arrangement was the same as the finished prototype (Jan. 1937) except for a variable length apron with 4-foot dentals added to the left end of the structure. In this test the cut-off wall of the apron, except in the region of the variable length apron was exposed to a depth varying from 4 to 6 feet, The scour reached elevation 2161 approximately 40 feet down stream from the spillway apron.



TEST 13

TEST 13

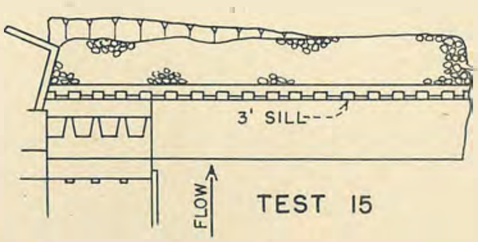
Same as test 12 except for the addition of a 20 foot sloping apron without a sill on its dowstream end, which was at elevation 2170. No improvement in the scour was noted over test 12. The scour reached elevation 2167 just downstream from the end of the apron.



TEST 14

TEST 14

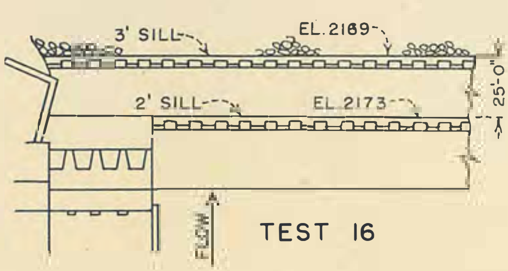
Same as test 13, except for a 3-foot sill placed downstream from the intake section and along the end of the sloping apron; also the 20 foot sloping apron was continued across to the left abutment and the down steam end of the variable length apron below the intake section was made level at elevation 2170. No serious erosion was observed except in an area imediately downstream and to the right of the intake section where the scour reached a depth of 7 feet below the apron.



TEST 15

TEST 15

Same as the finished prototype (Jan. 1937) except a 3-foot sill, extended to the left intake wall, was used in place of the 2-feet sill. The scour was serious downstream from the left intake wall of the structure exposing the cut-off wall to a depth of 7 feet in many places.



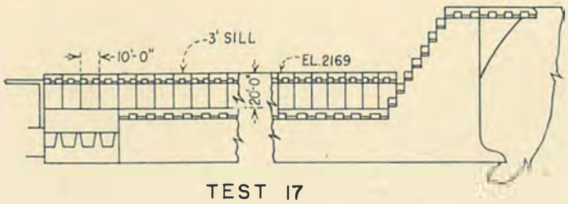
TEST 16

TEST 16

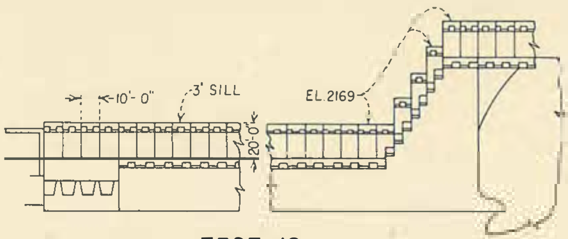
Same as the finished prototype (Jon. 1937) except a 25 foot sloping apron with its downstream end at elevation 2169 was added. A 3 foot sill was placed on the end of this apron. No serious scour occurred except just downstream from the intake section. The results were not appreciably different from test 14.

POWER CANAL DIVERSION DAM

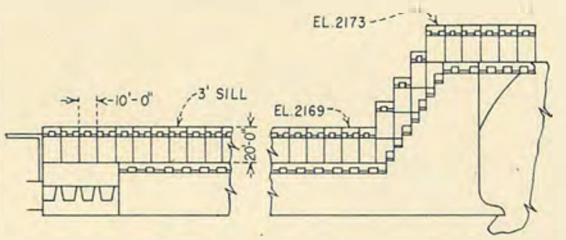
SUMMARY OF TESTS 17 TO 26 INCL.



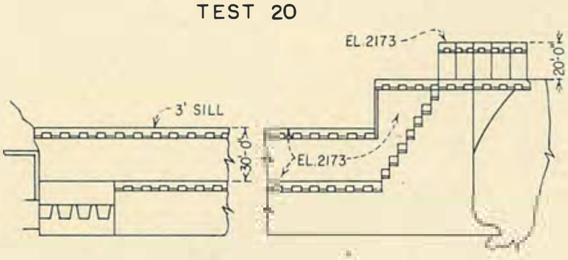
TEST 17



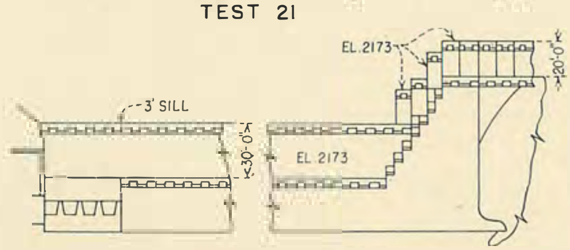
TEST 19



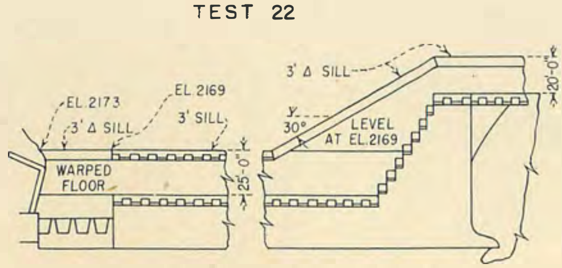
TEST 20



TEST 21



TEST 22



TEST 23 & 26

TEST 17

Same as finished prototype (Jan. 1937) except a 20-foot articulated apron sloping from elevation 2173 to elevation 2169 with 3-foot Rehbock sill at its downstream end, was placed along the downstream edge of the spillway. Apron sections were ten feet wide. The left intake wall was straightened. There was no scour just downstream from the sill. However, when an additional run of 2 hours was made with the tailwater 4-feet below the minimum estimated, serious scour occurred downstream from both abutments.

TEST 18

Same as test 17 except three short apron sections were extended 2 feet, 3 feet and 4 feet downstream from the 3-foot Rehbock sill to prevent scour at low discharges with no tailwater. A discharge of 20,000 second-feet was run with a tailwater of 2173. No serious erosion occurred along the apron either with or without the apron extenions.

FOR ALL SUBSEQUENT TESTS THE TAILWATER USED WAS 4-FEET BELOW THE MINIMUM ESTIMATED INSTEAD OF 2-FEET BELOW.

TEST 19

Same as finished prototype (Jan. 1937) except that an articulated apron extending 20 feet downstream and sloping from elevation 2173 to 2169, with a 3-foot Rehbock sill placed at the downstream end, was placed across the entire length of the spillway. The left intake wall was straigntened and rock topography was placed in the left side. Serious scour occurred downstream from the intake along the sharp edge left on the rock topography. Scour occurred to elevation 2161 downstream from the right abutment.

TEST 20

Same as test 19 except that the rock topography was revised to more closely approximate the prototype, also, a longer articulated apron was used downstream from the variable length portion near the right abutment. Improved scour conditions were noted downstream from the intake section, while those at the right abutment were worse.

TEST 21

Same as finished prototype (Jan. 1937) except an apron level at elevation 2173, with a 3-foot Rehbock sill at its downstream end, was extended 30 feet downstream. 4-foot sill at right abutment was extened 38 feet to the left. 5 sections of articulated apron, 20 feet long were placed downstream from the right abutment and the left intake wall was straight. No serious erosion occurred except downstream from the right abutment where the scour reached elevation 2155.

