UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF WEBSTER DAM SPILLWAY

Hydraulic Laboratory Report No. Hyd-390

ENGINEERING LABORATORIES

OFFICE OF THE ASSISTANT COMMISSIONER AND CHIEF ENGINEER
DENVER, COLORADO

November 26, 1954

FOREWORD

Hydraulic model studies of the Webster Dam Spillway, a part of the Missouri River Basin Project, were conducted in the Hydraulic Laboratory of the Bureau of Reclamation at Denver, Colorado, during the period October 1952 to August 1953.

The final plans evolved from this study were developed through the cooperation of the staffs of the Spillway and Outlets Section and the Hydraulic Laboratory.

During the course of the model studies, Messrs. H. W. Tabor, C. J. Hoffman, and G. H. Austin of the Spillway and Outlets Section frequently visited the laboratory to observe the model tests and discuss the results.

These studies were conducted by G. L. Beichley with the aid of Dr. Abdias Guzman from the University of Colombia at Bogata, Colombia, South America. The studies were supervised by W. E. Wagner, A. J. Peterka, and J. N. Bradley under the Hydraulic Laboratory direction of H. M. Martin.

CONTENTS

Figure

Figure

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Office of the Assistant Commissioner
and Chief Engineer
Engineering Laboratories
Denver, Colorado
November 26, 1954

Laboratory Report No. Hyd-390
Hydraulic Laboratory
Written by: G. L. Beichley
Reviewed by: A. J. Peterka

Subject: Hydraulic model studies of Webster Dam Spillway

SUMMARY

Hydraulic model studies of Webster Dam Spillway (Figures 1 through 8) were made on a 1:54 scale model (Figures 9, 10, and 11) for the purpose of developing and checking the hydraulic design. Data and notes taken on the flow in the model showed that the general concept of the preliminary design was satisfactory. However, the following design modifications and developments were accomplished: the inlet walls in the spillway approach (Figures 12, 14, and 15) were modified to provide economy without loss of satisfactory flow conditions; the location of the auxiliary float well intake was determined; the radial gate trunnion was relocated at a higher elevation so that it and the gate counterweight would clear the water surface (Figures 16 and 17); the minimum height of the training walls along the chute and stilling basin was determined from water surface profile measurements (Figure 23); and the stilling basin was modified to eliminate scouring in the discharge channel principally at the downstream corners of the basin (Figures 24, 29, 30, 31, 32, 33, and 35); the spillway was calibrated for use in operating the spillway gates of the prototype to control discharges (Figures 18 and 19), and the spillway was checked for subatmospheric pressures (Figure 20). Motion pictures were made showing the final recommended spillway discharging the design flow in the spillway approach, gate section, chute, and stilling basin. Motion pictures were also made of smaller discharges in the recommended stilling basin.

INTRODUCTION

Webster Dam is a part of the Webster Unit of Solomon Division of the Missouri River Basin Project. It is located on the south fork of the Solomon River approximately 1 mile downstream from the village of Webster in Rooks County in north central Kansas as shown on the location map in Figure 1. The dam, shown in Figures 2 and 3, is an earth-fill embankment approximately 10,600 feet long at the 30-foot wide crest with a maximum height of approximately 110 feet above the river bed in the diversion channel.

The concrete spillway is located near the left abutment. The spillway consists of an excavated approach channel, concrete lined at the downstream end; spillway crest structure with three radial gates, hoist deck, and highway bridge; a concrete-lined chute and stilling basin; and an excavated outlet channel which leads to the Solomon River; all shown in Figures 2 through 8. The spillway approach and outlet channels are to be lined with a crushed-rock and riprap blanket adjacent to the concrete structure.

The spillway is 116 feet wide at the crest and 843 feet long from the axis of crest to the end of the stilling basin. The crest is at elevation 1884.60 which is 5 feet below the normal reservoir elevation and 53.4 feet below the maximum reservoir. The flow is controlled by three 33-foot 4-inch wide radial gates separated by 8-foot piers. The spillway is designed to discharge 138,000 second feet which corresponds to 1,380 second feet per foot of usuable crest length. The flow drops a vertical distance of 82.6 feet in a horizontal distance of 713 feet measured from the crest axis to the upstream end of the stilling basin. The stilling basin is 264 feet wide by 130 feet long. Chute blocks are used along the upstream edge of the stilling basin and a dentated end sill at the downstream end as shown in Figure 8. Stilling basin wing walls are provided near the downstream end of the basin at right angles to the direction of flow.

THE MODEL

The model was constructed and tested in the Bureau of Reclamation Hydraulic Laboratory at the Federal Center near Denver, Colorado. It was a 1:54 scale reproduction of the spillway and surrounding area as shown in Figures 9, 10, and 11.

The reservoir topography was reproduced for a distance of approximately 700 feet upstream from the spillway crest and for 324 feet to the right and left of the spillway center line. Downstream from the end of the stilling basin, topography was reproduced for a distance of approximately 475 feet and for a distance of 324 feet to the right and left of the spillway center line.

Topography in the reservoir area of the model was molded of concrete mortar placed on metal lath which has been nailed over wooden templates shaped to the ground surface contour as shown in Figure 10. Model concrete surfaces simulating nonconcrete surfaces of the prototype, such as topography, were given a rough finish while concrete surfaces simulating concrete surfaces in the prototype were given a smooth finish. Topography in the downstream area was formed in sand in order to provide a movable bed in which to study the erosion characteristics of the flow leaving the structure.

The spillway crest, chute, and stilling basin floor were molded in cement mortar. Sheet-metal templates accurately cut and placed were used as guides. Piezometers over the crest section consisted of 1/16-inch inside-diameter copper tubes that were soldered at right angles to the profile shape of the template and filed flush.

Water was supplied to the model by means of the laboratory's permanent supply system. The water was pumped from the underfloor reservoir through a 12-inch main supply line to the model. The discharge was regulated at the automatic control board and measured by a venturi meter of appropriate size. The reservoir elevation was measured with the hook gage in well located approximately as shown in Figure 9. The tail water elevation was controlled by the tail water control gate and measured by use of a permanently mounted point gage. Certain water surface profiles were recorded by means of a sliding point gage mounted on a rail while others were recorded by measuring the depth of flow. Pressures on the spillway crest were measured by use of nine piezometers on the center line of spillway shown in Figure 9.

Head losses due to friction in the model are usually greater proportionately than indicated by the model scale because surfaces sufficiently smooth to represent prototype surfaces to scale do not exist. Therefore, to maintain the scale velocity throughout the model chute, it was necessary to either increase the slope of the chute or reduce the chute length. For this structure it was advantageous to increase the slope so that the geometrical similitude of the diverging chute in plan would be unaltered. It would then be possible to observe and study the flow pattern throughout the chute as it would occur in the prototype.

The slope correction was applied only to the constant sloping portion of the chute which extended from Station 9+80, at the downstream end of the crest profile, to Station 15+19.58, at the P. I. of the vertical curve. The slope required for the model chute was computed to be 0.0315 as compared to 0.02 in the prototype. This increased slope amounted to an additional drop of 1.38 inches between these two stations in the model. With this slope correction the velocity of the design flow entering the stilling basin more truly represented the prototype velocity and the energy to be dissipated in the model basin more closely represented the prototype energy.

THE INVESTIGATION

The investigation was concerned with the performance of the spillway and with the erosion of the river bed caused by the flow leaving the stilling basin. The maximum design flow which during the course of this investigation was 136,000 second feet was of primary concern. This discharge corresponds to about 1,360 second feet per foot of usable crest length with a head on the crest of 53.1 feet. After completion of this study the maximum head was increased to 53.4 feet and the maximum discharge to 138,000 second feet. To a lesser degree, the investigation was concerned with the spillway discharging flows less than maximum to be certain that the structure operated as intended and that the erosion pattern was satisfactory over the entire discharge range. The investigation included the testing of the spillway approach, gate section, chute, and the stilling basin as well as the investigation of the erosion pattern caused by the flow leaving the basin.

Spillway Approach

Flow Characteristics

The model of the spillway approach area with the preliminary inlet wall design is shown in Figure 12(A). Figures 12(B), 12(C), and 12(D) show 136,000 second feet approaching, entering, and passing through the gate section. The flow approached the spillway quite satisfactorily for all discharges; however, the flow piled up on the pier noses as shown in Figures 12 and 13 and the water surface was drawn down around them for discharges near maximum. This was a matter of interest however, rather than concern.

Development of the Inlet Walls

Flow along the preliminary inlet walls was very smooth even for the maximum design discharge as shown in Figure 12(B), but it was decided by the designers to test four alternate inlet wall designs that were more economical for prototype construction. All four alternate designs operating with the maximum discharge of 136,000 second feet are shown in Figures 14 and 15. Design No. 3 was the most economical design to construct, but the drawdown around the vertical portion of inlet wall was considered to be too great; therefore, the second most economical structure shown as

Design No. 4 was adopted by the designers for prototype construction. This design caused some disturbance along the sloping walls for discharges near maximum, but tests showed that this disturbance did not reduce the capacity of the spillway. Motion pictures of the design flow along the recommended inlet walls were taken.

Determination of the Auxiliary Float Well Intake Location

Float wells are provided in the piers of the prototype structure to adjust the radial gate openings automatically when the reservoir elevation fluctuates. The main entrance intake for the float wells is located in the approach channel approximately 500 feet upstream from the spillway and below crest elevation. For high reservoir elevations this intake entrance and the intake supply line to the float wells is very deeply submerged and emergency repairs would be very difficult. Therefore, an auxiliary intake and supply line was to be provided and its location was to be determined from model tests.

It was necessary that the head on the auxiliary float well intake represent the reservoir elevation very closely. A velocity head drawdown of approximately one-half foot from reservoir elevation was considered allowable by the designers for the maximum design discharge. Three locations for the auxiliary intake were tested in the model.

The first location tested was at elevation 1877 on the right pier nose. A piezometer in the model at this point revealed that for maximum design discharge, the pressure head was about 2.6 feet less than reservoir elevation as shown in Figure 13. This location, therefore, was considered unsuitable.

The second location tested was in the reservoir on the extended line of the left training wall and about 3 feet above the ground surface with the intake opening facing downstream. Measurements were made to determine how far upstream along this line the intake entrance would need to be to meet the velocity head drawdown limitation. Velocities in the direction of flow were measured along this line in the model, at about 3 feet above the ground surface, using a Stevens midget current meter. The following results were obtained:

75 feet upstream from the crest axis and in line with left training wall--11.8 ft/sec--2.16-ft drawdown

115 feet upstream from the crest axis and in line with left training wall--9.5 ft/sec--1.40-ft drawdown

160 feet upstream from the crest axis and in line with left training wall--7.4 ft/sec--0.85-ft drawdown

187 feet upstream from the crest axis and in line with left training wall--5.9 ft/sec--0.54-ft drawdown

Therefore, it was necessary that the intake be located at least 187 feet upstream where the velocity in the direction of flow was about 6 feet per second and the velocity head was a little over one-half foot. This was too far to be desirable.

After the revised inlet walls were developed, a third location in the face of the left inlet wall near the upstream end shown in Figure 5 was tested and recommended for prototype construction. A point gage measurement at this point in the model showed the water surface to average approximately 0.4 of a foot below the reservoir elevation which was acceptable. Surface waves in the intake area, shown in Figure 15(D), measuring 0.5 of a foot high from crest to trough were objectionable, but since the auxiliary intake would be used only in an emergency, and since in the wall it was more easily accessible than in the reservoir, the wall location was considered satisfactory by the designers.

Spillway Gate Section

Water Surface Profiles

Flow through the gate section is shown in Figure 12(D) for the design discharge of 136,000 second feet and in Figure 11(B) for 11,000 second feet through a 3-foot opening of the gates. Flow around the trailing edge of the piers was satisfactory. Motion pictures of the flow through the gate section were made for the design discharge.

Water surface profiles shown in Figure 16 were recorded along the right and left training walls of the gate section, the center line of the spillway, and along both faces of both piers for the maximum discharge of 136,000 second feet. The profiles showed that the counterweight frames on the three radial gates shown in Figure 7 would be partially submerged when the gates were fully open with the gate pin at the preliminary location. In the preliminary design the gates seated on the crest axis, but as a result of these water surface measurements, the designers moved the gate pin higher and farther downstream so that the gates seated downstream from the crest axis as shown in Figures 6 and 7.

With the gates relocated and fully open for free flow, transverse water surface profiles shown in Figure 17 were recorded for the design discharge under the bottom edge of the gates and under the counterweight frames. From these profiles, the minimum bottom elevation of the skin plates and the minimum bottom elevation of the counterweight frames were determined for the design flow. As a result of these tests, the designers set the bottom of the gate at elevation 1930 for gates fully open. The relocation of the gates pin as tested here was, therefore, recommended for prototype construction.

Calibration

Calibration of the free crest disclosed that the crest was capable of discharging the maximum design discharge of 136,000 second feet at maximum reservoir elevation as shown in Figure 18. The efficiency of the crest was indicated by the discharge coefficient in the equation:

$$Q = CLH^{3/2}$$

where

Q is the discharge

L is the crest length, and

H is the total head or difference in elevation of reservoir and crest

For the maximum discharge the coefficient was approximately 3.53 as shown by the discharge coefficient curve in Figure 18.

The crest section was calibrated for gate-controlled flow for use in prototype operation of the structure. The gates were calibrated both in the preliminary and revised location. Both are shown in Figure 18. For the very small gate openings of about 3 feet or less, and for the large gate openings of about 40 feet or more, the discharge was about the same whether the gates seated on the crest axis as for the preliminary location or whether they seated downstream as for the final recommended location. For any given gate opening between these two extremes there was less flow when the gates seated downstream from the crest axis. The reason for this is that the effective gate opening, that is, the shortest distances between the crest profile and the bottom edge of the gate, is less when the gates seat downstream from the crest axis. The gate-controlled discharge curves for the recommended gate location are cross plotted in Figure 19 so that discharges may be quite accurately interpolated for any combination of reservoir elevation and gate opening.

Pressures

Pressures on the crest profile of the spillway were recorded for a range of uncontrolled discharges as shown in Figure 20(A). All pressures were considerably above atmospheric. Pressures were also recorded for a range of controlled flows with the gate in both preliminary and recommended locations as shown in Figure 20(B) and 20(C), respectively. In all cases pressures were above atmospheric, except for small gate openings which showed subatmospheric pressures of approximately 2 feet of water. Since the discharge coefficient was satisfactory and no severe subatmospheric pressures were encountered, the crest shape is recommended for prototype construction.

Spillway Chute

Flow entering the chute is shown in Figure 11(B) for 11,000 second feet with the gates open 3 feet and in Figure 12(D) for the design discharge of 136,000 second feet. Figure 21 shows the flow through the chute for the design discharge. A standing wave occurred on the center line of the spillway chute a short distance downstream from the gate section as shown in Figure 21(C) and others occurred as shown in Figure 21(D); however, the flow entered the stilling basin fairly uniformly distributed from one training wall to the other as shown in Figures 21(A) and (B). For 136,000 second feet the flow entering the basin was slightly more concentrated near the center of the chute as shown in Figure 22. For all lesser discharges the flow in the chute was fairly uniform from one training wall to the other before entering the stilling basin. This good distribution was instrumental in providing the good action observed in the stilling basin. The flow distribution appeared to be satisfactory also for gate-controlled flows if the openings of the three gates were symmetrical.

Water surface profiles were recorded along the left training wall for several discharge and gate setting combinations as shown in Figure 23. The profiles were used by the designers for determining the most economical training wall heights. The left training wall was chosen for the water surface profile measurements because the flow through the chute was in general slightly deeper along the left training wall than along the right. The reason for this was that the spillway approach was not symmetrical about the center line of the spillway.

Except for training wall heights, the chute as preliminarily designed is recommended for prototype construction. Motion pictures of the flow through the chute were made for the design discharge.

Spillway Stilling Basin

Preliminary Basin

The stilling basin was tested to develop an economical basin that would dissipate the energy of the flow satisfactorily. The preliminary stilling basin discharging the design flow of 136,000 second feet is shown in Figures 24, 25, and 26.

A 30-minute model erosion test with the basin discharging the design flow showed some erosion to occur at the downstream corners of the structure and in the center of the discharge channel approximately 100 feet downstream from the structure as shown in Figure 24(C). Except for the corners, the downstream edge of the structure was protected very well by a ground roller that deposited bed material there. The discharge channel near the corners of the basin was eroded by side eddy undercurrents. Erosion at the left corner was not quite as severe as erosion at the right because the higher channel bank on the left helped to prevent formation of the side eddy current. At the intersection of the channel banks and the 90° wing walls erosion occurred along the wing walls as a result of the side eddies. The banks of the discharge channel were also eroded due to sloughing of the wet sand but this does not truly represent a prototype condition. Sloughing is common in model erosion tests where steep banks are molded in sand and does not necessarily represent the action on prototype banks which are usually of more stable material.

The stability of the jump was determined for a range of discharges by lowering the tail water elevation below the expected tail water elevation shown in Figure 27. The tail water was lowered first to the elevation at which the chute blocks became partially visible, and then further to the elevation at which the jump swept out of the basin. These elevations are shown as curves in Figure 27 for a range of discharges near the design flow.

The factor of safety between the expected tail water elevation and the elevation at which sweep out occurred appeared to be more than ample; about 11 feet at maximum discharge and increasingly more than 11 for smaller discharges. In fact, it was necessary, in conducting the tests, to lower the elevation of the discharge channel in order to lower the tail water sufficiently to cause the sweep out.

The basin was considered to operate at maximum efficiency as long as the chute blocks were covered. With the chute blocks partially uncovered some of the basin was not utilized in dissipating energy. The chute block visibility curve in Figure 27 indicates the tail water elevation at which some of the chute blocks momentarily appear and then disappear. The curve shows that the tail water can be lowered several feet before the basin efficiency is reduced. For the maximum discharge the tail water could be lowered 2 feet, shown in Figure 25(B), without any part of the chute blocks becoming visible. For smaller discharges the margin became rapidly greater.

The necessity for chute blocks on the apron was determined as shown in Figure 28. Figures 28(A) and (B) show a basin operating with and without the blocks. The blocks were found to be a real aid in helping the jump to form well upstream in the basin and in increasing the stability of the jump.

Water surface profiles were recorded along the left training wall of the stilling basin, Figure 23. The profiles show that the height of the preliminary basin walls can be reduced. The profiles also show that the chute blocks are well submerged at the upstream end of the basin.

The effect of the dentated end sill was determined by testing the preliminary basin without one, Figure 29. The water surface at the toe of the jump was rougher without the sill than with, but downstream from the jump the water surface was smoother as shown by comparison of Figures 24 (B) and 29(A). Without the sill the erosion was much more severe as shown by comparison of Figures 24(C) and 29(B). For the design discharge the entire downstream edge of the basin was in danger of being undermined by erosion. Therefore, the use of a dentated end sill is very essential.

While the dentated end sill was out of the basin an interesting bit of data was observed. It was learned that with no sill the jump remained in the basin for lower tail water depths than when the sill was used, as shown by the sweep out curves in Figure 27. The reason for this was that without the sill the length of the hyddrualic jump extended beyond the end of the basin. This permitted water to flow back into the basin from the sides making sweep out more difficult.