TEST 22

Same as test 21 except 3 sections of articulated apron were used instead of extending the 4-foot sill 38 feet to the left. Serious scour, reaching on elevation of 2151 still occurred downstream from the right abutment.

TEST 23

Same as finished prototype (Jan. 1937) except an apron sloping from elevation 2173 to 2169 was extended 25 feet downstream with a 3-foot Rehbock sill on the downstream end of its central portion. Additional extension near the right abutment level at elevation 2169. 3-foot trapezoidal sill downstream from both abutments. No serious erosion resulted during the first run so the model was operated on additinol 2 hours at a discharge of 150,000 second-feet with the tailwater at 4 feet below the minimum estimated. The rear of the sill was still covered with sand except on the rock topography in the region of the intake section.

TEST 24

Same set-up as test 23 except the 3-foot triangular sill was used across the entire width of the apron extension. After a discharge of 150,000 second-feet for $2\frac{1}{2}$ hours the back of the sill was still completely covered. The slope of the sand downstream from the sill was much steeper than in test 23

TEST 25

Same set-up as test 23 except a 20-foot sloping apron to elevation 2169 used instead of the 25-foot one. After a discharge of 150,000 second-feet for 2 hours, the erosion was greater than at the end of test 23.

TEST 26

Same set-up as test 23. A cumulative test was made using discharges of 30,000, 60,000, 90,000, 120,000 and 150,000 second-feet with the tailwater 4-feet below the minimum estimated. The results were satisfactory assuming the rock topography downstream from the intake section was correct.

The ends of three sections of the articulated apron were lowered 1, 2, and 3 feet to ascertain what would happen if, in the course of operations, some of the sections became lowered. No change was detected for the 1-foot drop when the model was operated for 2-1/2 hours at 130,000 second-feet. Only slight scour occurred with the 2-foot lowering, while severe scour with undermining back of the cutoff wall of the existing apron was obtained for the 3-foot drop. The severe action in the latter case was ascribed to currents created by the flow of water through the opening upstream from the sill between the lowered section and the adjacent section.

A test to determine the consequences of the flash flood with practically no tailwater downstream from the apron was made. Extensions of 2, 3, and 4 feet were placed downstream from short sections of the articulated apron. The purpose of these extensions was to prevent erosion by the jets issuing from the back of the downstream sill. In all cases, the tailwater increased so rapidly that a water cushion formed on the sloping apron upstream from the second sill and no serious erosion resulted. Moreover, the extensions never functioned as scour protection. Of the quantities tested, 20,000 second-feet gave the worst conditions. This test was not repeated on the recommended design, as it was indicated that the 5-foot additional apron length would serve to improve conditions by lengthening the cushioning pool between the sills. Unless there was severe retrogression of the riverbed, the sill on the sloping apron would always be submerged when water was flowing.

The articulated apron design was abandoned when a solid sloping apron gave equal results along the central portion of the spillway and when it seemed impossible to adequately protect the abutments by using the narrow apron section. A level apron extending 30 feet downstream was tested to save the excavation required in constructing the sloping design. This was not satisfactory as the maximum depth of scour moved upstream nearer the apron causing a much steeper slope of the riverbed immediately downstream from the sill. Articulated sections and solid extensions below the right abutment (Figure 21) were of no benefit, and the 25-foot sloping apron with the 3-foot Rehbock sill was again installed. A level extension downstream and to the left of the right abutment with a trapezoidal sill along its edges proved adequate when the model was operated at 130,000 second-feet for 2-1/2 hours. The maximum depth of scour which reached elevation 2159 occurred some distance downstream. Slight improvement was obtained below the intake when a short length of the 3-foot trapezoidal sill was placed at that end of the apron. Conditions in this region were still severe but were not considered serious because borings made during the original construction of the dam indicated solid rock in this region.

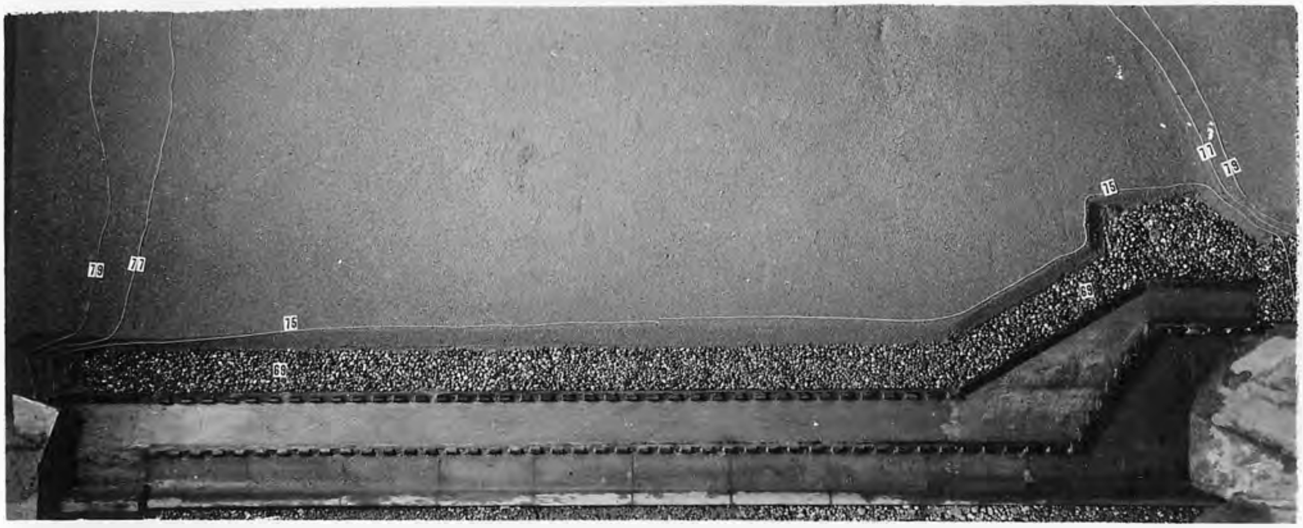
After this design had proven satisfactory for 130,000 second-feet, the laboratory pump was reworked for higher speed, and the model was tested for 150,000 second-feet. Some increase in erosion was obtained at this flow but no critical change occurred. The design was considered satisfactory. The model was then operated at various discharges to investigate the conditions throughout the flood range. Very desirable results were obtained adjacent to all parts of the structure except the intake section. The riprap downstream from the apron in this region washed downstream at 60,000 second-feet, and the conditions became more severe as the flow was increased to 150,000 second-feet when a large area of the model floor, elevation 2151, was exposed (Figures 22 and 23).

This condition seemed objectionable unless the rock surface below the intake section was solid and near the elevation shown by borings made in 1905. Because of the uncertain composition of the foundation and because of the unknown depth and extent of the rock surface, tests for improvement were made assuming a riverbed (composed) entirely of gravel and boulders. The first attempt involved extensions similar to those used below the right abutment together with various sill arrangements. None of this type proved effective so other methods were tried.

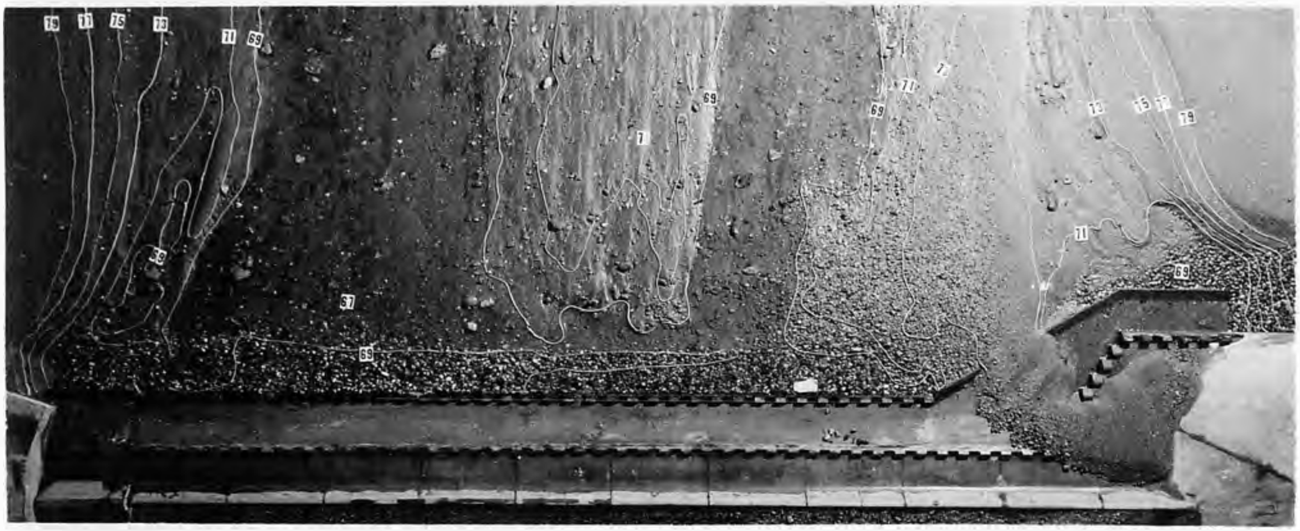
The top of the intake section was raised above the water surface for 150,000 second-feet. This only served to accentuate the larger eddy which formed below the intake section and caused the erosion to occur more rapidly. A wall extended alongside the intake to the end of the apron made very little difference. These tests indicated that the scour could be minimized only by eliminating the violent eddy and these walls were removed. The left downstream wall of the intake was replaced by a warped wall extending 27.75 feet downstream from the end of the apron. The conditions were somewhat improved but not sufficient to warrant the cost of the change in the wall.

Studies of the flow downstream from the section indicated the formation of two distinct prisms of water, one below the spillway where the flow moved swiftly downstream at a low elevation, the other below the intake where a higher tailwater formed due to the decreased quantity of water per foot passing over the sluice section of the intake. This latter prism, or body of water, having a surface higher than the first crowded to the right and directed the edge of the fast-moving jet along the riverbed causing deep scour.

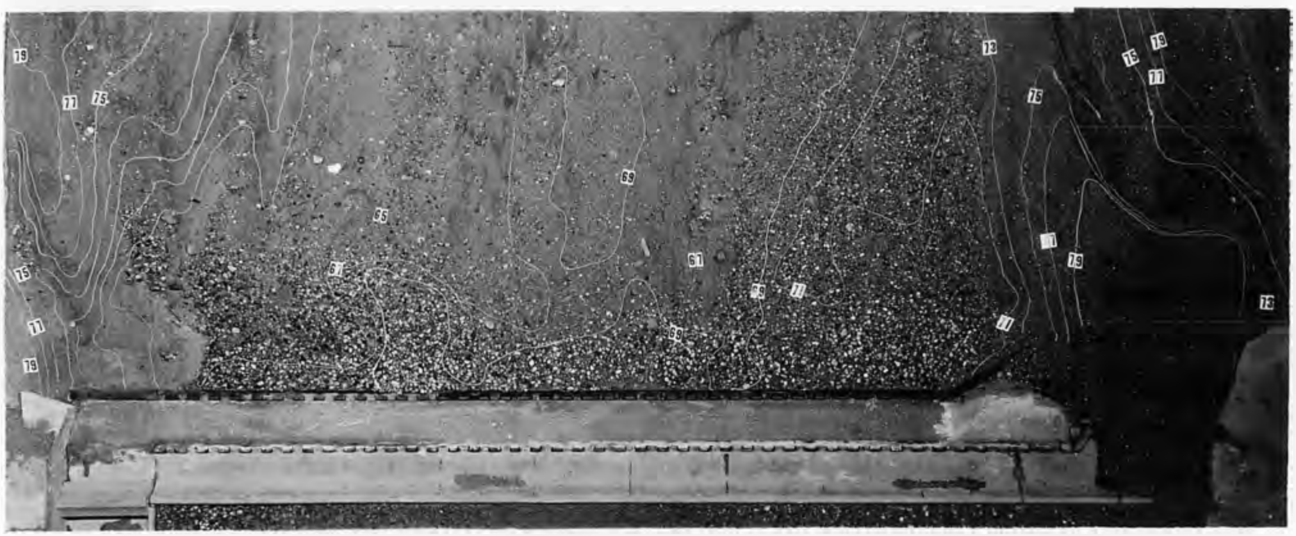
Apparently a separation of these two prisms until they reached approximately the same tailwater elevation would let each pass unmolested downstream and prevent crowding of the high-velocity jet, thus minimizing erosion from this source. An intermediate training wall seemed the likely solution, and one of excessive length and 18 inches prototype thickness was installed (Test 36) directly downstream from the right wall of the intake structure (Figure 24). This wall separated the two bodies of water, as anticipated, practically eliminating the eddy next to the bank and producing about the same depth and slope of erosion on the right



A. RIVER BED BEFORE RUN.



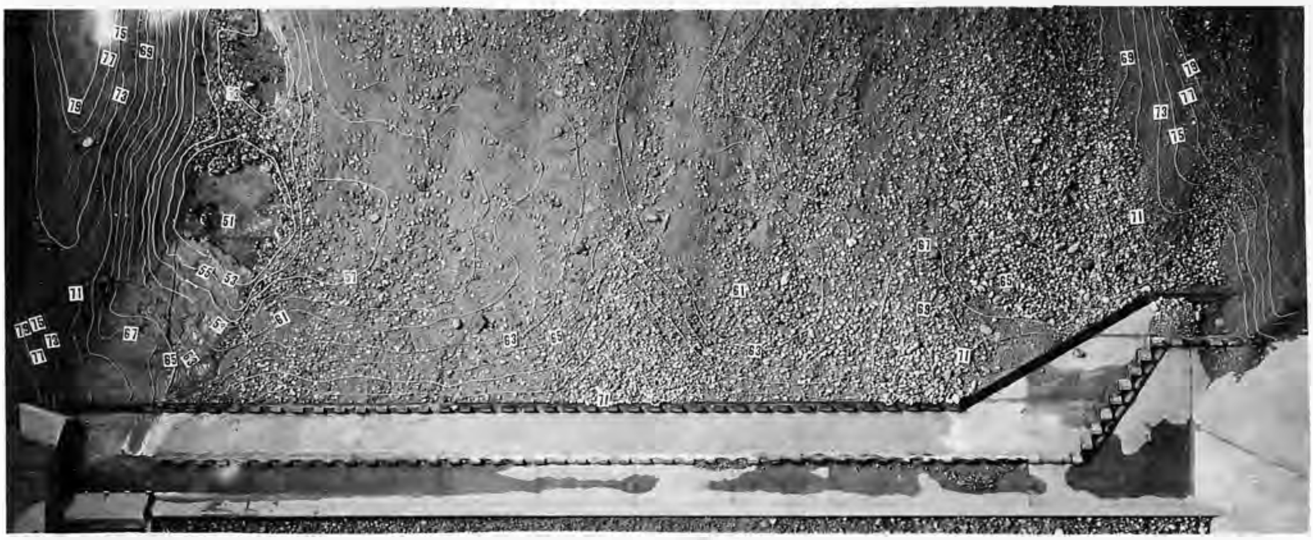
B. RIVER BED AFTER CONSTANT CONDITIONS WERE OBTAINED WITH A FLOW OF 30,000 SECOND-FEET.