It was concluded from these tests that the preliminary basin was well designed. However, it was thought that perhaps a more economical structure could be developed and that perhaps the erosion at the corners of the basin could be reduced without increasing the cost of the structure.

Proposed Stilling Basin Modifications Tested

Several proposed modifications to the stilling basin were tested to develop the most economical basin and to reduce the erosion at the apron corners. The erosion test results for most of the modifications tested are shown in the photographs of Figures 30 and 31. A summary of all erosion test results is shown in the table in Figure 32.

With the apron floor 4 feet higher than in the preliminary basin the structure would be more economical to construct, but the erosion at the apron corners was increased, Figure 30(A), and the water surface was rougher. With the apron elevation unchanged and the dentated end sill of the preliminary design moved upstream, erosion was increased still more as shown in Figure 30(B). With the preliminary basin lengthened 45 feet the erosion at the corners of the basin was not improved, Figure 30(C). With 45° spur walls added at the basin corners as shown in Figure 30(D), erosion at the corners was eliminated but the structure became more costly. Replacing the dentated end sill with a smaller sill with wider slots and smaller dentates, approximately $0.15 d_2$ high, was not effective in improving the scour, Figure 31(A). A basin with a sloping apron was a more economical design, but the erosion pattern was more severe than for the preliminary design, Figure 31(B), and the sweep out factor of safety was reduced as shown by the sweep out curve in Figure 27.

A low spur wall that extended only 10 feet above the basin floor elevation was added to each side of the basin and was found to produce very unsatisfactory erosion. A short portion of the dentated sill was removed at each end of the preliminary sill adjoining the basin training walls, and this, too was found to be unsatisfactory. An additional high block placed on the ends of the sill adjoining the basin training walls did not improve the scour pattern either. It was also found that the scour pattern at the corners was not affected by the use of a slot instead of a dentate adjoining the basin walls.

Since the preliminary basin plus the additional 45° spur walls that extended above the water surface proved satisfactory as shown in Figure 30(D), it was decided to test a more economical arrangement of this structure to accomplish about the same thing. The training walls of the basin were turned outward from the upstream end of the sill on a 45° angle to simulate to some extent the 45° spur walls. Erosion test results shown in Figure 31(C) were better than for the unaltered preliminary design because no erosion was found to occur at the corners, but, instead, occurred 40 or 50 feet in from the corners along the end sill. Some erosion still occurred along the 90° wing walls.

The 45° diverging walls made it possible for the designers to relocate the 90° wing walls 12 feet upstream from the preliminary location at the downstream edge of the sill. The designers considered this a step in the direction of economy, so the model was tested for this arrangement, Figure 31(D). The erosion pattern indicated that a little less erosion occurred along the 90° wing walls and at the end sill; compare Figures 31(C) and (D). The results of these two modifications led to the development of the recommended stilling basin.

Recommended Stilling Basin

The recommended design included the 45° corner training walls and relocation of the preliminary 90° wing walls 12 feet upstream from the end of the basin, plus one further modification of the end sill as shown in Figures 8 and 33. The slots between dentates at each end of the sill were filled in to sill height and fillets were added to the sill at the downstream corners to conform to the slope of the channel banks. The slots were filled in at each end of the sill as far out from the training wall as erosion along the sill occurred in the preceding test described above. Six slots on each end were therefore filled; a distance of 64.5 feet out from the training wall.

An erosion test shown in Figures 33(A) and (B) showed that a small amount of scour occurred near the ends of the 45° training walls, but the bottom elevation of the hole was still above the elevation of the basin floor. No erosion occurred along the 90° wing walls except for unavoidable sloughing of the model sand banks. Erosion tests also showed that the scour pattern was improved if a dentate rather than a slot was adjacent to the 45° training walls.

The boil over the sill for the design discharge was greater than for the preliminary design, particularly over the filled in ends of the sill as shown in Figures 33(C) and (D) but this was not considered objectionable. Sweep out tests were not made since the chute blocks were well submerged by the hydraulic jump for all discharges with normal expected tail water as shown in Figure 4. The curves for the preliminary design in Figure 27 should indicate the sweep out characteristics of the basin.

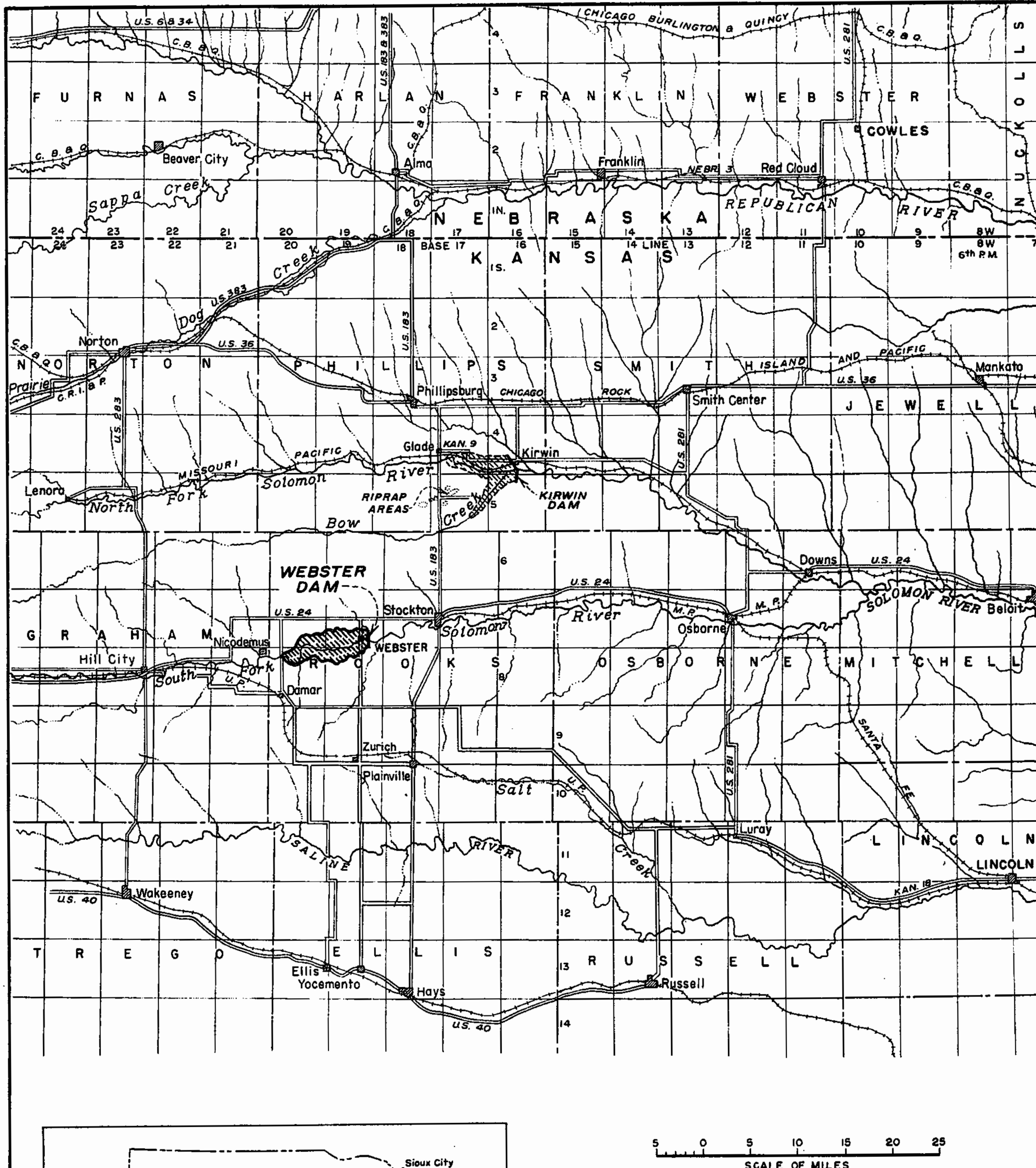
The flow distribution across the width of the basin was quite uniform for all discharges as shown in Figure 34 except for 1/4 maximum discharge. For 1/4 maximum discharge the appearance of the hydraulic jump indicated a tendency for flow concentration near the training walls. This is not important, however, since the entire length of the basin is not needed to fully dissipate the energy of a small flow entering it. Motion pictures were made of the basin discharging the flows shown in Figure 34.

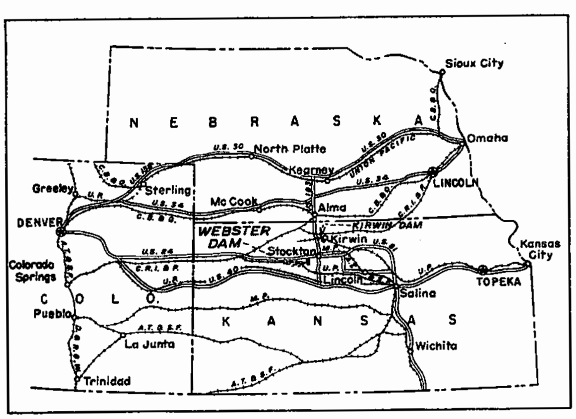
Further erosion tests were conducted using 3/4-inch crushed rock to represent prototype riprap. The 3/4-inch crushed rock simulated geometrically the 36-inch prototype riprap fairly closely. The area that was riprapped is shown in Figure 35(A) and was partially determined by noting the eroded areas evident in the erosion test in sand, Figure 33(B). The initial 2-hour test run showed no movement of riprap to occur along the end of the structure, Figure 35(B). Even on the side slopes along the 90° wing walls the riprap did not move as did the sand without the riprap protection. The downstream portion of the left bank failed, however. The cause was determined to be insufficient thickness of riprap in the model test or insufficient packing of the sand underneath because a repeat test did not disclose this failure. However, the initial test did indicate that the waves from the boil at the corners of the basin may cause riprap failure of the prototype channel banks if the thickness of the riprap layer is not sufficient. The repeat test shown in Figure 35(C) was an 8-hour test, 2 hours of which was with 1/4 maximum flow, 2 hours with 1/2, 2 hours with 3/4, and the final 2 hours with full maximum discharge. None of the riprapped areas failed in this test. Some of the riprap along the end of the structure was covered with sand but the riprap, in general, was not dislocated.

An additional 4-hour test with maximum flow continuing from where the previous 8-hour test left off finally produced a gradual failure of the riprapped area on the downstream portions of the right bank as shown in Figure 35(D). One hour, model, is equivalent to 7.3 hours, prototype. The results of these tests were considered satisfactory by the designers, therefore, this basin was recommended for prototype construction.

Interior - Reclamation - Denver, Colo.

FIGURE 1
REPORT HYD. 390





KEY MAP

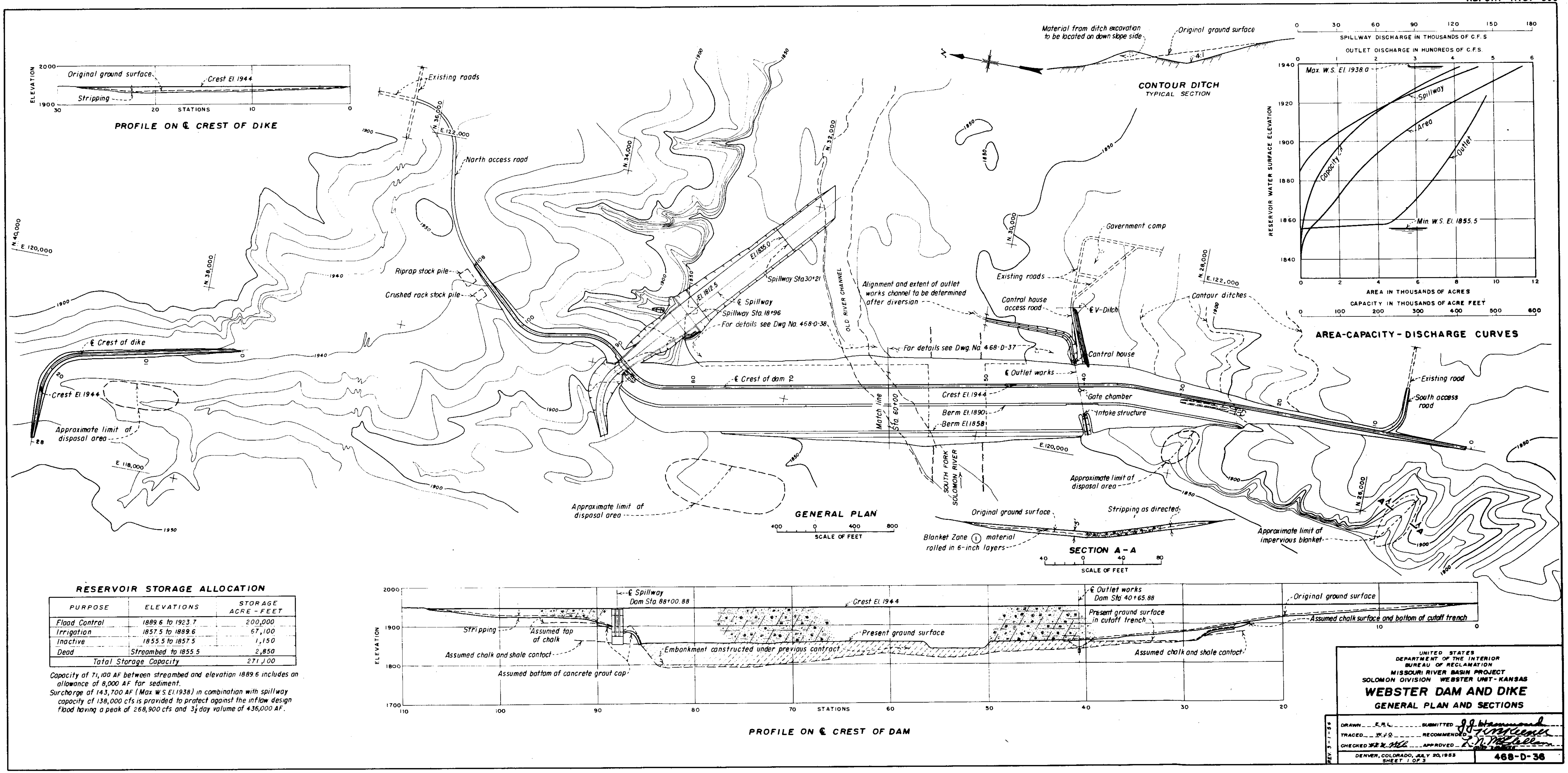
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MISSOURI RIVER BASIN PROJECT
SOLOMON DIV. - WEBSTER UNIT - KAN.

WEBSTER DAM AND DIKE

LOCATION MAP

DRAWN G.W.M.-R.V.S. SUBMITTED W. E. Collins
TRACED R.V.S. RECOMMENDED Nelson A. Bunt
CHECKED H.W.M. APPROVED A. T. Rice
ASST. CHIEF DESIGNING ENGINEER
DENVER, COLORADO, SEPT. 2, 1952
REV. 7-20-53
468-D-2

FIGURE 2
REPORT HYD. 390

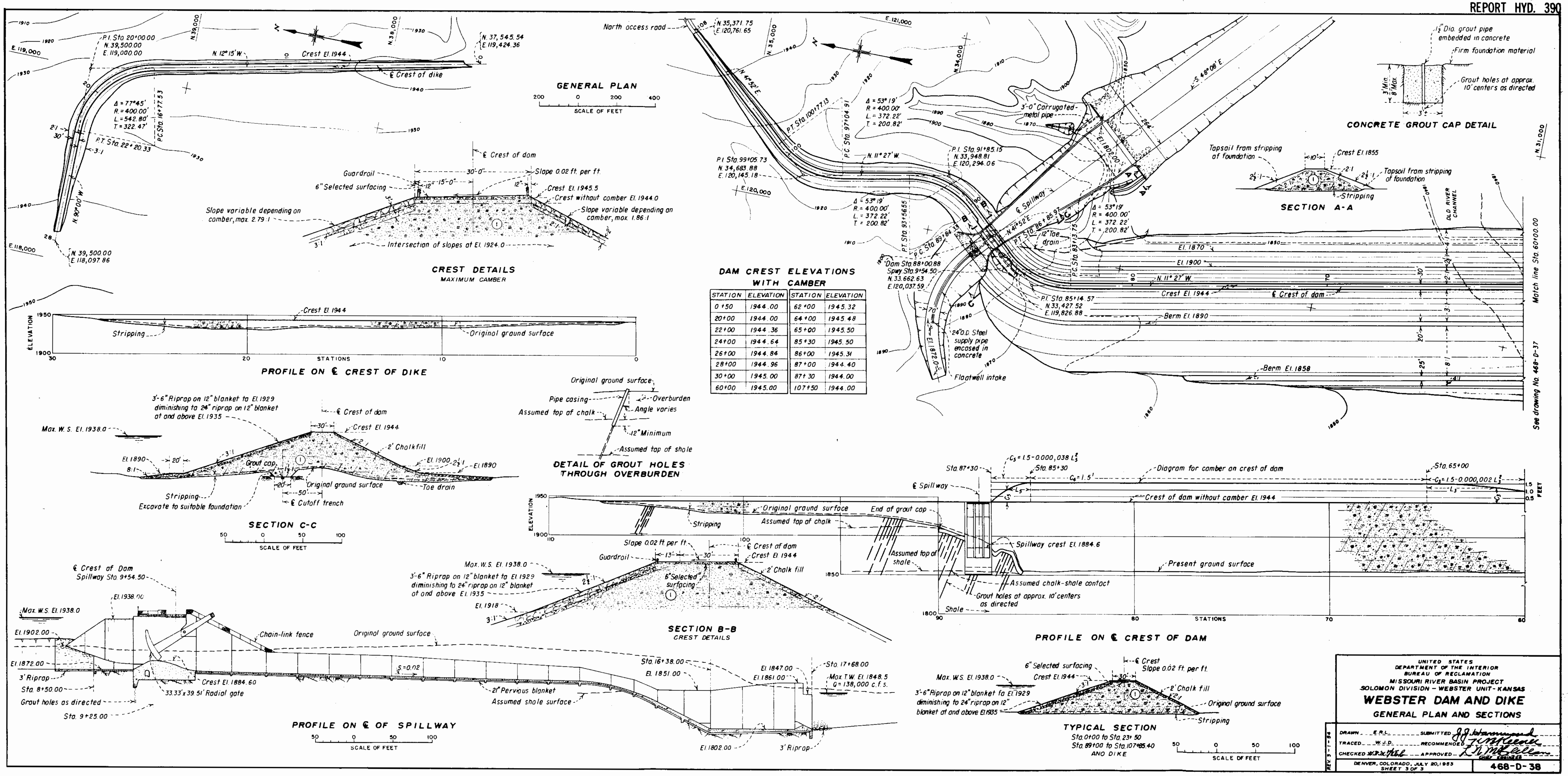


RESERVOIR STORAGE ALLOCATION

PURPOSE	ELEVATIONS	STORAGE ACRE-FEET
Flood Control	1889.6 to 1923.7	200,000
Irrigation	1857.5 to 1889.6	67,100
Inactive	1855.5 to 1857.5	1,150
Dead	Streambed to 1855.5	2,850
Total Storage Capacity		271,100

Capacity of 71,100 AF between streambed and elevation 1889.6 includes an allowance of 8,000 AF for sediment.