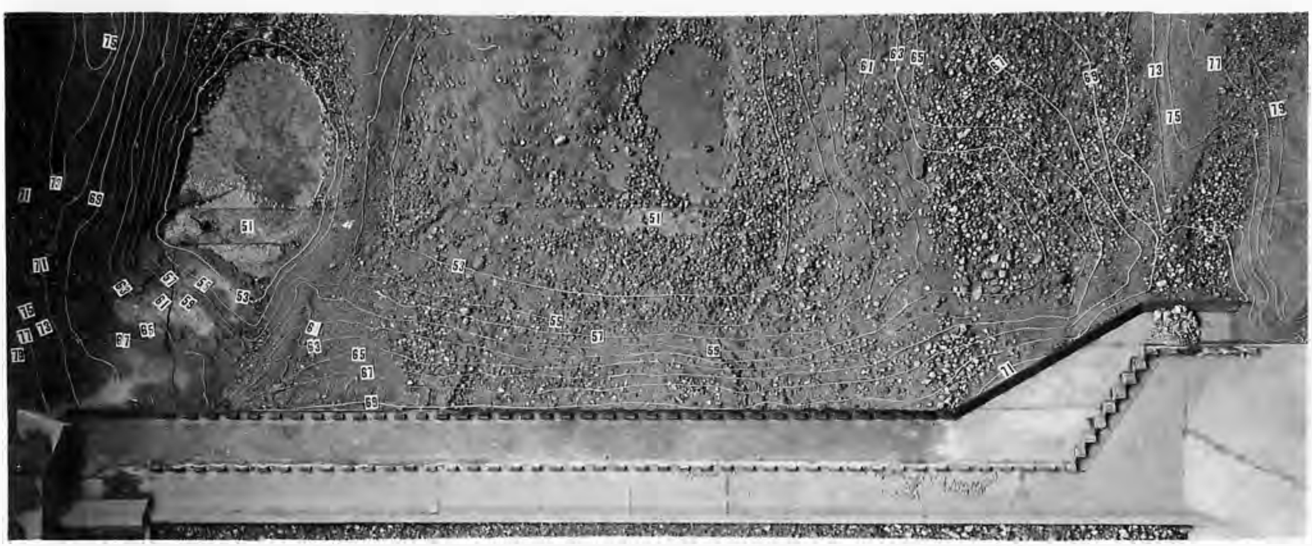
C. RIVER BED AFTER DISCHARGE OF 60,000 SECOND-FEET FOR AN ADDITIONAL PERIOD OF 50 MINUTES.

ACCUMULATIVE SCOUR FOR VARIOUS DISCHARGES WITH TAILWATER FOUR FEET BELOW MINIMUM ESTIMATED - SLOPING APRON EXTENSION OF 25 FEET. RECOMMENDED DESIGN EXCEPT DOWNSTREAM FROM INTAKE SECTION.

A. RIVER BED AFTER A DISCHARGE OF 90,000 SECOND-FEET FOR AN ADDITIONAL PERIOD OF ONE HOUR.

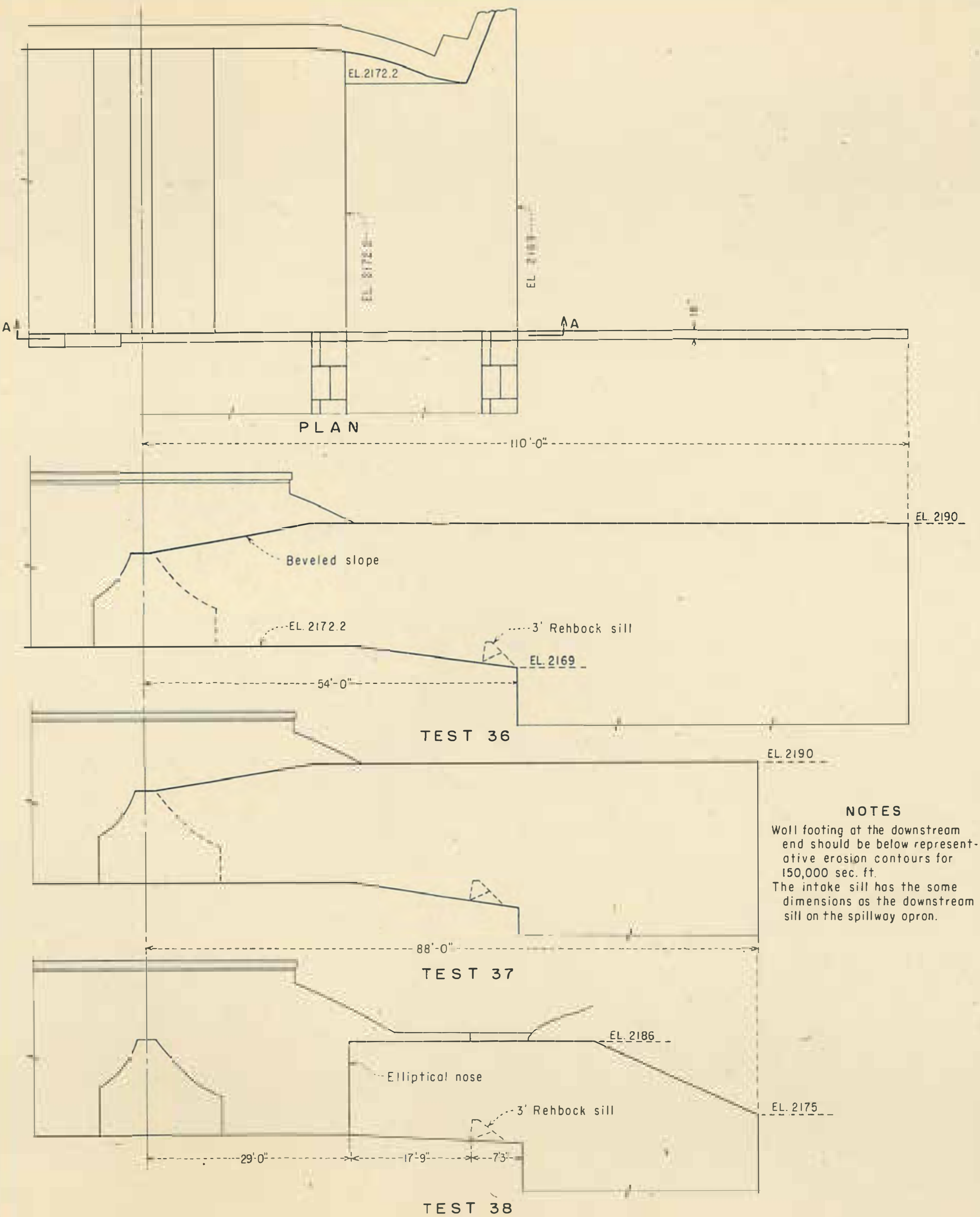


B. RIVER BED AFTER A DISCHARGE OF 120,000 SECOND-FEET FOR AN ADDITIONAL PERIOD OF ONE HOUR AND 25 MINUTES.



C. RIVER BED AFTER A DISCHARGE OF 150,000 SECOND-FEET FOR AN ADDITIONAL PERIOD OF TWO HOURS AND 40 MINUTES.

ACCUMULATIVE SCOUR FOR VARIOUS DISCHARGES WITH TAILWATER FOUR FEET BELOW MINIMUM ESTIMATED - SLOPING APRON EXTENSION OF 25 FEET. RECOMMENDED DESIGN EXCEPT DOWNSTREAM FROM INTAKE SECTION.



NOTES

Wall footing at the downstream end should be below representative erosion contours for 150,000 sec. ft.

The intake sill has the same dimensions as the downstream sill on the spillway apron.

POWER CANAL DIVERSION DAM
1:48 MODEL
INTERMEDIATE TRAINING WALL

side of the wall as had been previously obtained below the central portion of the spillway. Deeper scour occurred on the left side of the wall which was attributed to the current directed against the wall by the warped apron floor of the intake structure. The existing prototype design downstream from the left intake wall was reinstalled and the end of the apron floor was lowered to elevation 2172.2 adjacent to the wall to prevent undercutting of the footing during construction (Figure 24). A section of the 3-foot dentated sill was placed at the end of the apron. This alteration eliminated the concentration against the left side of the intermediate wall and less erosion resulted even though the wall seemed too low.

After this scour-minimizing method was developed, the problem of obtaining the most economical size of wall was studied. The wall was raised to prevent interference by water flowing over the top onto the spillway jet and shortened 22 feet to a point (Test 37) where the difference in water depth along the two sides of the wall was about 1 foot prototype at the downstream end. The 3-foot Rehbock sill was moved upstream until the front face was at the corner of the left intake wall (Figure 24). The results were substantially the same as with the longer wall. However, the eddy along the bank became slightly more pronounced and crowded the flow from the intake structure toward the wall causing slightly more erosion on that side than on the spillway side.

The fact that this increase was at the downstream end indicated that the wall could not be shortened, especially at the base. The water surface on the intake side seldom reached to within 2 feet of the top, and the wall was lowered for the next test. With the downstream limit determined, attention was transferred to the upstream end. The walls previously tested were constructed partly on the existing prototype apron. As this complicated the construction, the wall was cut vertical at the junction of the new and old construction and the upstream end given the shape of a parabolic pier nose (Test 38). The top of the wall was cut on a slope making it similar to the design obtained by model studies for the training walls on the Marshall Ford Dam (Figure 24). The wall was overtopped slightly at 150,000 second-feet, but this arrangement gave good results with the Rehbock sill in place. The necessity of this sill was clearly shown when it was replaced by the trapezoidal shape and the conditions became more severe below the intake section (Figure 25).

A 2-hour run at 150,000 second-feet produced more scour than in previous tests. The wall was revised and lengthened 10 feet so that its total length became 69 feet and the 2-hour run was repeated. Favorable results occurred, especially when compared with a similar run using no wall (Figure 26). This intermediate wall (Test 38, Figure 24) was recommended in the event the rock foundation near the intake section proved unsound or was found to be at a lower elevation than indicated by the borings made in 1905.