Surcharge of 143,700 AF (Max. W.S. El. 1938) in combination with spillway capacity of 138,000 cfs is provided to protect against the inflow design flood having a peak of 268,900 cfs and $3\frac{1}{2}$ day volume of 436,000 AF.



DAM CREST ELEVATIONS WITH CAMBER

STATION	ELEVATION	STATION	ELEVATION
0+50	1944.00	62+00	1945.32
20+00	1944.00	64+00	1945.48
22+00	1944.36	65+00	1945.50
24+00	1944.64	85+30	1945.50
26+00	1944.84	86+00	1945.31
28+00	1944.96	87+00	1944.40
30+00	1945.00	87+30	1944.00
60+00	1945.00	107+50	1944.00

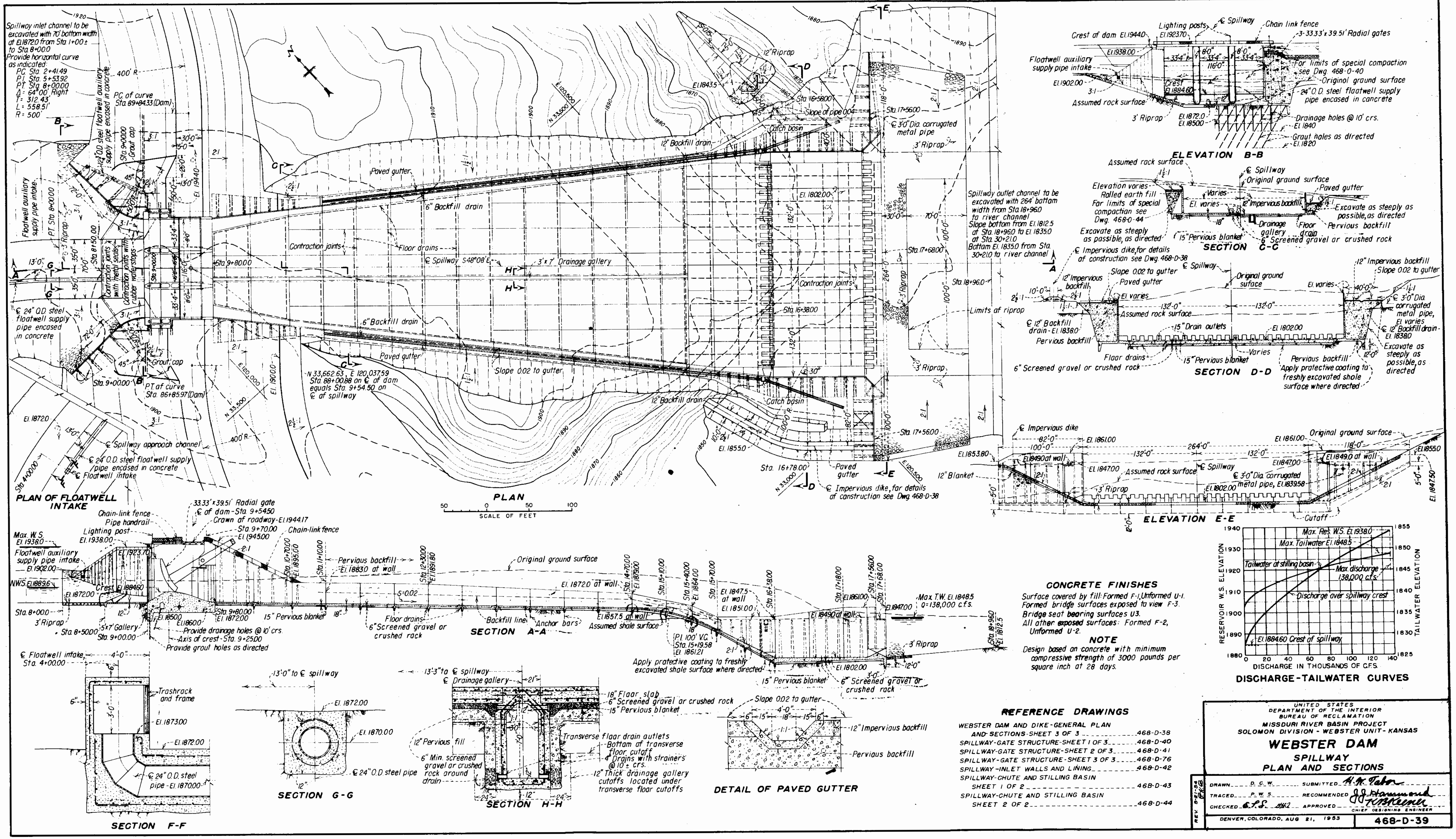

PLAN OF FLOATWELL INTAKE
PLAN
SCALE OF FEET
ELEVATION B-B
SECTION C-C
SECTION D-D
ELEVATION E-E
SECTION A-A
SECTION F-F
SECTION G-G
SECTION H-H
DETAIL OF PAVED GUTTER
CONCRETE FINISHES
Surface covered by fill: Formed F-1, Unformed U-1.
Formed bridge surfaces exposed to view F-3.
Bridge seat bearing surfaces U3.
All other exposed surfaces: Formed F-2, Unformed U-2.
NOTE
Design based on concrete with minimum compressive strength of 3000 pounds per square inch at 28 days.
DISCHARGE-TAILWATER CURVES
DISCHARGE IN THOUSANDS OF C.F.S.
RESERVOIR W.S. ELEVATION
TAILWATER ELEVATION
Max. Res. W.S. El. 1938.0
Max. Tailwater El. 1848.5
Tailwater at stilling basin
Max. discharge 138,000 c.f.s.
Discharge over spillway crest
El. 1884.60 Crest of spillway
REFERENCE DRAWINGS
WEBSTER DAM AND DIKE-GENERAL PLAN AND SECTIONS-SHEET 3 OF 3 ..... 468-D-38
SPILLWAY-GATE STRUCTURE-SHEET 1 OF 3 ..... 468-D-40
SPILLWAY-GATE STRUCTURE-SHEET 2 OF 3 ..... 468-D-41
SPILLWAY-GATE STRUCTURE-SHEET 3 OF 3 ..... 468-D-76
SPILLWAY-INLET WALLS AND LINING 468-D-42
SPILLWAY-CHUTE AND STILLING BASIN SHEET 1 OF 2 ..... 468-D-43
SPILLWAY-CHUTE AND STILLING BASIN SHEET 2 OF 2 ..... 468-D-44
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MISSOURI RIVER BASIN PROJECT
SOLOMON DIVISION - WEBSTER UNIT - KANSAS
WEBSTER DAM
SPILLWAY
PLAN AND SECTIONS
DRAWN D.C.W. SUBMITTED
TRACED P.W.S. RECOMMENDED
CHECKED APPROVED
CHIEF DESIGNING ENGINEER
DENVER, COLORADO, AUG. 21, 1953
468-D-39

FIGURE 5
REPORT HYD. 390

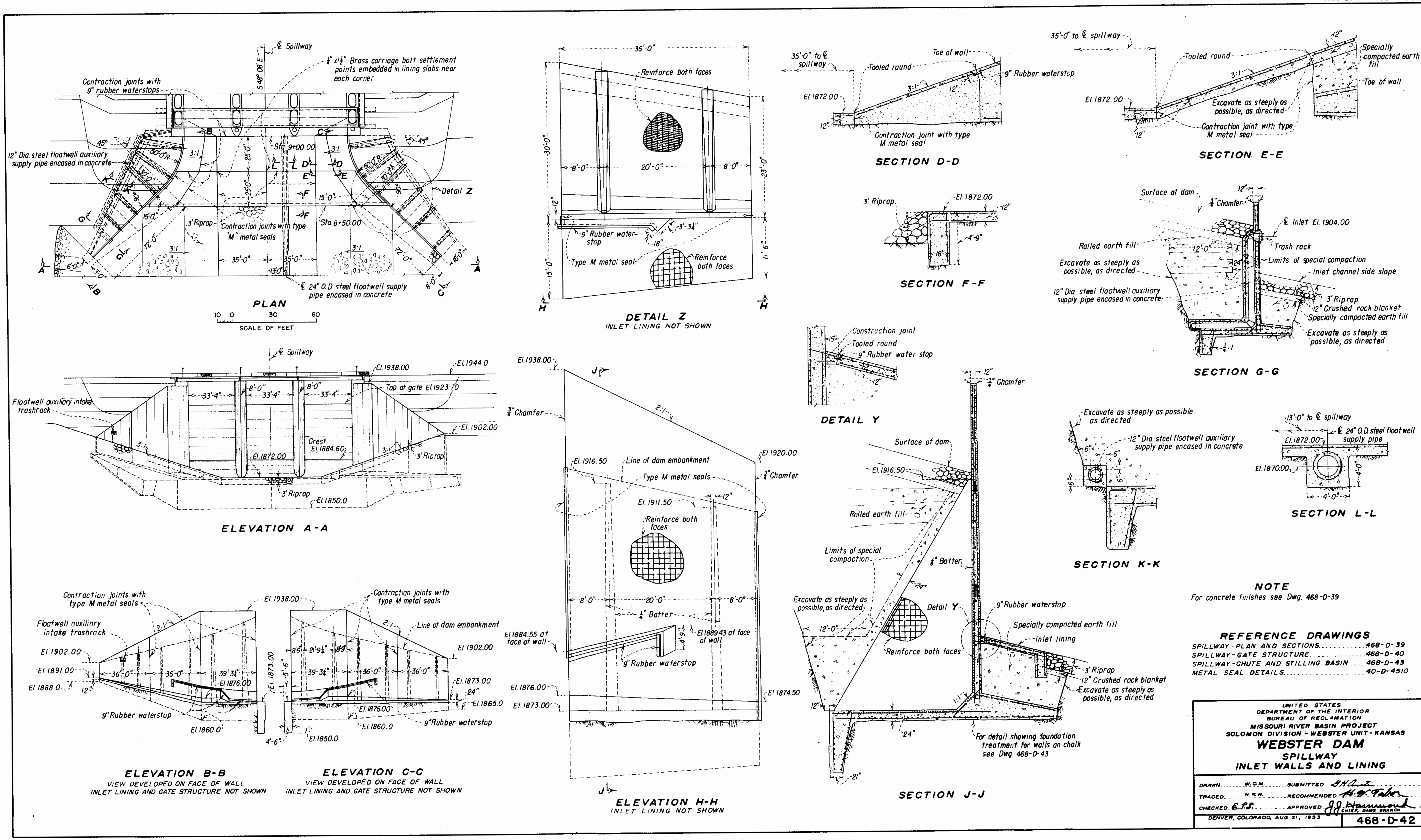
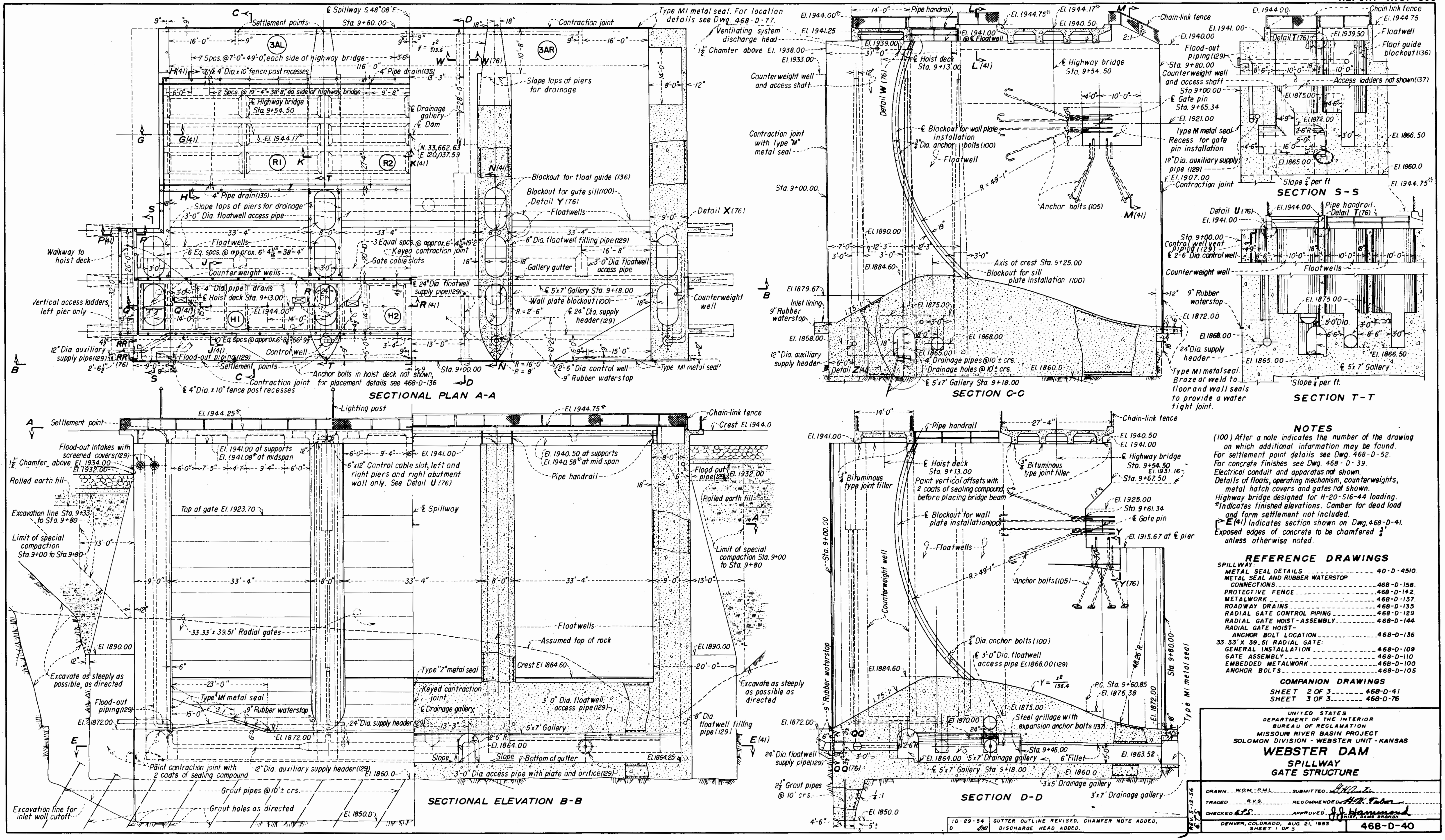


FIGURE 6
REPORT HYD. 390



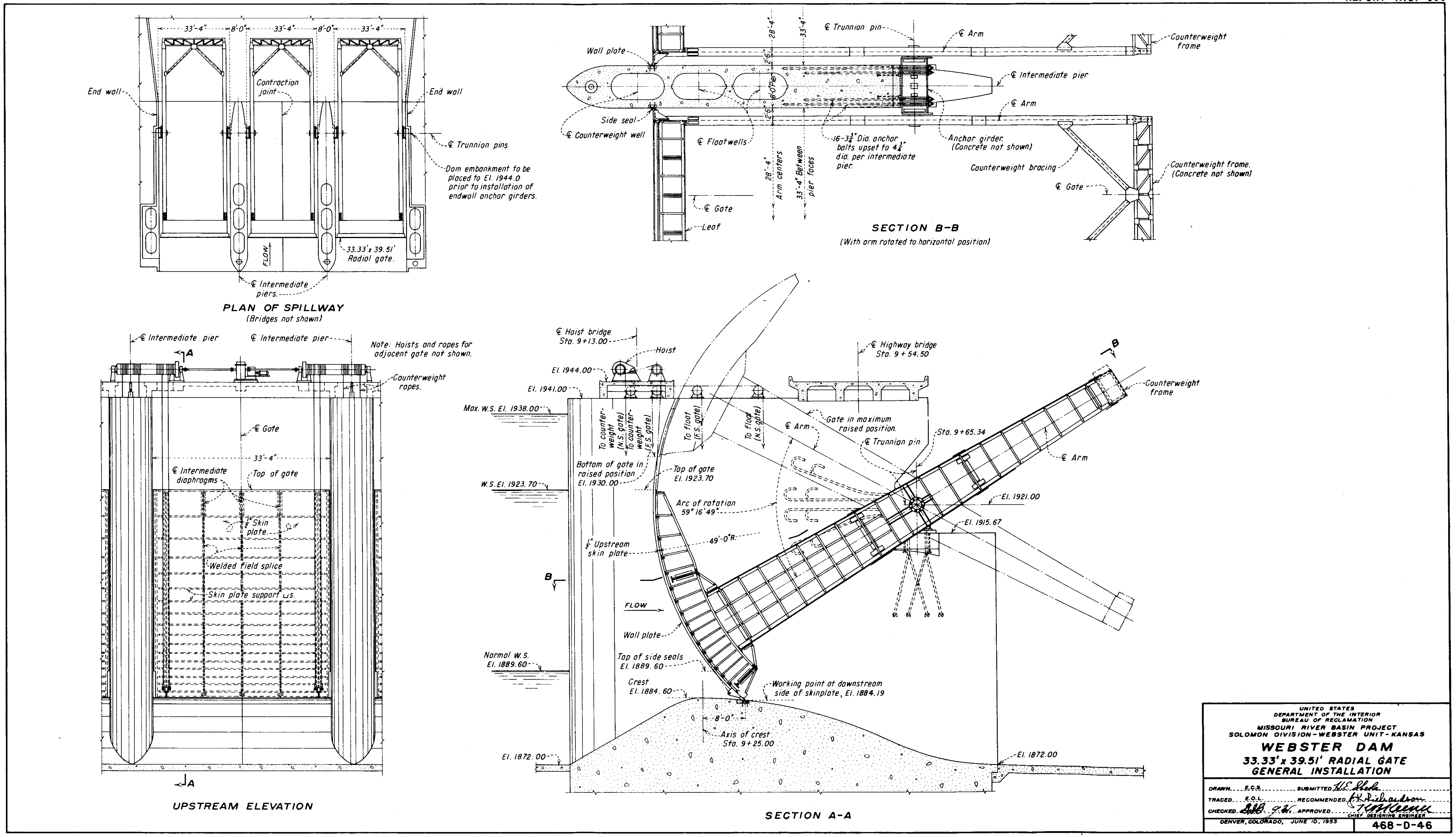
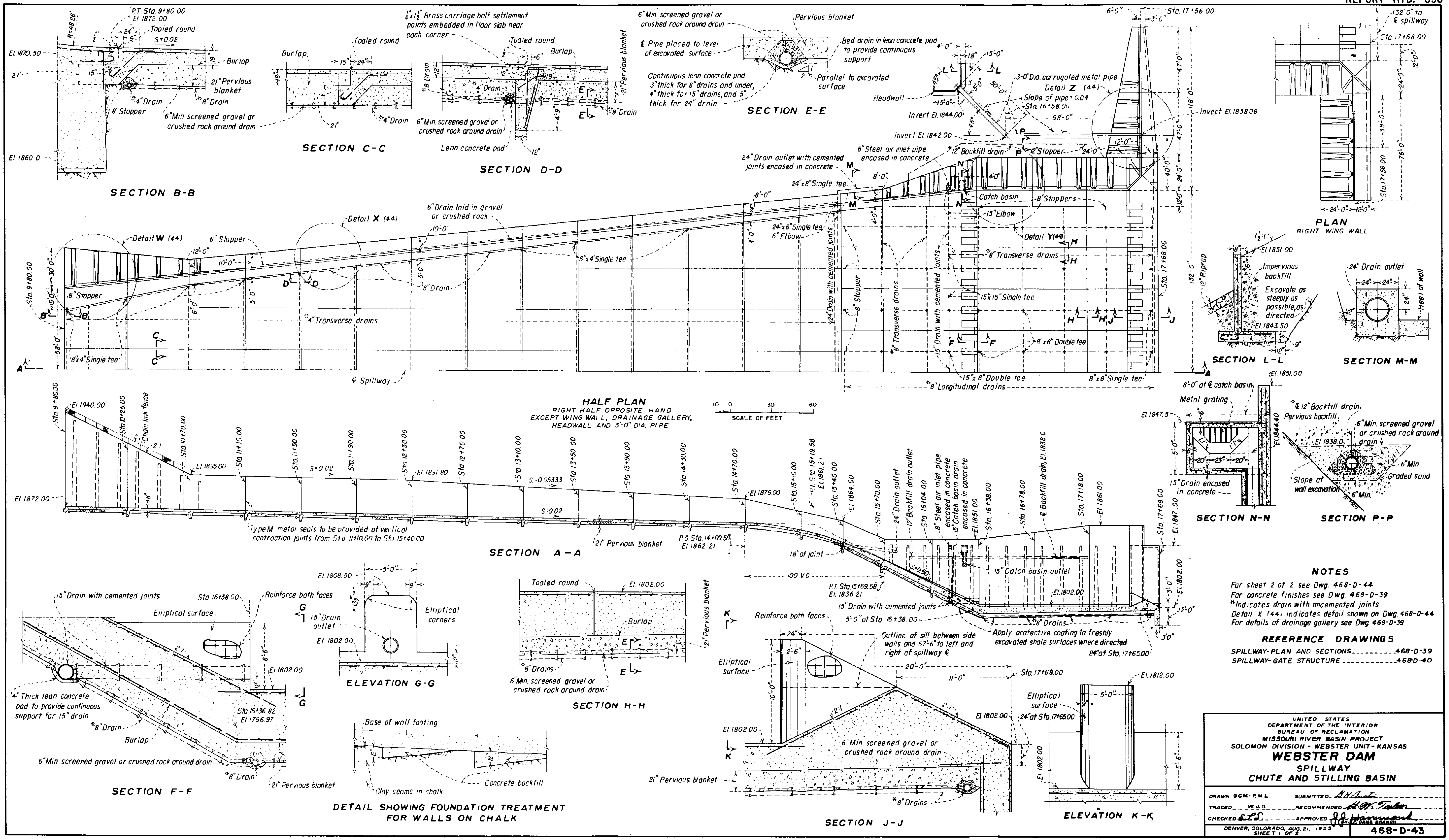
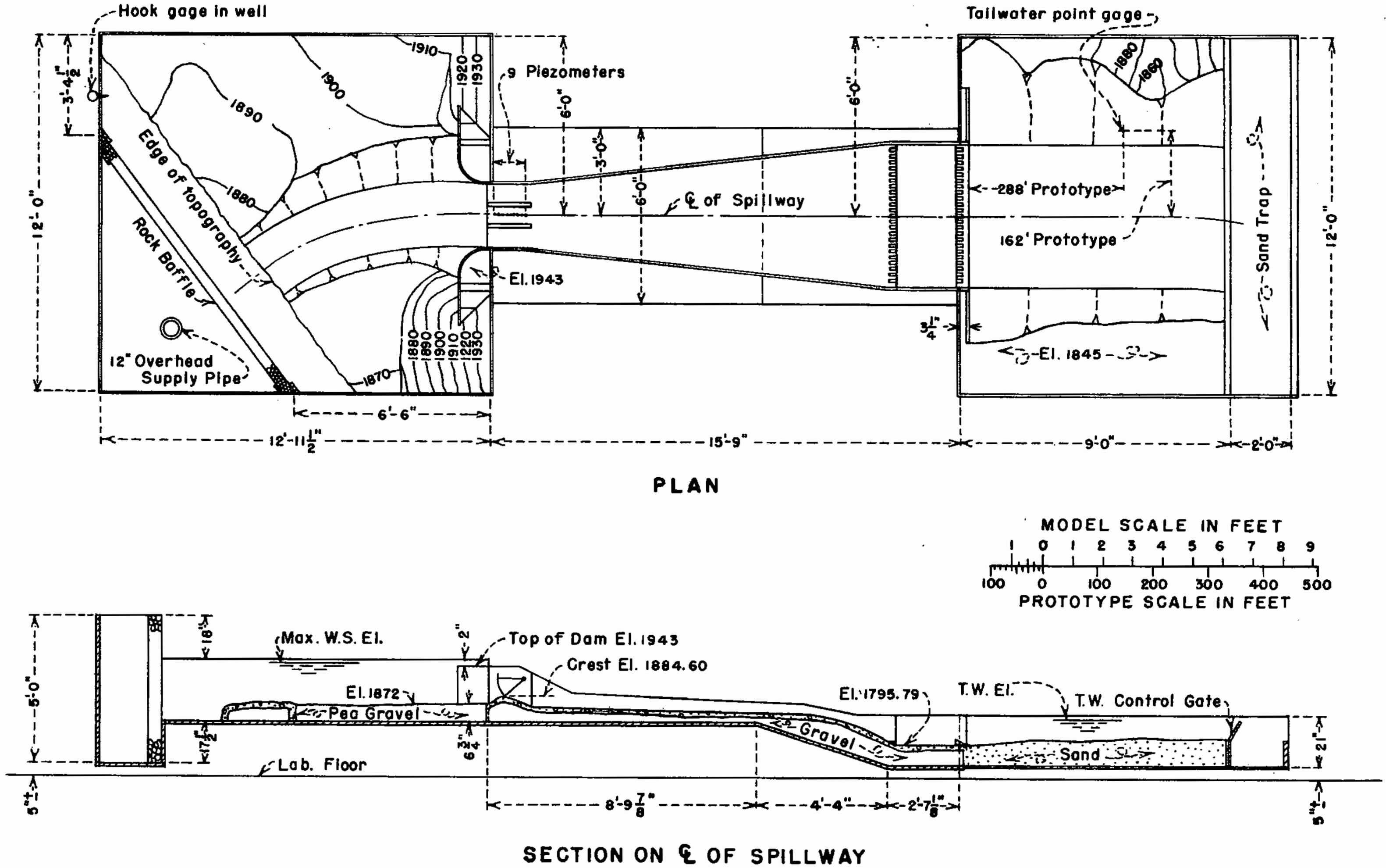

33'-4"
8'-0"
33'-4"
8'-0"
33'-4"
End wall
Contraction joint
End wall
℄ Trunnion pins
Dam embankment to be placed to El. 1944.0 prior to installation of endwall anchor girders.
FLOW
33.33' x 39.51' Radial gate.
℄ Intermediate piers.
PLAN OF SPILLWAY
(Bridges not shown)
℄ Trunnion pin
℄ Arm
Counterweight frame
Wall plate
28'-4"
33'-4"
2'-6"
6'-0" Pier
℄ Intermediate pier
℄ Arm
Side seal
℄ Counterweight well
℄ Floatwells
28'-4" Arm centers
33'-4" Between pier faces
16-3¾" Dia. anchor bolts upset to 4½" dia. per intermediate pier.
Anchor girder. (Concrete not shown)
Counterweight bracing
Counterweight frame. (Concrete not shown)
℄ Gate
℄ Gate
Leaf
SECTION B-B
(With arm rotated to horizontal position)
℄ Intermediate pier
℄ Intermediate pier
A
Note: Hoists and ropes for adjacent gate not shown.
Counterweight ropes.
℄ Gate
33'-4"
℄ Intermediate diaphragms
Top of gate
½" Skin plate.
Welded field splice
Skin plate support ⊔s.
A
UPSTREAM ELEVATION
℄ Hoist bridge Sta. 9+13.00
Hoist
El. 1944.00
El. 1941.00
Max. W.S. El. 1938.00
To counterweight (N.S. gate)
To counterweight (F.S. gate)
To float (F.S. gate)
To float (N.S. gate)
℄ Highway bridge Sta. 9+54.50
℄ Arm
Gate in maximum raised position.
Sta. 9+65.34
℄ Trunnion pin
Counterweight frame
B
℄ Arm
Bottom of gate in raised position El. 1930.00
Top of gate El. 1923.70
W.S. El. 1923.70
Arc of rotation 59° 16' 49"
El. 1921.00
El. 1915.67
½" Upstream skin plate
49'-0" R.
B
FLOW
Wall plate
Normal W.S. El. 1889.60
Top of side seals El. 1889.60
Crest El. 1884.60
Working point at downstream side of skinplate, El. 1884.19
8'-0"
Axis of crest Sta. 9+25.00
El. 1872.00
El. 1872.00
SECTION A-A
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MISSOURI RIVER BASIN PROJECT
SOLOMON DIVISION-WEBSTER UNIT-KANSAS
WEBSTER DAM
33.33' x 39.51' RADIAL GATE
GENERAL INSTALLATION
DRAWN E.C.S.
SUBMITTED
TRACED E.O.L.
RECOMMENDED
CHECKED
APPROVED
CHIEF DESIGNING ENGINEER
DENVER, COLORADO, JUNE 10, 1953
468-D-46

FIGURE 8
REPORT HYD. 390





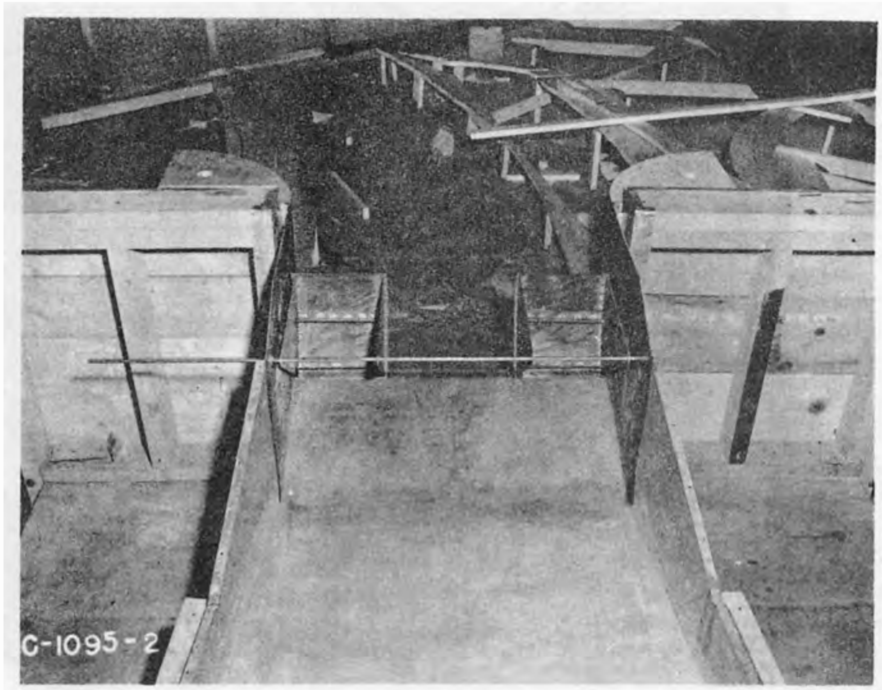
WEBSTER DAM SPILLWAY

MODEL LAYOUT

1:54 SCALE MODEL

FIGURE 9

REPORT HYD. 390

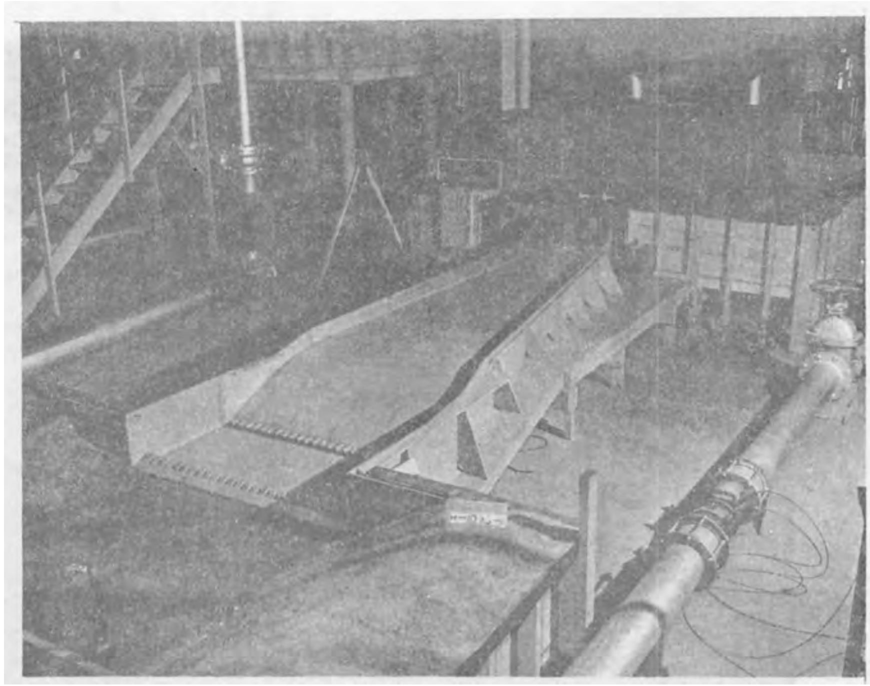


A. Note model gates and wood contours

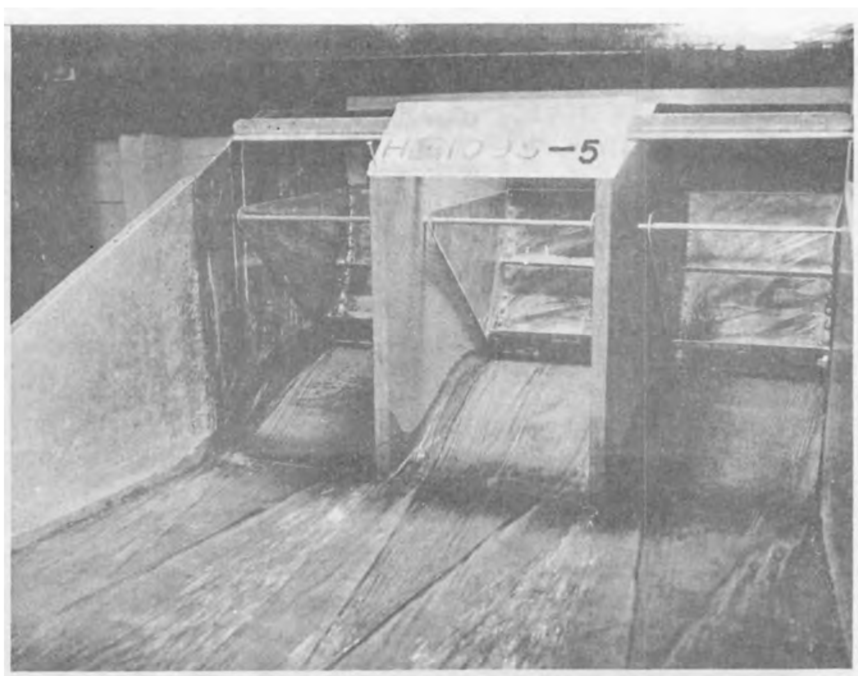
B. Note wood contours being covered with metal lath

WEBSTER DAM SPILLWAY
Model Construction
1:54 SCALE MODEL

FIGURE 11
Report Hyd-390

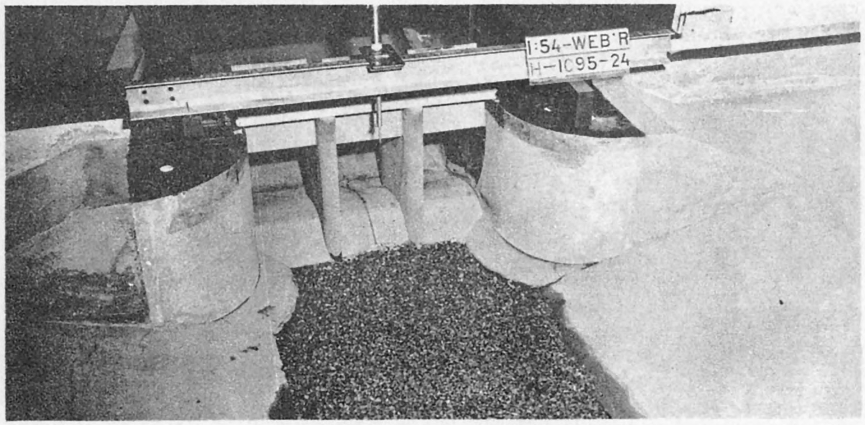


A. General overall view

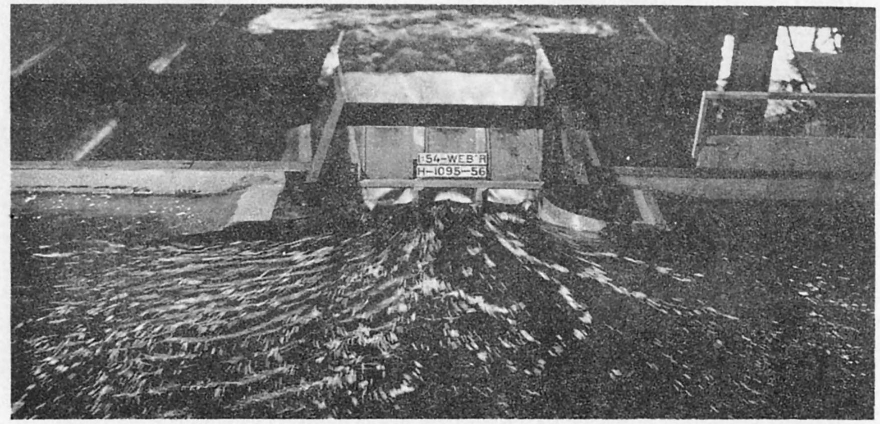


B. Gate opening 3 feet--Discharge 11,000 second-feet--Recommended gate pin location. Note flow pattern

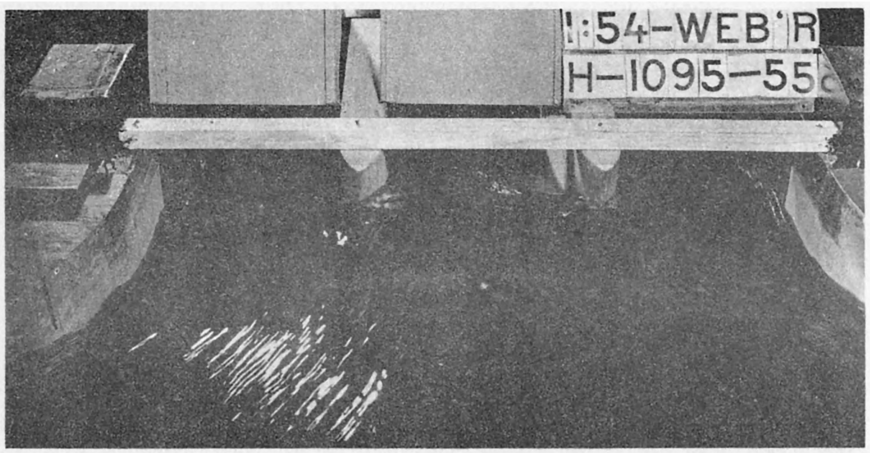
WEBSTER DAM SPILLWAY
Model Views
1:54 SCALE MODEL