A. River bed before run

B. Discharge 150,000 second-feet

C. Bed after one-hour run

REHBOCK SILL

A. River bed before run

B. Discharge 150,000 second-feet

C. Bed after one hour run

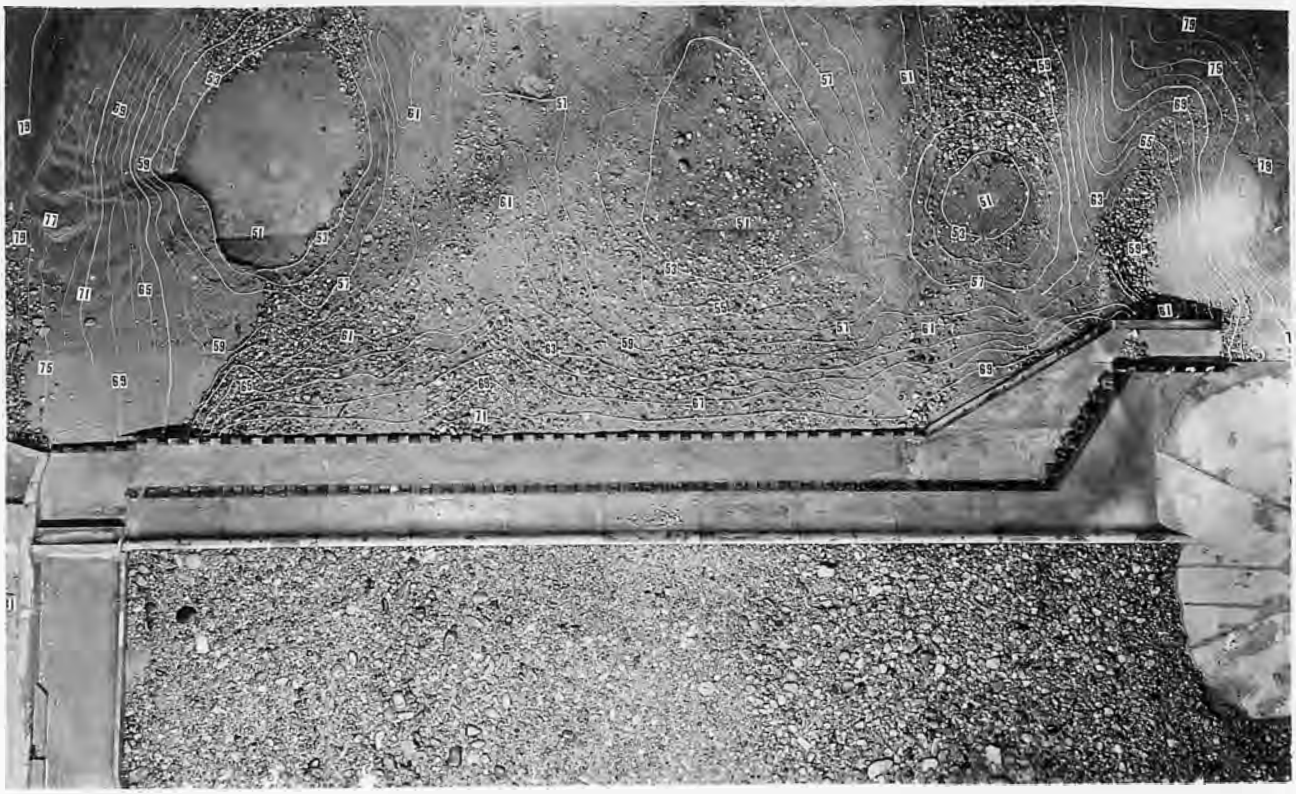
TRIANGULAR SILL

1:48 MODEL

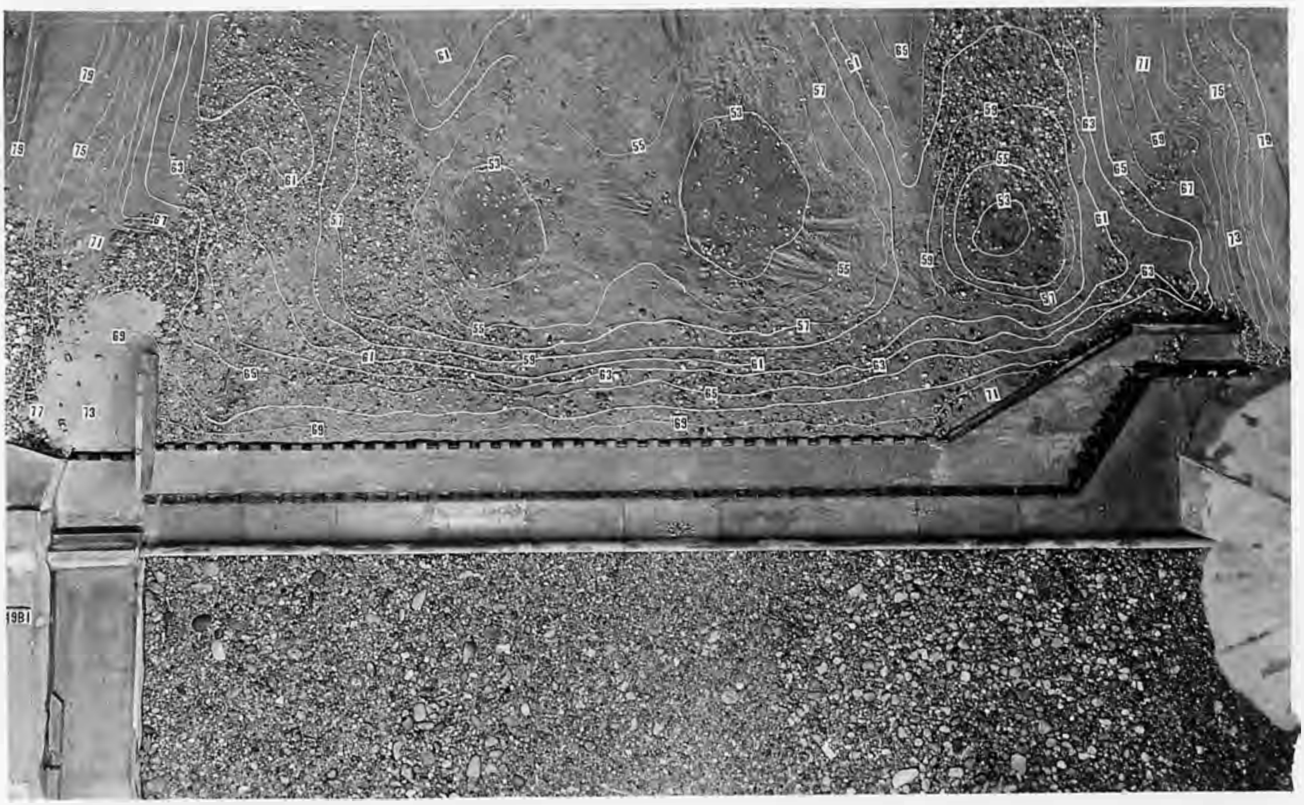
COMPARISON OF INTERMEDIATE TRAINING WALL WITH

DIFFERENT TYPE OF SILL DOWNSTREAM FROM THE INTAKE SECTION

Figure 25



A. Without intermediate training wall



B. With intermediate training wall

1:48 MODEL
RIVER BED AFTER DISCHARGE OF 150,000 SECOND-FEET FOR 2 HOURS (MODEL)
WITH TAILWATER 4-FEET BELOW MINIMUM ESTIMATED

CONCLUSIONS

Large objects, such as the displaced section of the original dam, should not be left immediately downstream from a dam similar to the one discussed. Undermining at the upstream side of such an object will occur, and the object is likely to move upstream to damage the toe of the dam.

A major factor causing failure of the original dam was due to piping resulting from severe scour at the upstream face of the dam. This condition was shown on the model, but was impossible to observe on the prototype.

The best protection upstream from the dam consisted of a blanket of riprap 25 feet wide and 3 feet deep placed along the entire length of the spillway (Figure 30).

Protection for the downstream parts of the dam considered best prior to the 1937 flood and based on uniform flow over the spillway consisted of a 30-foot apron with a 2-foot Rehbock sill at the end. Minimum scour was obtained downstream from the end of the apron and along the cutoff wall of the right abutment by extending the upstream portion of the abutment 25 feet, extending the apron in this area, and using a 4-foot instead of a 2-foot Rehbock sill (Figure 27).

The best redesign for dam protection downstream after the 1937 flood consisted of:

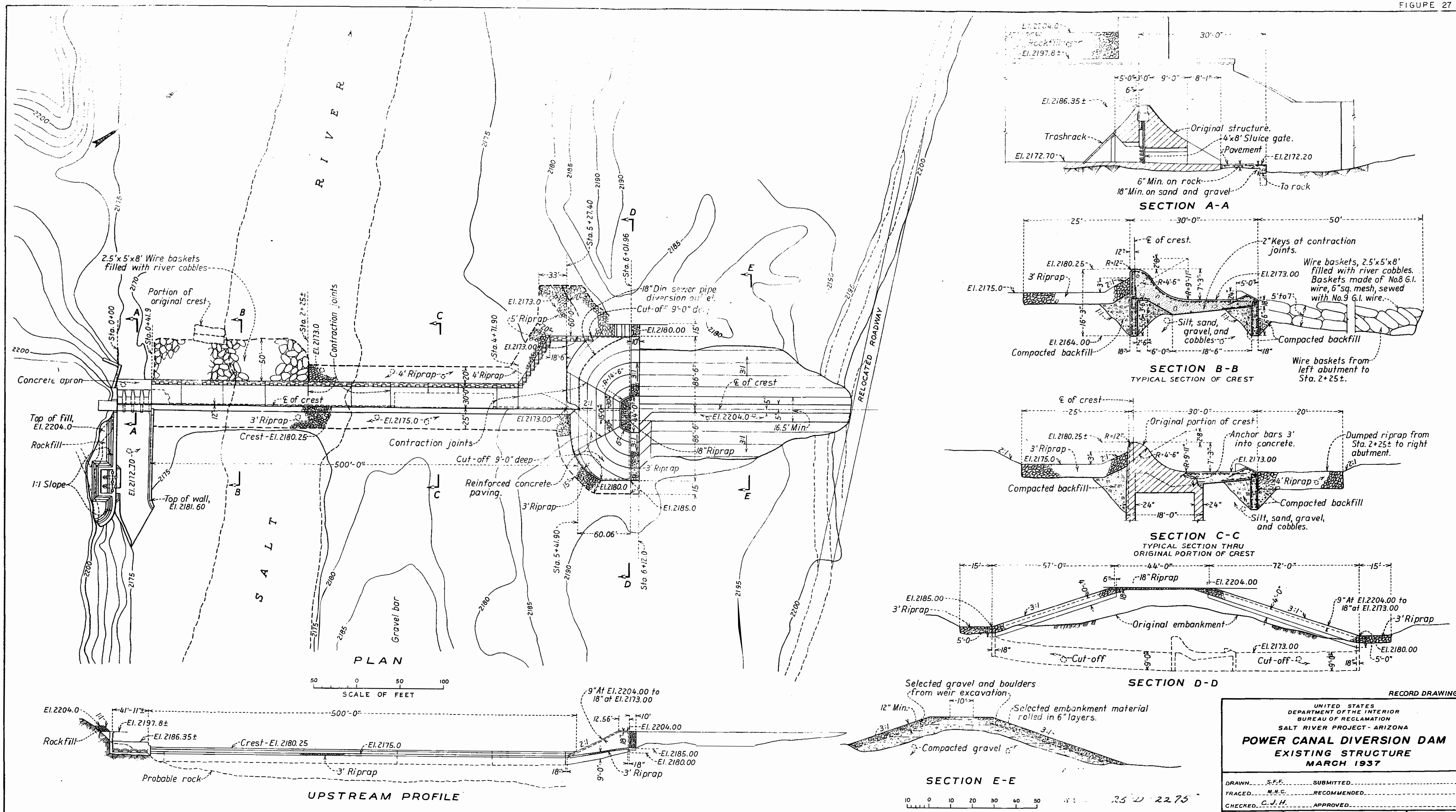
a. A 25-foot sloping apron extension with a 3-foot Rehbock sill in the spillway section (Figure 30)

b. A section of level apron, with a 3-foot trapezoidal sill, extended downstream from the right abutment (Figure 30), and