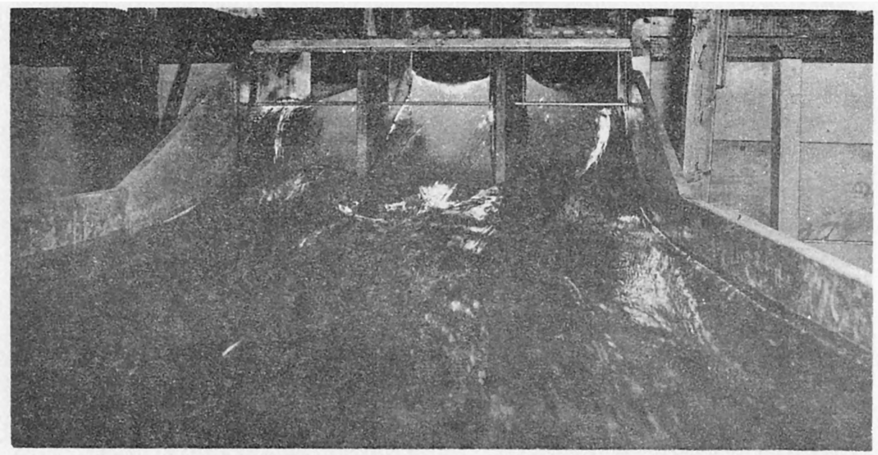
A. Preliminary inlet walls and water surface point gage



B. Flow approaching the gate section. Note smooth flow along inlet walls

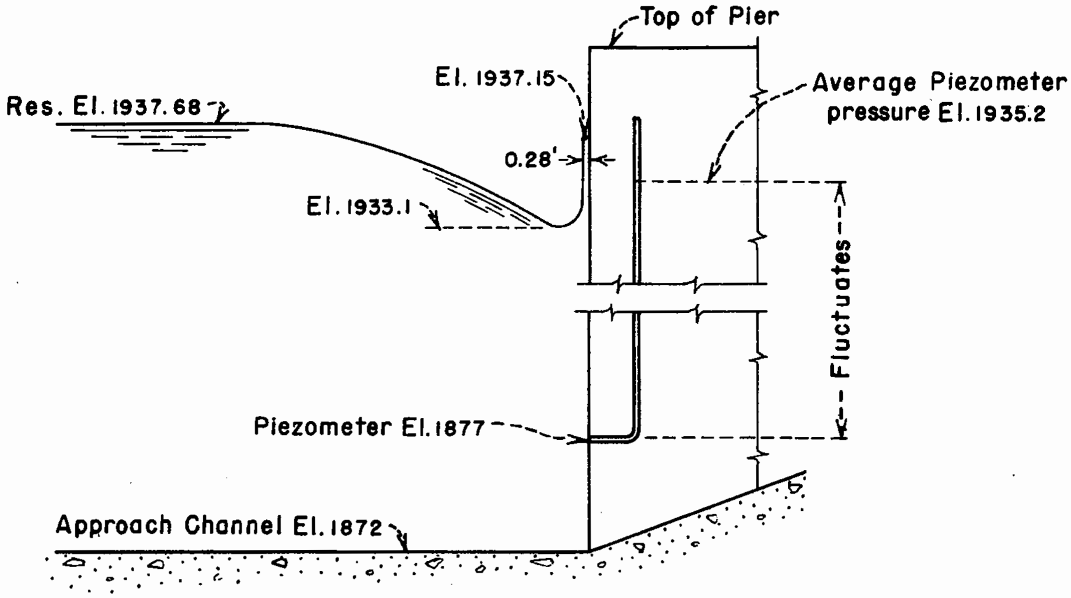


C. Flow entering gate section

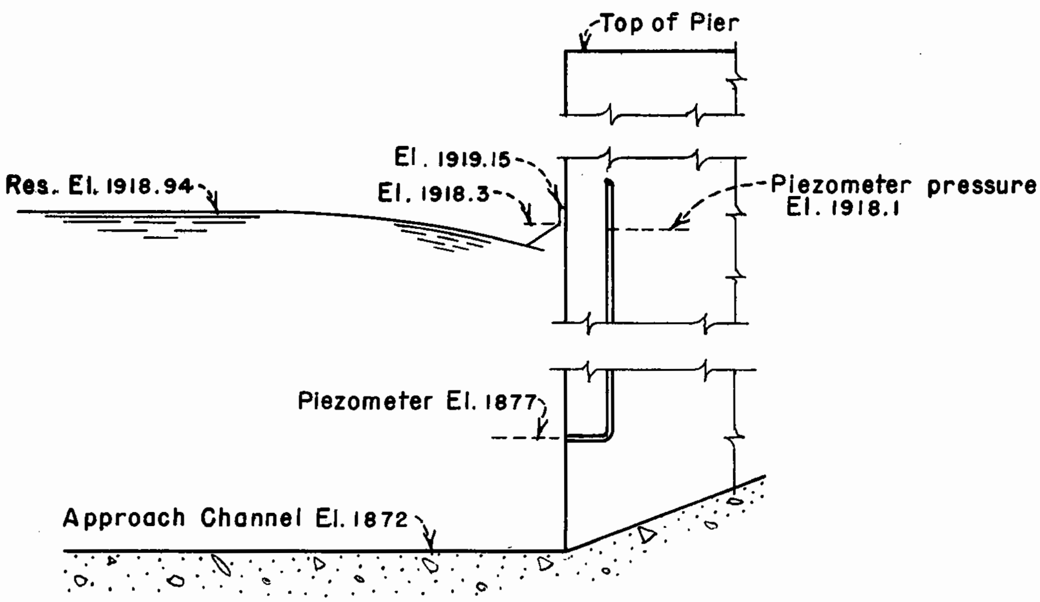


D. Flow leaving gate section

WEBSTER DAM SPILLWAY
Flow Through the Preliminary Spillway Entrance--136,000 Second-feet
1:54 SCALE MODEL



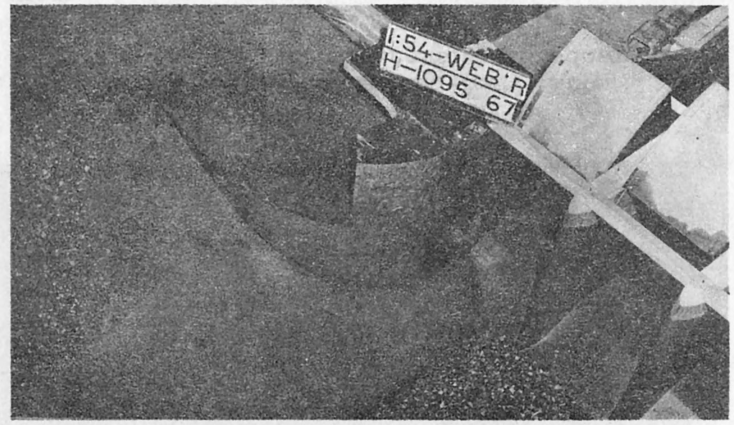
136,000 SECOND FEET



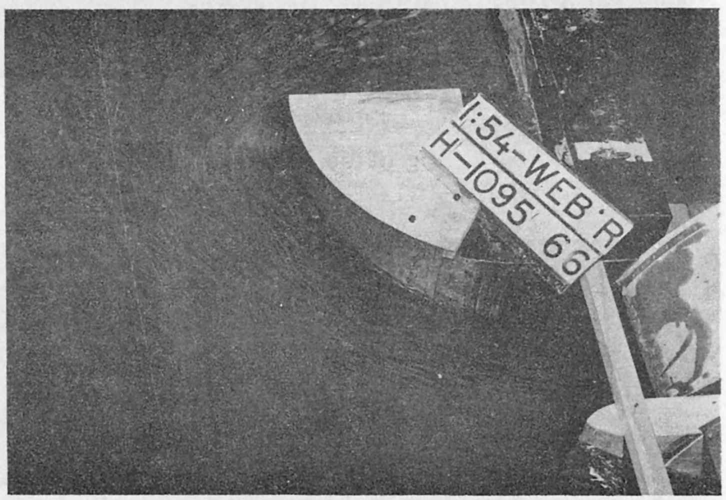
68,000 SECOND FEET

WEBSTER DAM SPILLWAY
WATER SURFACE AND PRESSURE AT RIGHT PIER
1 : 54 SCALE MODEL

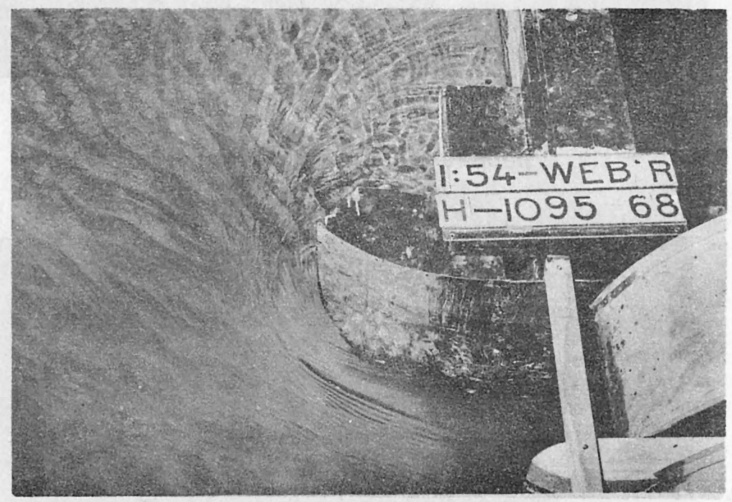
A. Design 1



C. Design 2



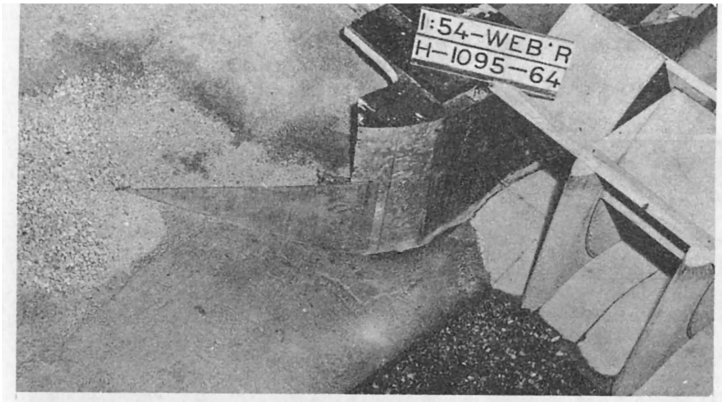
B. Design 1--136,000 second-feet. Note absence of drawdown



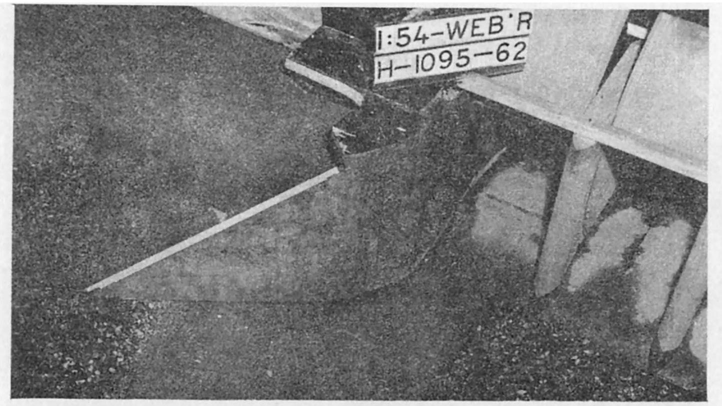
D. Design 2--136,000 second-feet. Note excessive drawdown

Sheet 1 of 2

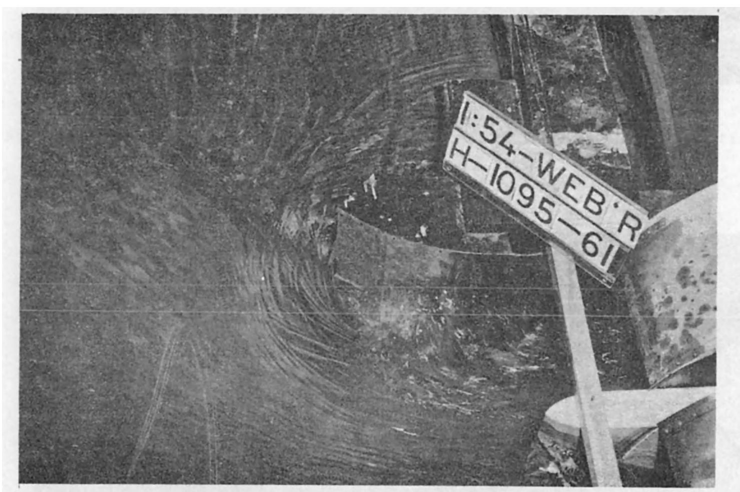
WEBSTER DAM SPILLWAY
Proposed Inlet Walls Tested
1:54 SCALE MODEL



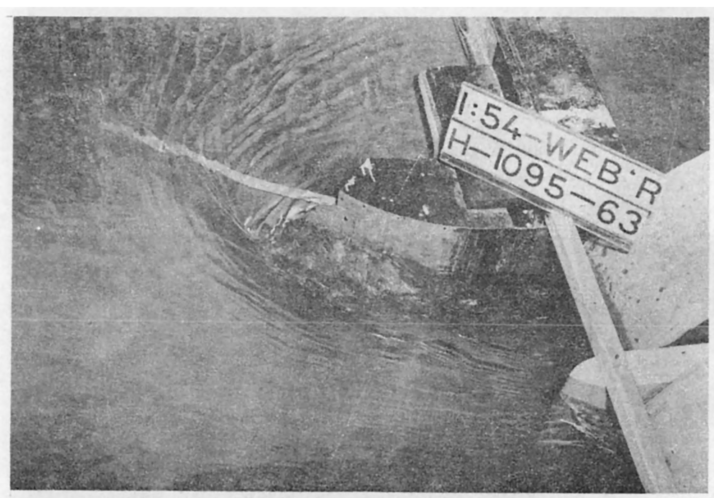
A. Design 3



C. Design 4--Recommended

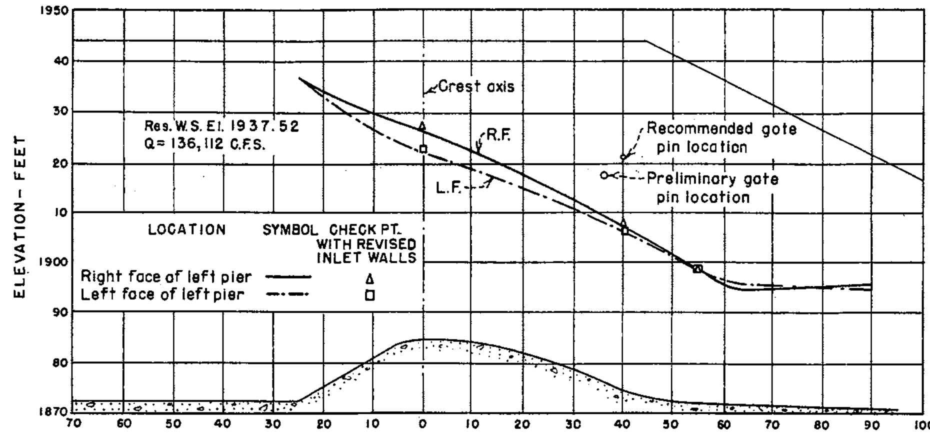


B. Design 3--136,000 second-feet. Note disturbance and drawdown

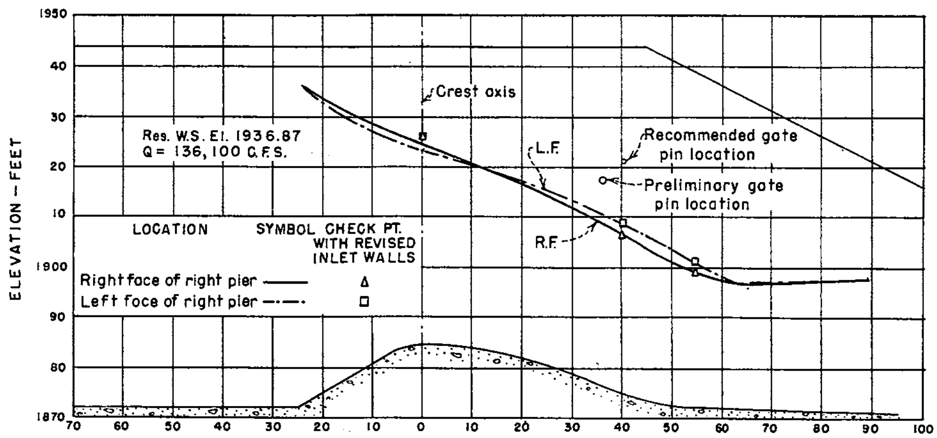


D. Design 4--136,000 second-feet. Note disturbance but not excessive drawdown

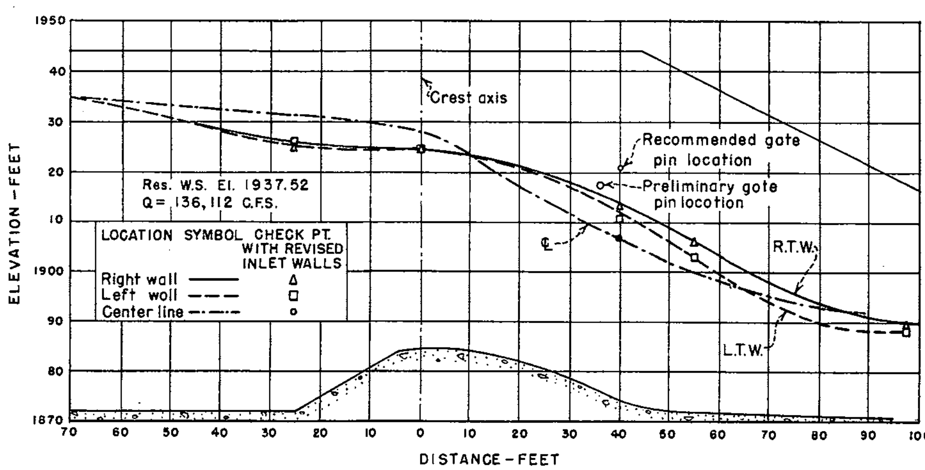
WEBSTER DAM SPILLWAY
Proposed Inlet Walls Tested
1:54 SCALE MODEL



LEFT PIER



RIGHT PIER

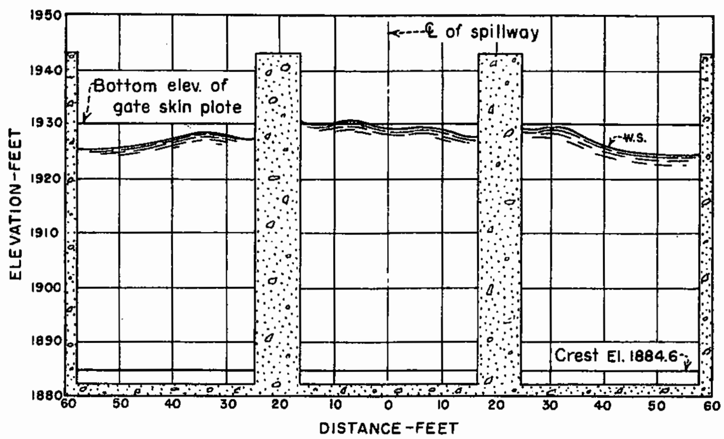


TRAINING WALLS AND CENTER LINE OF SPILLWAY

WEBSTER DAM SPILLWAY

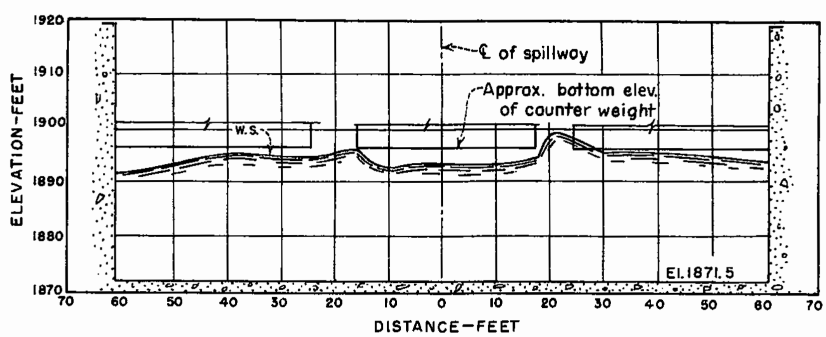
WATER SURFACE PROFILES THROUGH GATE SECTION

1:54 SCALE MODEL



NOTE: When bottom of skin plate on all three gates is set at elevation 1926, the water surface first touches the center gate when the discharge reaches 115,500 second feet. The center gate opening is completely submerged when the discharge reaches 119,400 second feet.

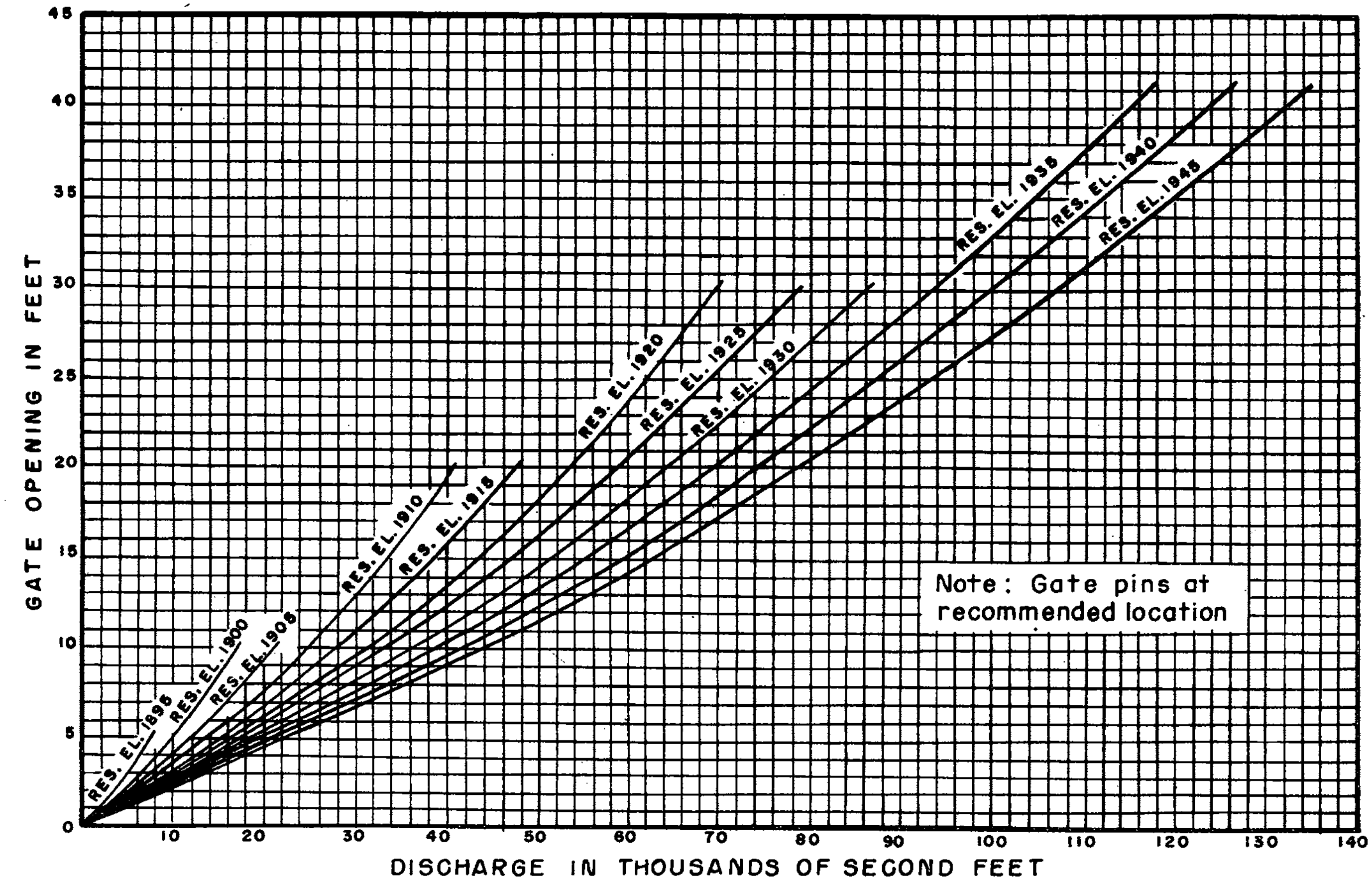
A. WATER SURFACE PROFILE UNDER BOTTOM OF GATE SKIN PLATE
APPROXIMATELY STA. 9+17 LOOKING DOWNSTREAM



NOTE: Water surface profile at Sta 10+02 is nearly one foot higher than at Sta. 10+04

B. WATER SURFACE PROFILE UNDER GATE COUNTER WEIGHT
APPROXIMATELY STA. 10+04 LOOKING DOWNSTREAM

WEBSTER DAM SPILLWAY
WATER SURFACE PROFILES UNDER GATE
SKIN PLATE AND COUNTERWEIGHT
FOR 136,000 SECOND FEET AND MAXIMUM DESIGN GATE OPENING
1:54 SCALE MODEL

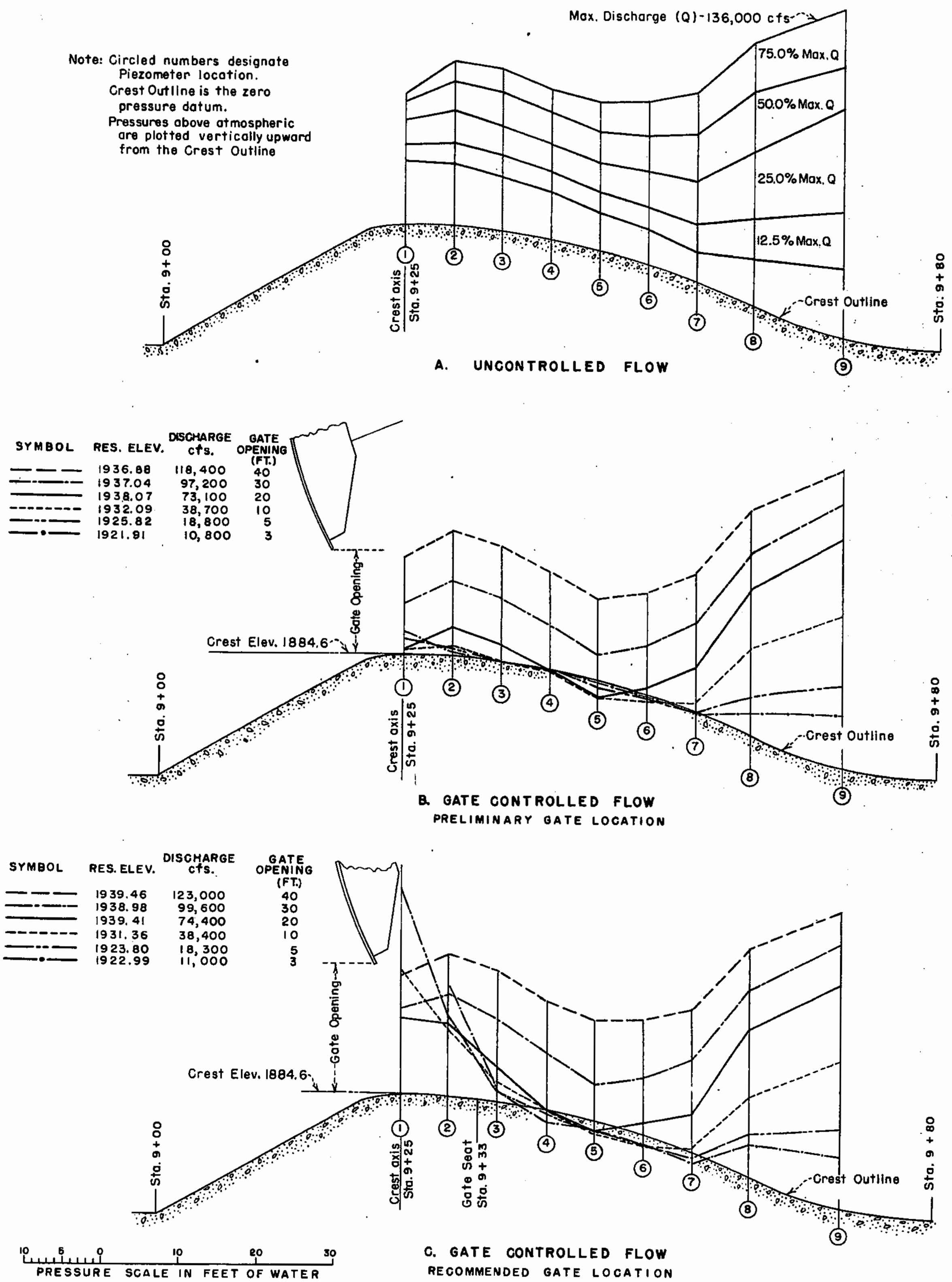


WEBSTER DAM SPILLWAY

GATE OPENING VS DISCHARGE
FOR A RANGE RESERVOIR ELEVATIONS

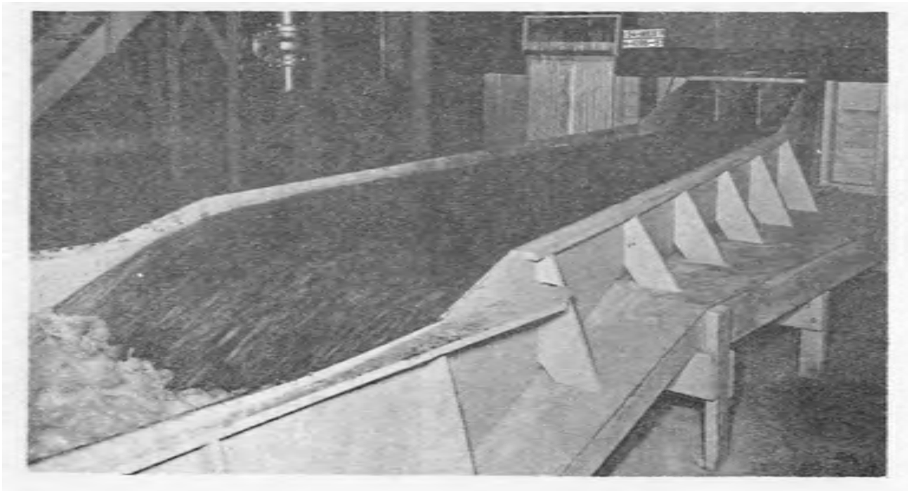
1 : 54 SCALE MODEL

FIGURE 19
REPORT HYD. 390



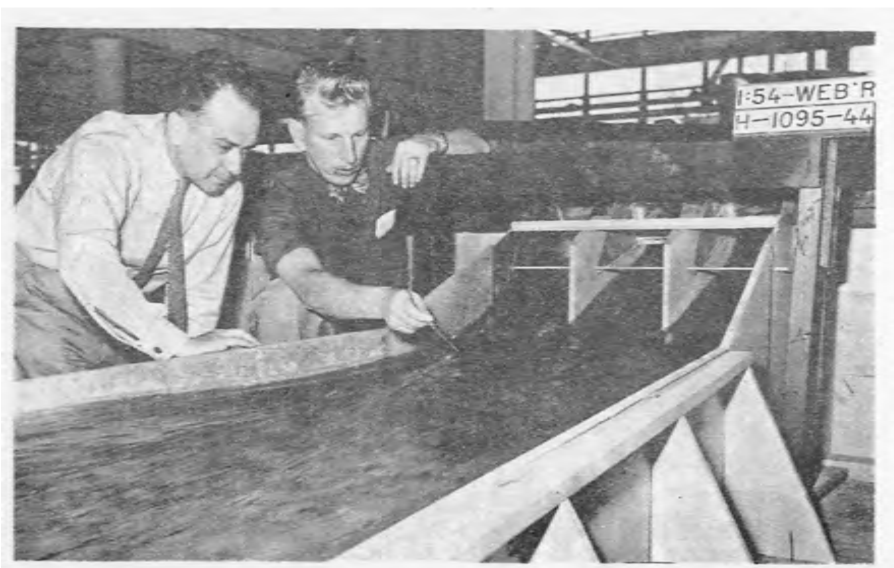
WEBSTER DAM SPILLWAY
CREST PRESSURES
1:54 SCALE MODEL

FILE NO. 476

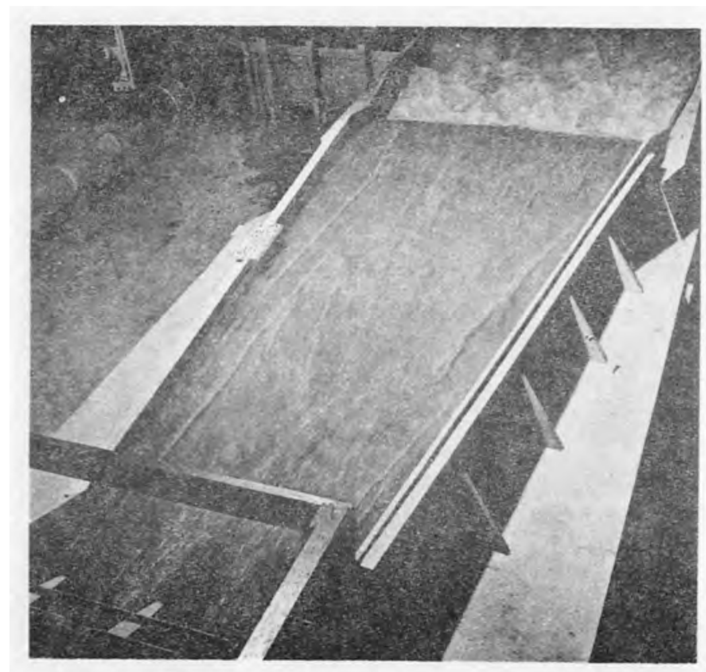


A. Good distribution of flow in chute

B. Flow entering basin is evenly distributed



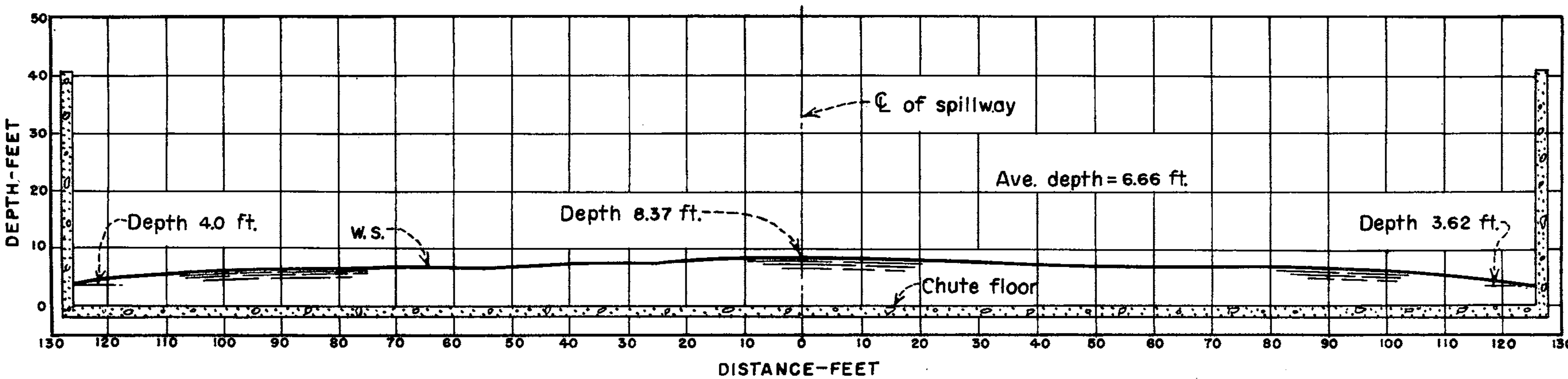
C. Note standing wave



D. Note standing waves

WEBSTER DAM SPILLWAY
Flow Through the Chute--136,000 Second-feet
1:54 SCALE MODEL

FIGURE 21
Report Hyd-390



NOTE: Depth is measured normal to the chute floor

LOOKING DOWNSTREAM AT APPROXIMATELY STA. 16+22

WEBSTER DAM SPILLWAY

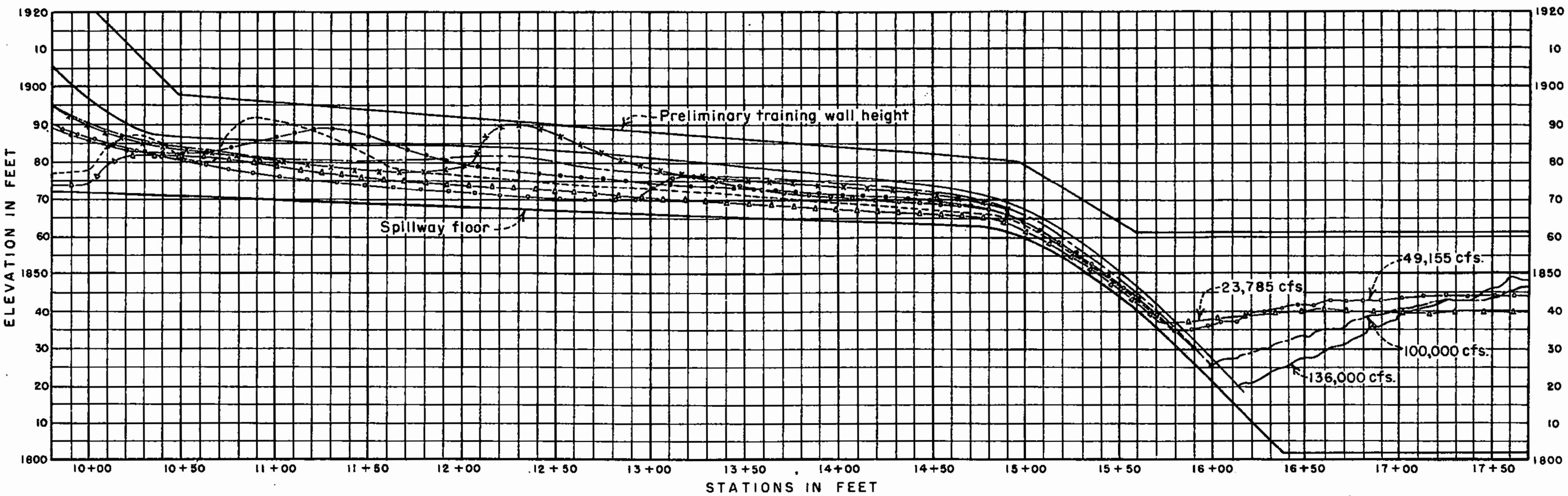
WATER SURFACE PROFILE AT ENTRANCE TO STILLING BASIN FOR 136,000 SECOND FEET

1:54 SCALE MODEL

FIGURE 22
REPORT HYD. 390

476

FIGURE 23
REPORT HYD. 390



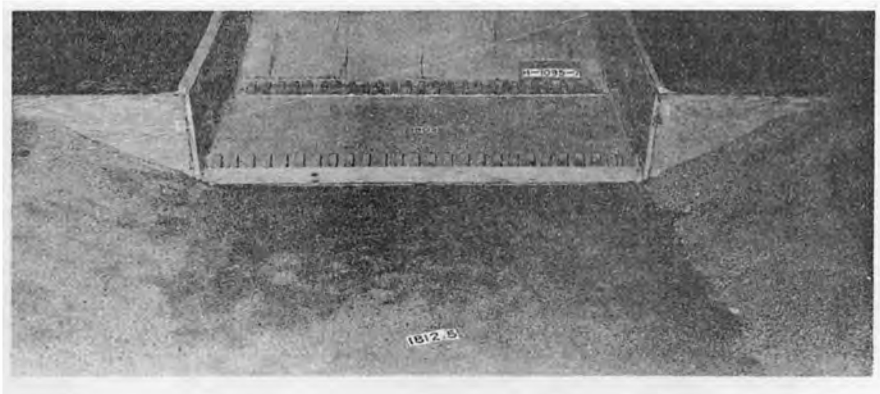
SYMBOL	DISCHARGE	GATE SETTING	RES. ELEV.
———————	136,000 cfs.	All gates opened.	
—— - ——	100,000 cfs.	All gates opened.	
—x—x—x—	100,000 cfs.	Side gates opened, center gate partially closed.	1931.74
—•—•—•—	100,000 cfs.	Side gates partially closed, center gate opened.	1937.52
—o—o—o—	49,155 cfs.	Side gates opened, center gate closed.	1920.02
- - - - - - -	49,155 cfs.	Side gates partially closed, center gate opened.	1928.50
—Δ—Δ—Δ—	23,785 cfs.	Side gates closed, center gate opened.	1920.02

NOTE: In each test the side gate openings were identical.