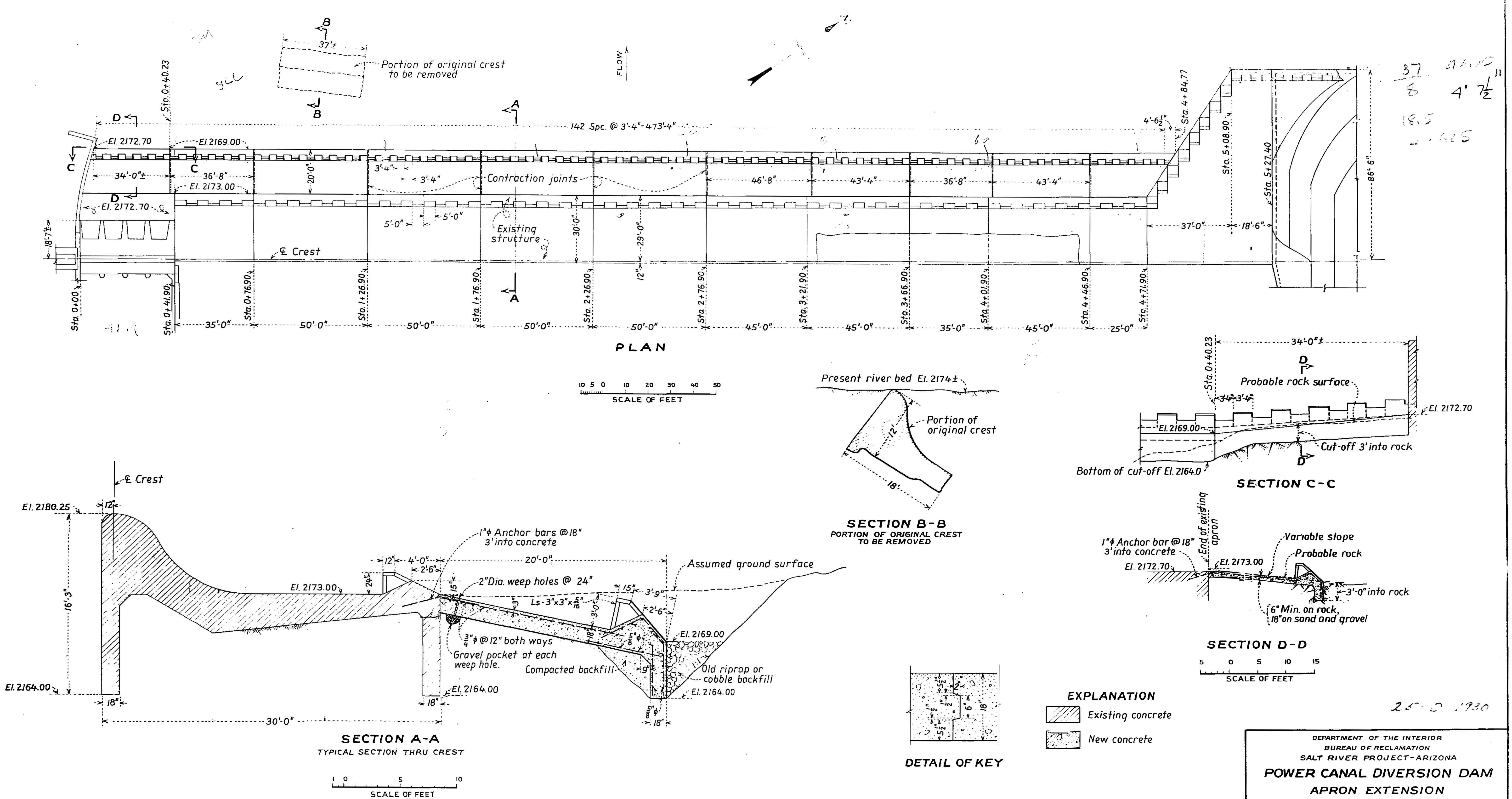
c. An intermediate training wall immediately downstream from the right wall of the intake structure. This wall is required only of the rock foundation in this area is proved unsound or is at a lower elevation than indicated by the borings made in 1905

Slight negative-crest pressures, that occur at 25,000 and 50,000 second-feet flow, are not considered serious.

FIGURE 27



SALT RIVER
PLAN
SCALE OF FEET
RELOCATED ROADWAY
Gravel bar
2.5'x5'x8' Wire baskets filled with river cobbles
Portion of original crest
Concrete apron
Top of fill, El. 2204.0
Rockfill
1:1 Slope
Top of wall, El. 2181.60
Crest-El. 2180.25
3' Riprap
4' Riprap
5' Riprap
18" Riprap
Contraction joints
Cut-off 9'-0" deep
Reinforced concrete paving.
18" Dia. sewer pipe diversion outlet
℄ of crest
500'-0"
Sta. 0+00
Sta. 0+41.9
Sta. 2+25±
Sta. 4+71.90
Sta. 5+27.40
Sta. 5+41.90
Sta. 6+01.96
Sta. 6+12.0
SECTION A-A
Trashrack
Original structure.
4'x8' Sluice gate.
Pavement
El. 2186.35±
El. 2172.70
El. 2172.20
6" Min. on rock
18" Min. on sand and gravel
To rock
SECTION B-B
TYPICAL SECTION OF CREST
2" Keys at contraction joints.
Wire baskets, 2.5'x5'x8' filled with river cobbles. Baskets made of No.8 G.I. wire, 6" sq. mesh, sewed with No.9 G.I. wire.
Silt, sand, gravel, and cobbles
Compacted backfill
Wire baskets from left abutment to Sta. 2+25±.
SECTION C-C
TYPICAL SECTION THRU ORIGINAL PORTION OF CREST
Original portion of crest
Anchor bars 3' into concrete.
Dumped riprap from Sta. 2+25± to right abutment.
SECTION D-D
Original embankment
Cut-off
9" At El. 2204.00 to 18" at El. 2173.00
SECTION E-E
Selected gravel and boulders from weir excavation
Selected embankment material rolled in 6" layers.
Compacted gravel
12" Min.
UPSTREAM PROFILE
Probable rock
RECORD DRAWING
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SALT RIVER PROJECT - ARIZONA
POWER CANAL DIVERSION DAM
EXISTING STRUCTURE
MARCH 1937
DRAWN S.F.F. SUBMITTED
TRACED M.H.C. RECOMMENDED
CHECKED C.J.H. APPROVED
DENVER COLORADO JUNE 1, 1937
25-D-1823