WEBSTER DAM SPILLWAY

WATER SURFACE PROFILES ALONG LEFT TRAINING WALL OF SPILLWAY CHUTE

1:54 SCALE MODEL



A. Discharge channel prepared for erosion test. Note higher bank on the right

B. Erosion test in progress

C. Scour pattern after 30 minute model erosion test. Note erosion at corners of basin and along 90° wing walls

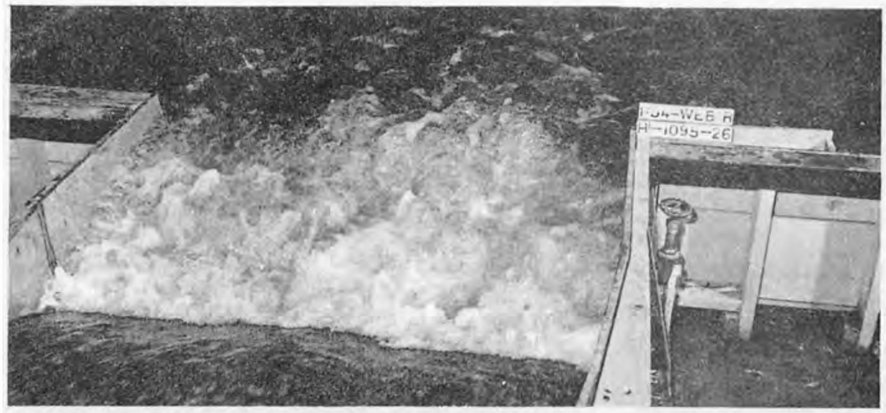
WEBSTER DAM SPILLWAY
Preliminary Stilling Basin Erosion Test-136,000 Second-feet
1:54 SCALE MODEL

A. Normal tail water elevation

B. Tail water elevation 2 feet below normal

C. Tail water elevation 4 feet below normal

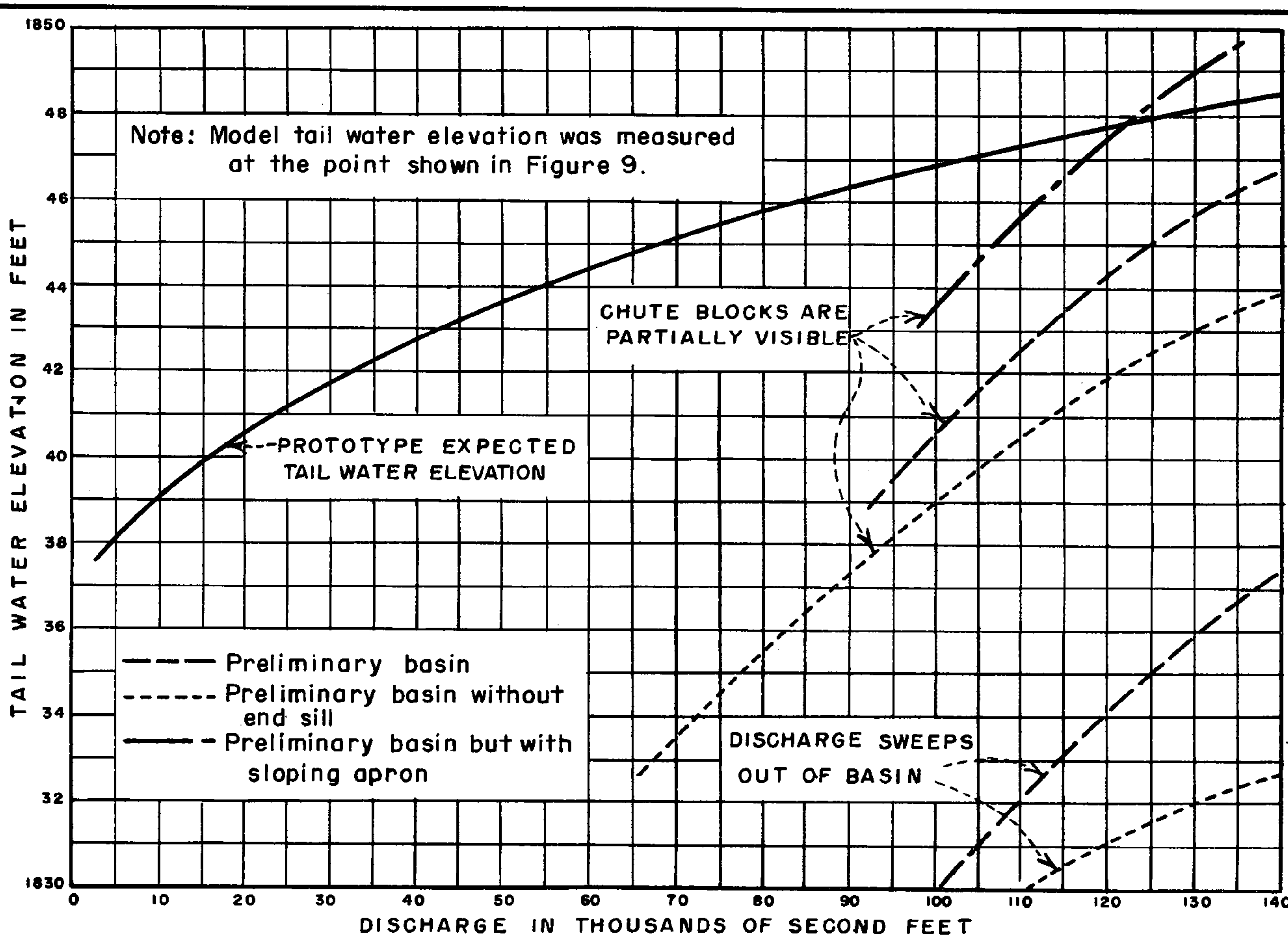
WEBSTER DAM SPILLWAY
Preliminary Basin Discharging 136,000 Second-feet
1:54 SCALE MODEL



A. Preliminary basin. Normal tail water

B. Preliminary basin-tail water elev. 4 feet below normal

WEBSTER DAM SPILLWAY
Preliminary Basin Discharging 136,000
Second-feet
1:54 SCALE MODEL

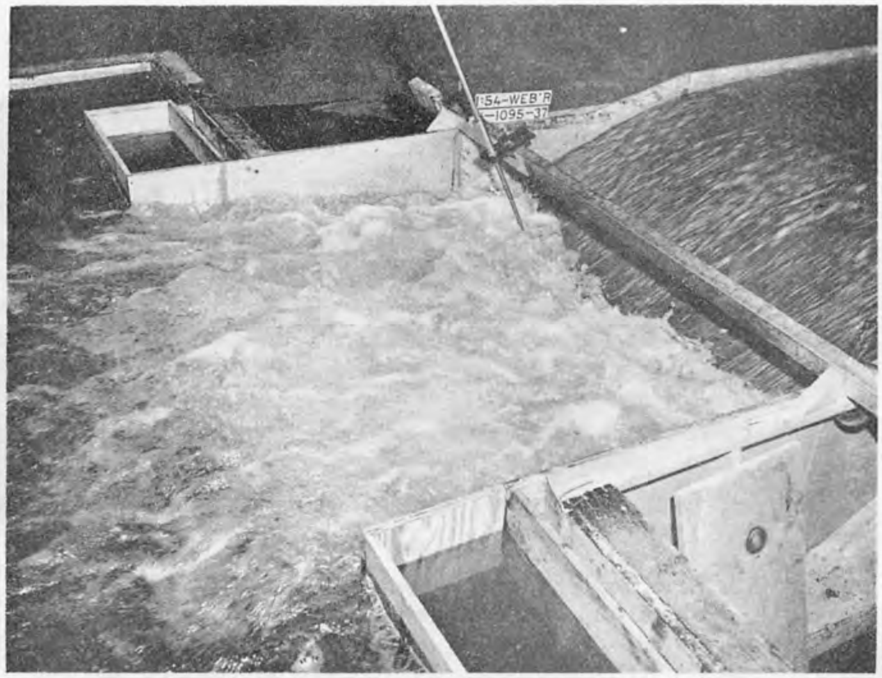


WEBSTER DAM SPILLWAY

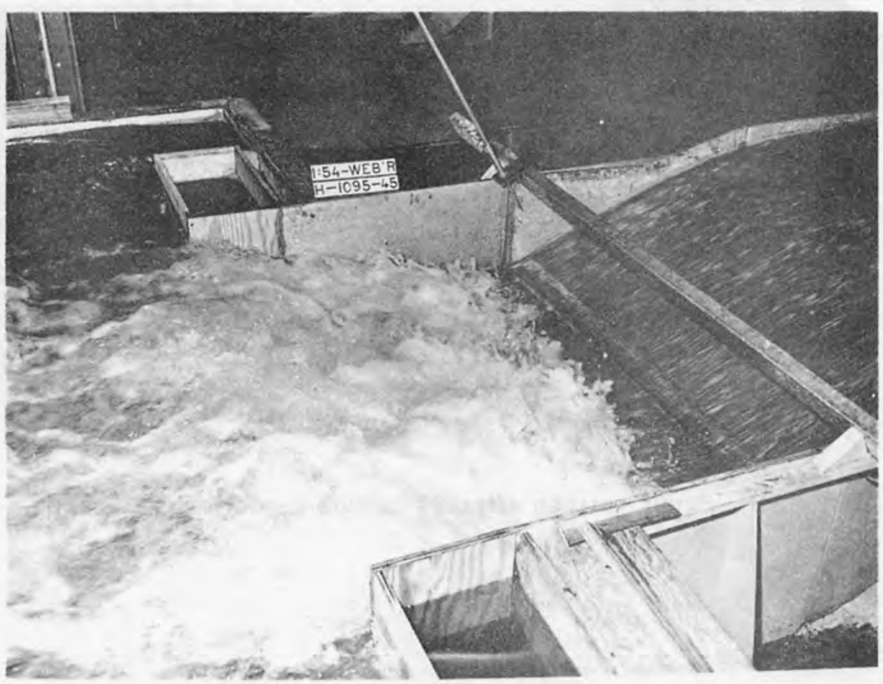
HYDRAULIC JUMP SWEEPOUT CURVES

1:54 SCALE MODEL

FIGURE 27
REPORT HYD. 390



A. With chute blocks



B. Without chute blocks

Note: Tail water depth is the same for A & B

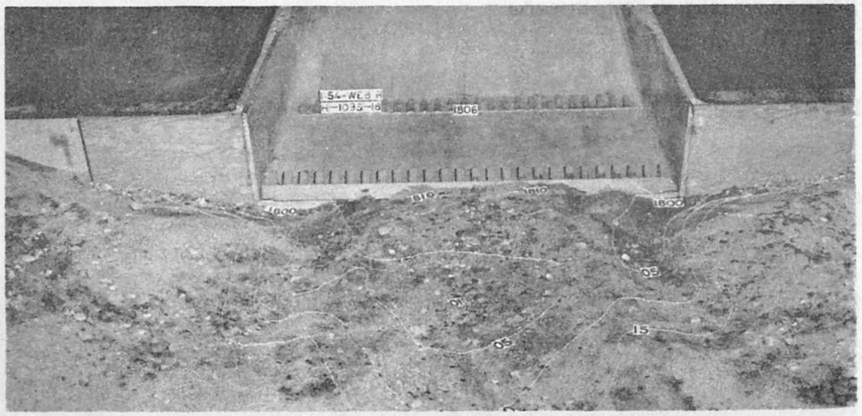
WEBSTER DAM SPILLWAY
Stilling Basin With and Without Chute Blocks--136,000 Second-feet
1:54 SCALE MODEL

FIGURE 29
Report Hyd-390

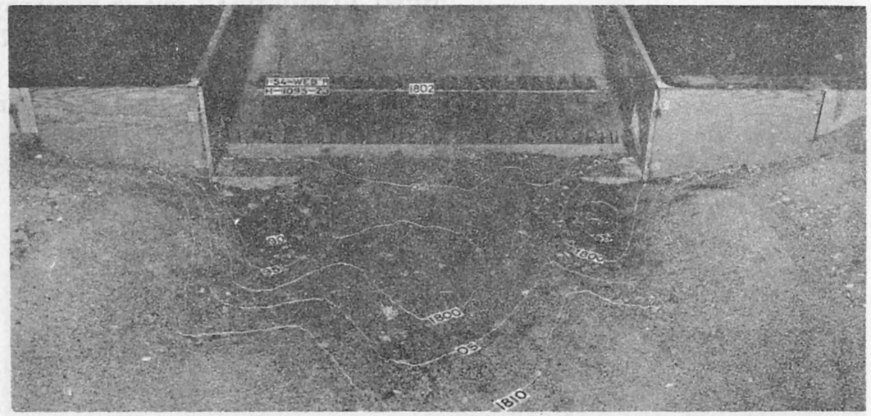
A. Erosion test in progress, normal tail water water elevation. Discharge 136,000 second-feet

B. Scour pattern after 30 minute model erosion test

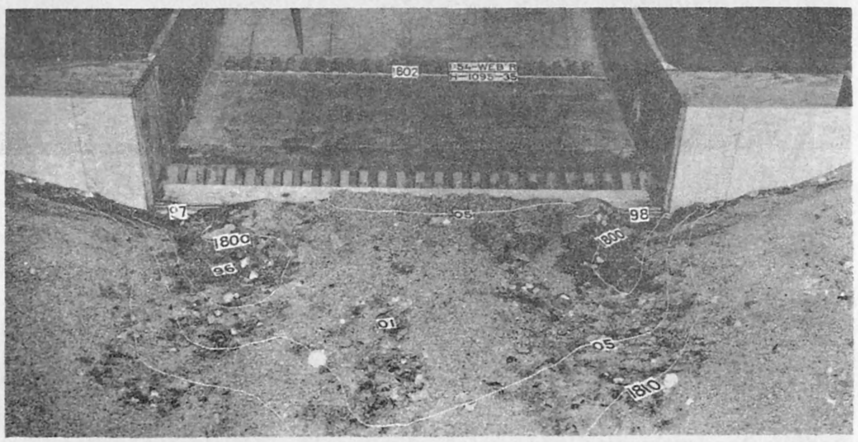
WEBSTER DAM SPILLWAY
Preliminary Basin Without End Sill
1:54 SCALE MODEL



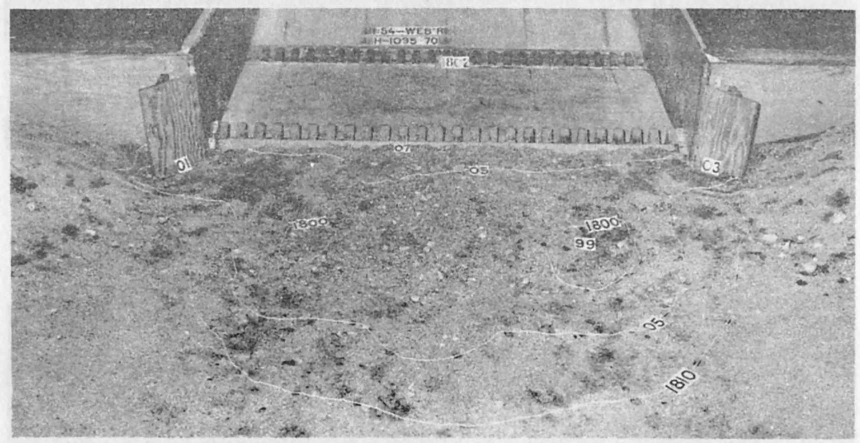
A. Basin apron elevated 4 feet



B. End sill moved upstream 22 feet



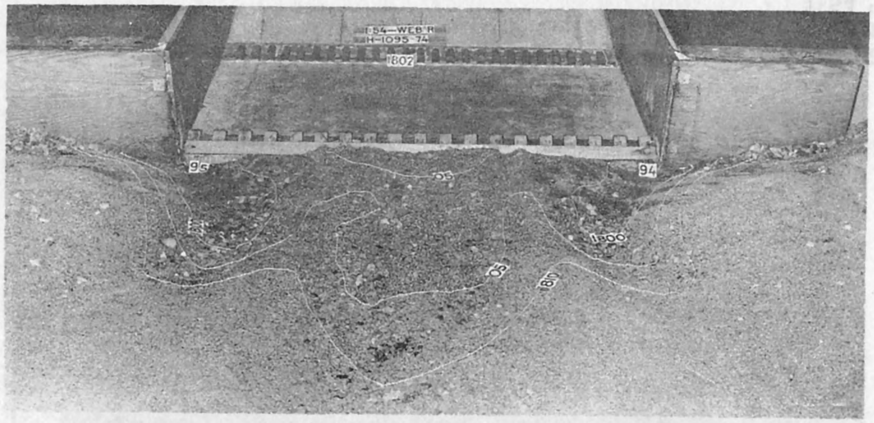
C. Basin lengthened 45 feet



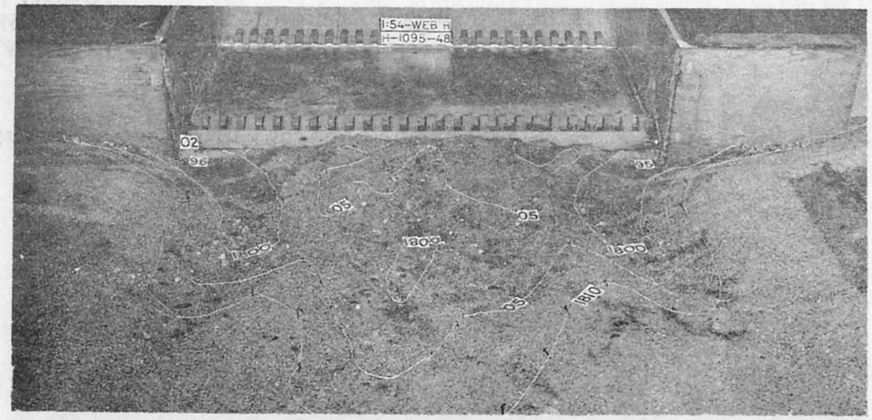
D. 45° spur walls added at basin corners