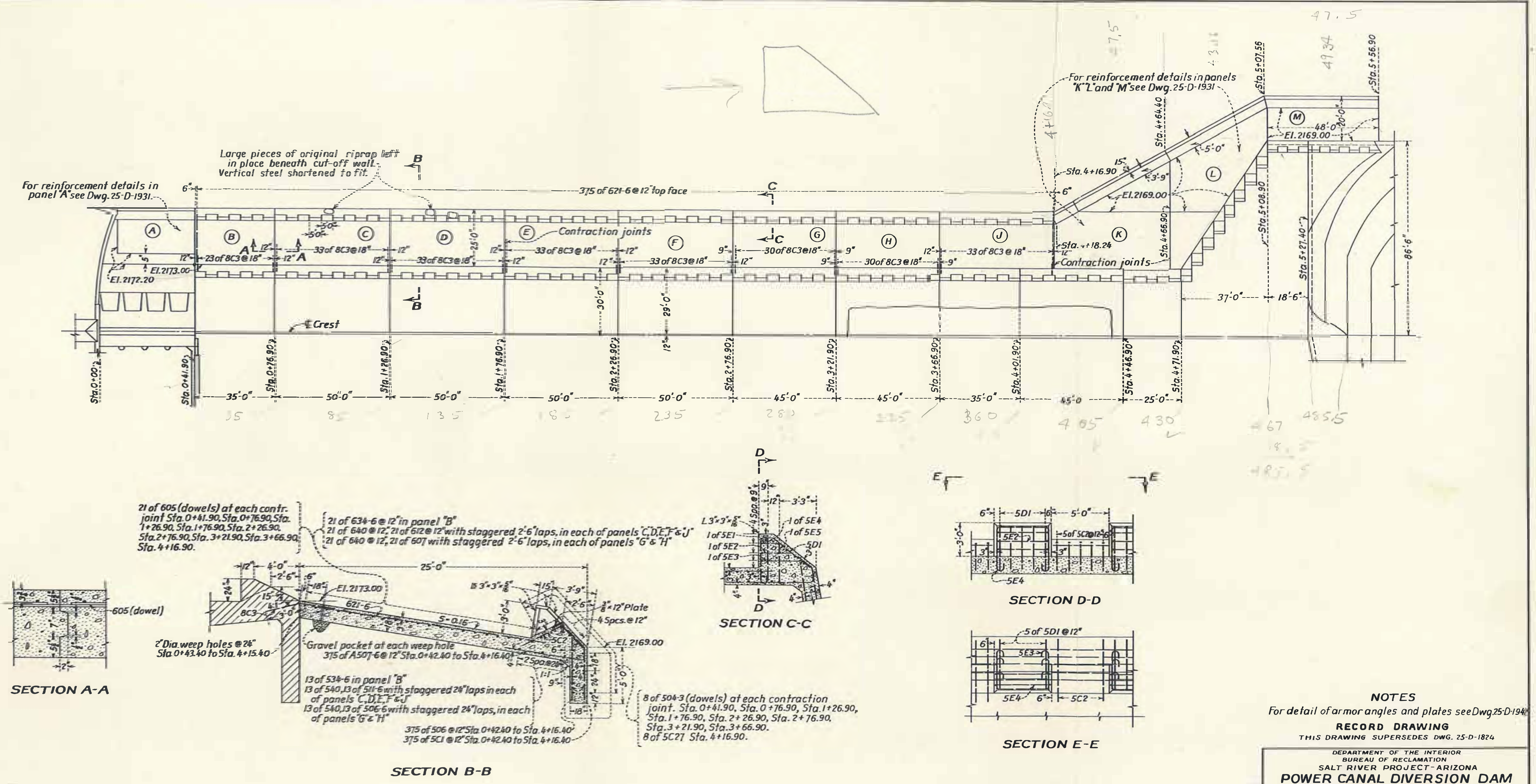
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SALT RIVER PROJECT-ARIZONA

POWER CANAL DIVERSION DAM

APRON EXTENSION

DRAWN J.H.L. SUBMITTED J.J. Hammond
TRACED M.T.Y. RECOMMENDED W.H. Nalder
CHECKED L.M.S. APPROVED R.F. Walter

DENVER, COLORADO, JUNE 1, 1937 125-D-1824



NOTES

For detail of armor angles and plates see Dwg. 25-D-1941

RECORD DRAWING

THIS DRAWING SUPERSEDES DWG. 25-D-1824

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SALT RIVER PROJECT-ARIZONA

POWER CANAL DIVERSION DAM

APRON EXTENSION

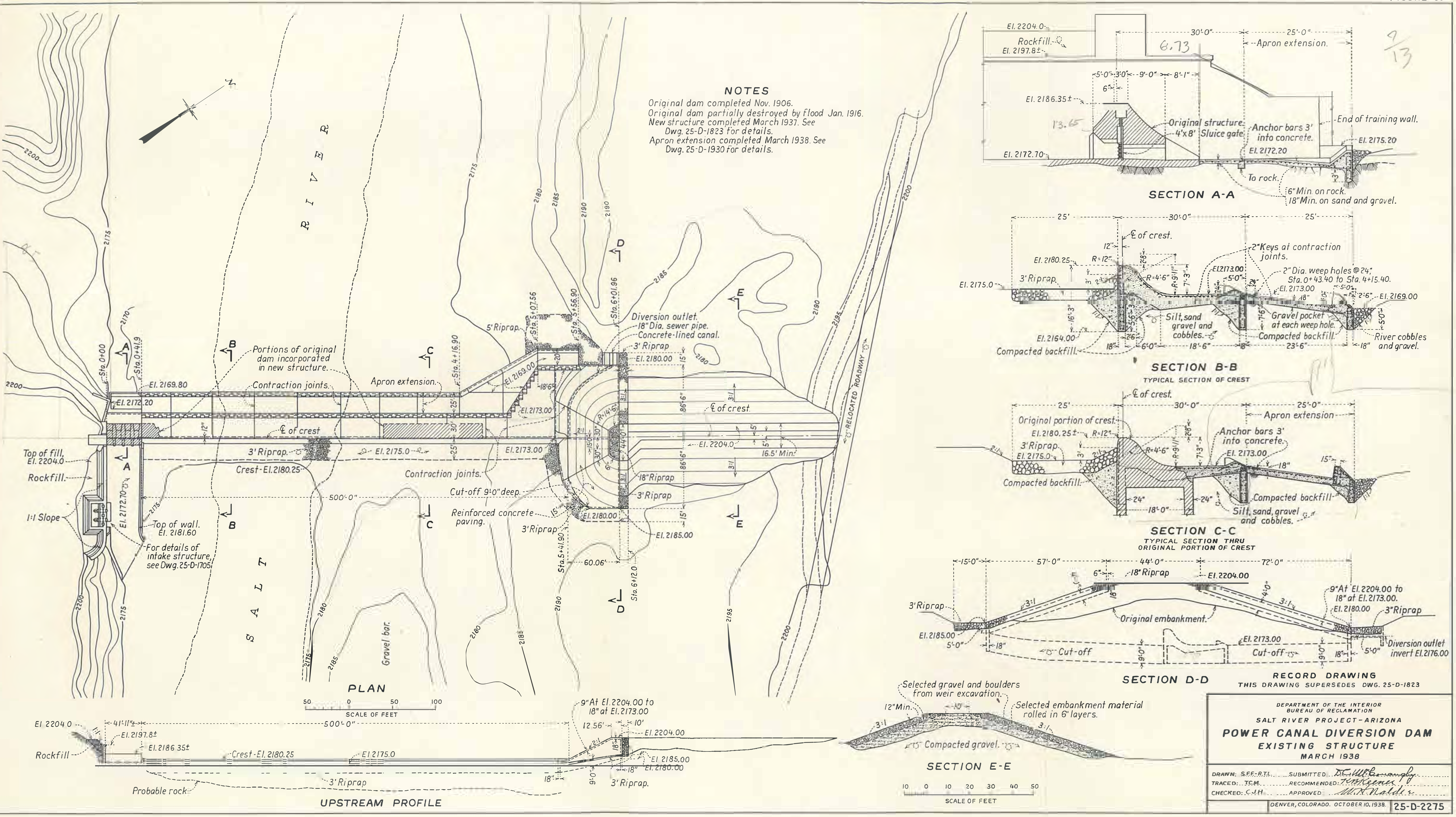
REINFORCEMENT DETAILS
STA. 0+41.90 TO STA. 4+16.90

DRAWN J.H.L. SUBMITTED
TRACED E.W.O. RECOMMENDED
CHECKED DVCB APPROVED

DENVER, COLORADO, AUG. 11, 1937 25-D-1930

TRACED AND REVISED 10-25-38

FIGURE 30



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