Scour Patterns After 30 Minute Model Erosion Tests With 136,000 Second-feet

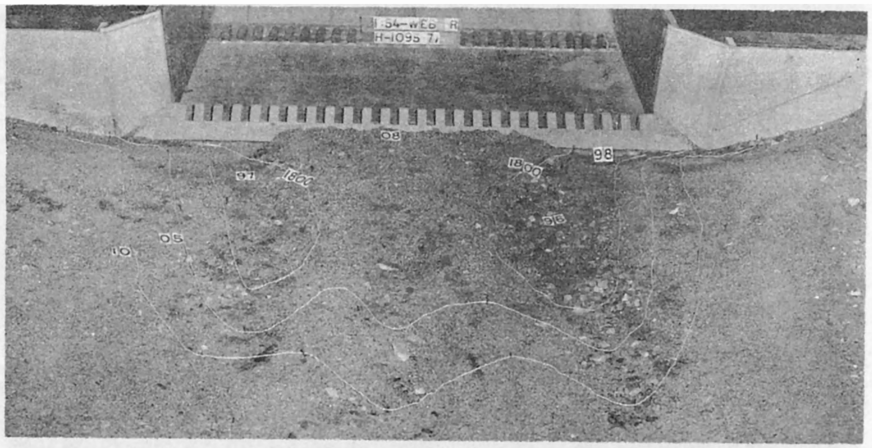
WEBSTER DAM SPILLWAY
Erosion Tests for Proposed Modifications of the Preliminary Basin
1:54 SCALE MODEL



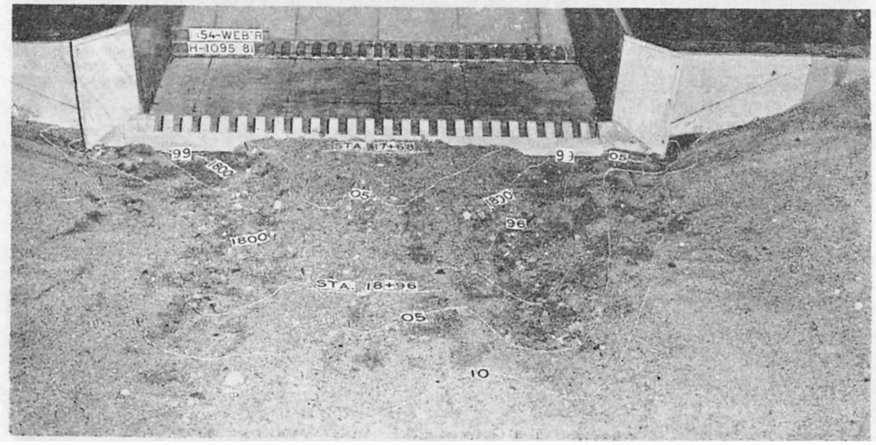
A. Smaller end sill, 7-1/2 feet high, wider slots



B. Sloping apron, upstream end elevated 3 feet



C. 45° training walls at basin corners



D. 45° training walls with 90° wing walls moved upstream 12 feet

Scour Patterns After 30 Minute Model Erosion Tests with 136,000 Second-feet

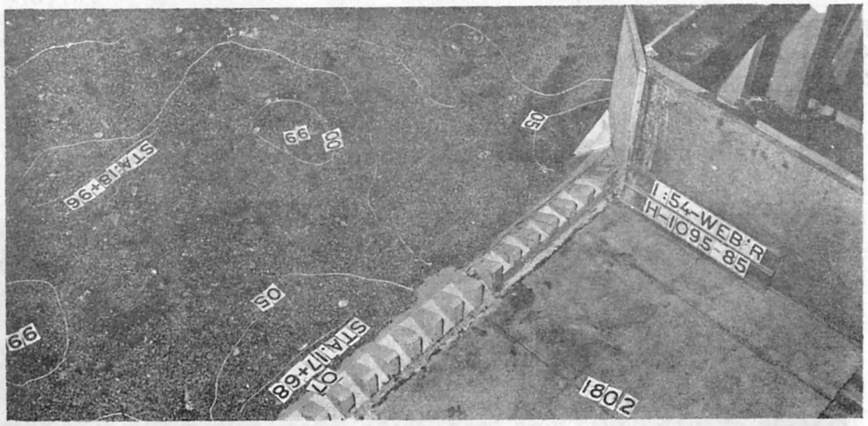
Sheet 2 of 2

WEBSTER DAM SPILLWAY
Erosion Tests for Proposed Modifications of the Preliminary Basin
1:54 SCALE MODEL

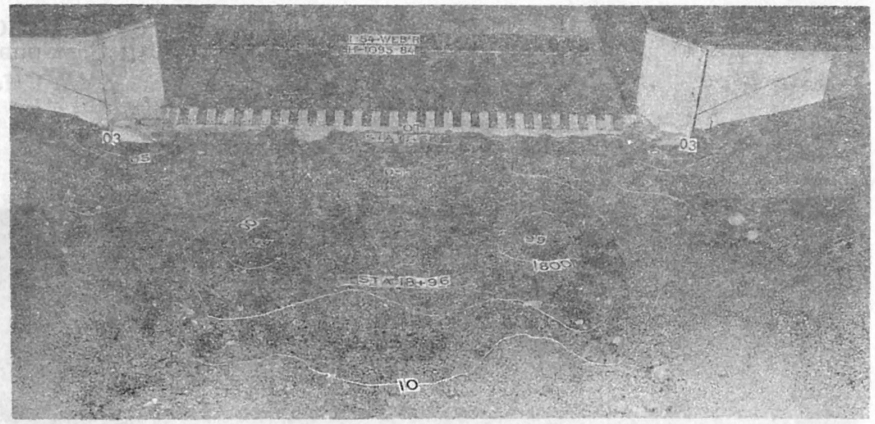
MODEL STILLING BASIN ARRANGEMENT	Erosion test Figure No.	Scour depth (ft)* Right corner	Left corner
Preliminary basin	24	-6	-3
Preliminary basin without end sill	29	-30	-21
Preliminary basin with apron 4 feet higher	30(A)	-6	-6
Preliminary basin with end sill moved upstream 22 feet	30(B)	-12	-9
Preliminary basin lengthened 45 feet	30(C)	-5	-4
Preliminary basin lengthened 45 feet and apron 4 feet higher	-	-6	-6
Preliminary basin lengthened 45 feet plus 45° spur walls	-	+3	+5
Preliminary basin with 45° spur walls	30(D)	-1	+1
Preliminary basin with 45° spur walls and apron 4 feet higher	-	-5	0
Preliminary basin with smaller end sill	31(A)	-7	-8
Preliminary basin with upstream end of apron elevated 3 feet (sloping apron)	31(B)	-6	-7
Preliminary basin with sloping apron with tail water depth 2 feet above normal	-	-5	-4
Preliminary basin with sloping apron with tail water depth 3 feet above normal	-	-4	-3
Preliminary basin with sloping apron plus 45° spur walls 10 feet high above end of basin and tail water depth 3 feet above normal	-	-10	-7
Preliminary basin with 45° corner training walls	31(C)	+3	+3
Preliminary basin with 45° corners and 90° wing walls relocated 12 feet farther upstream	31(D)	+3	+3

*Scour depth is the depth of erosion or deposition as indicated by the plus and minus signs at the left and right corners of the basin looking downstream measured from apron elevation along end sill.

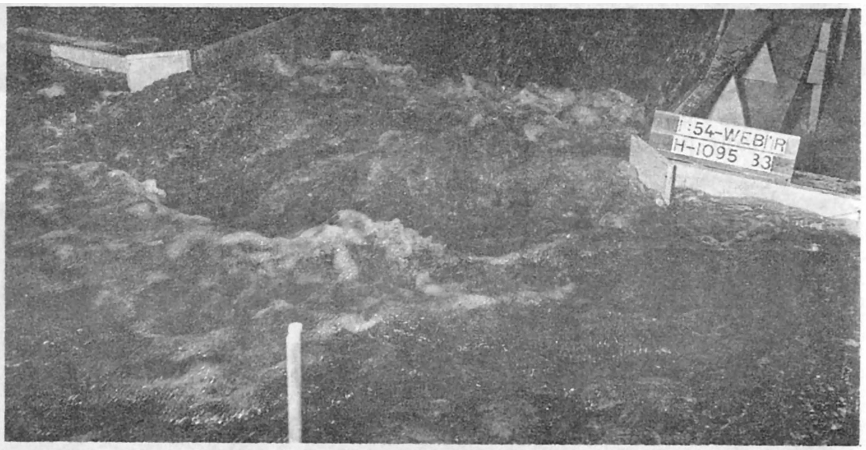
WEBSTER DAM SPILLWAY
Erosion Test Summary--136,000 Second Feet
1:54 scale model



A. The six slots nearest training wall are filled in to sill height. See Figure 8



B. Scour pattern after 30 minute model erosion test. Note absence of erosion along end sill

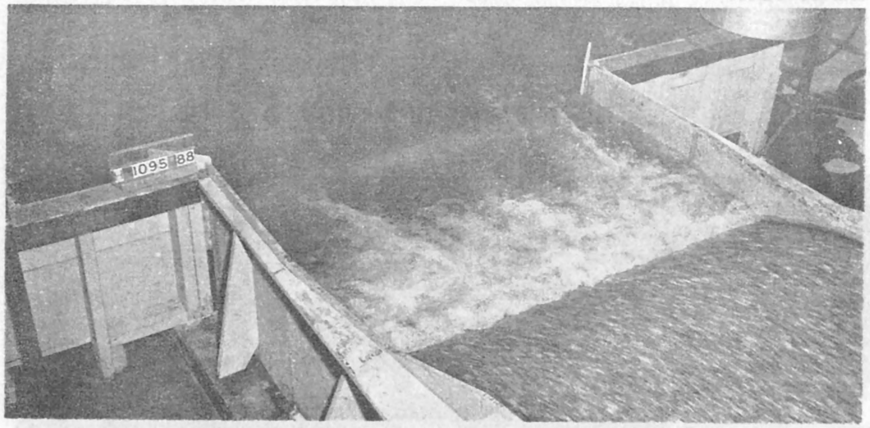


C. Note boil over end sill

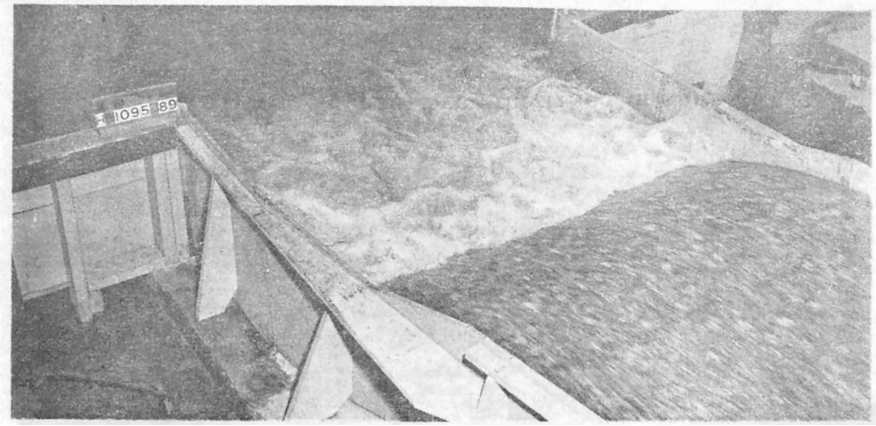
D. Boil is highest over the filled in slots

WEBSTER DAM SPILLWAY
Performance and Erosion Tests on Recommended Stilling Basin--136,000 Second-feet
1:54 SCALE MODEL

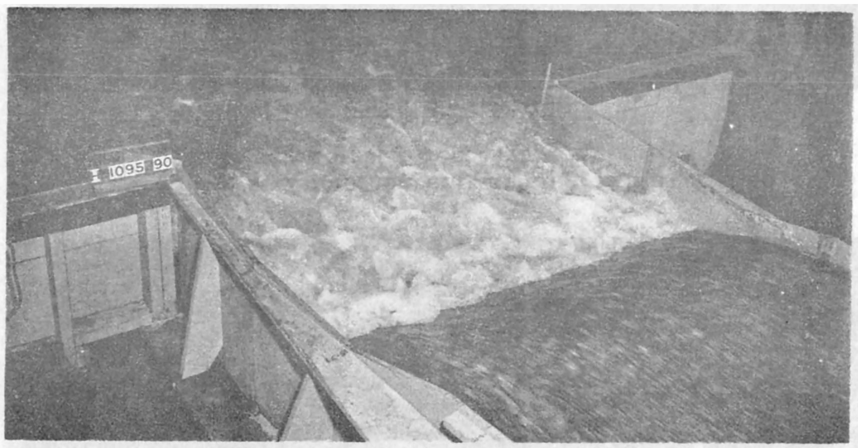
FIGURE 33
Report Hyd-390



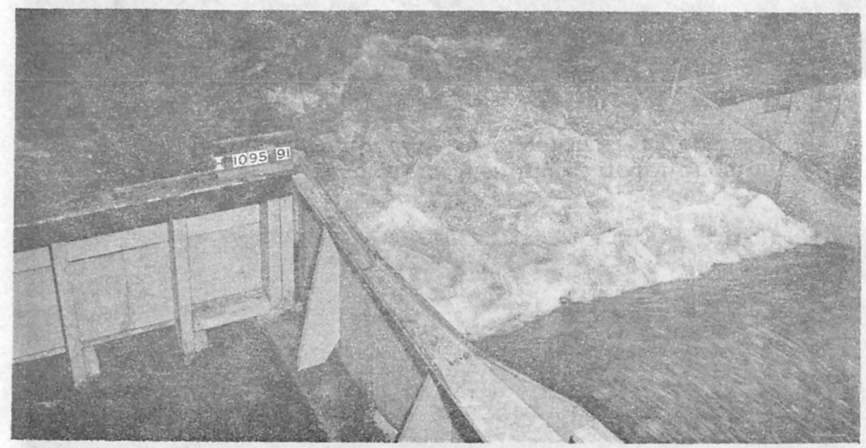
A. 1/4 maximum discharge, 34,000 second-feet. Flow distribution in basin is not uniform



B. 1/2 maximum discharge, 68,000 second-feet

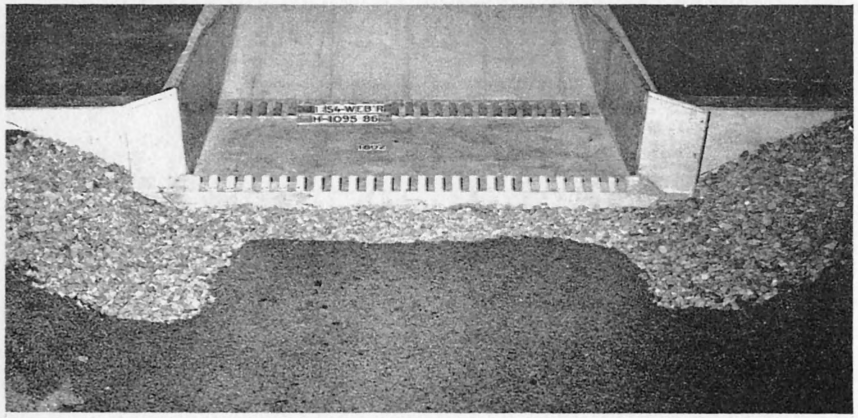


C. 3/4 maximum discharge, 102,000 second-feet

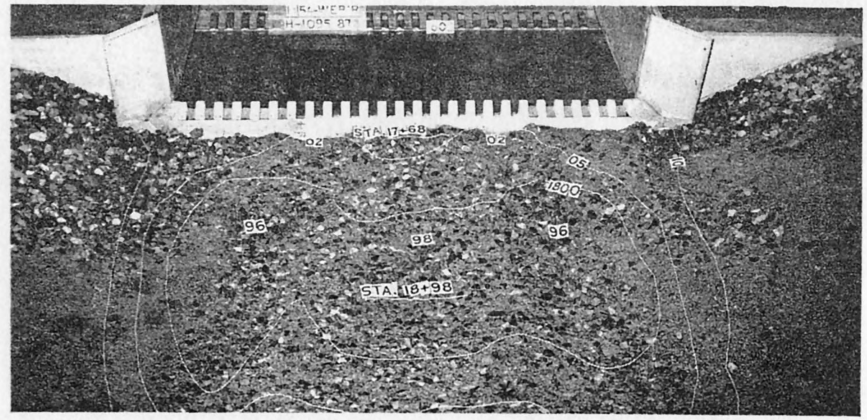


D. Maximum design discharge, 136,000 second-feet

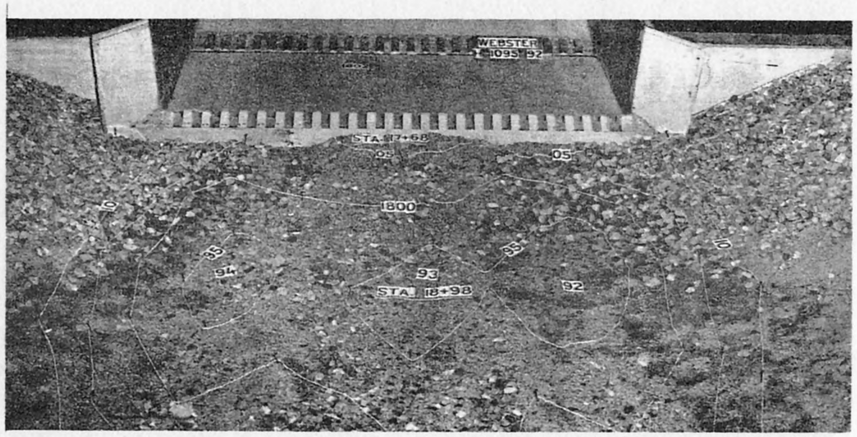
WEBSTER DAM SPILLWAY
Flow Distribution in Recommended Stilling Basin
1:54 SCALE MODEL



A. Discharge channel prepared for erosion tests



B. Scour pattern after 2 hour model erosion test, 136,000 second-feet. Note failure of left bank riprap

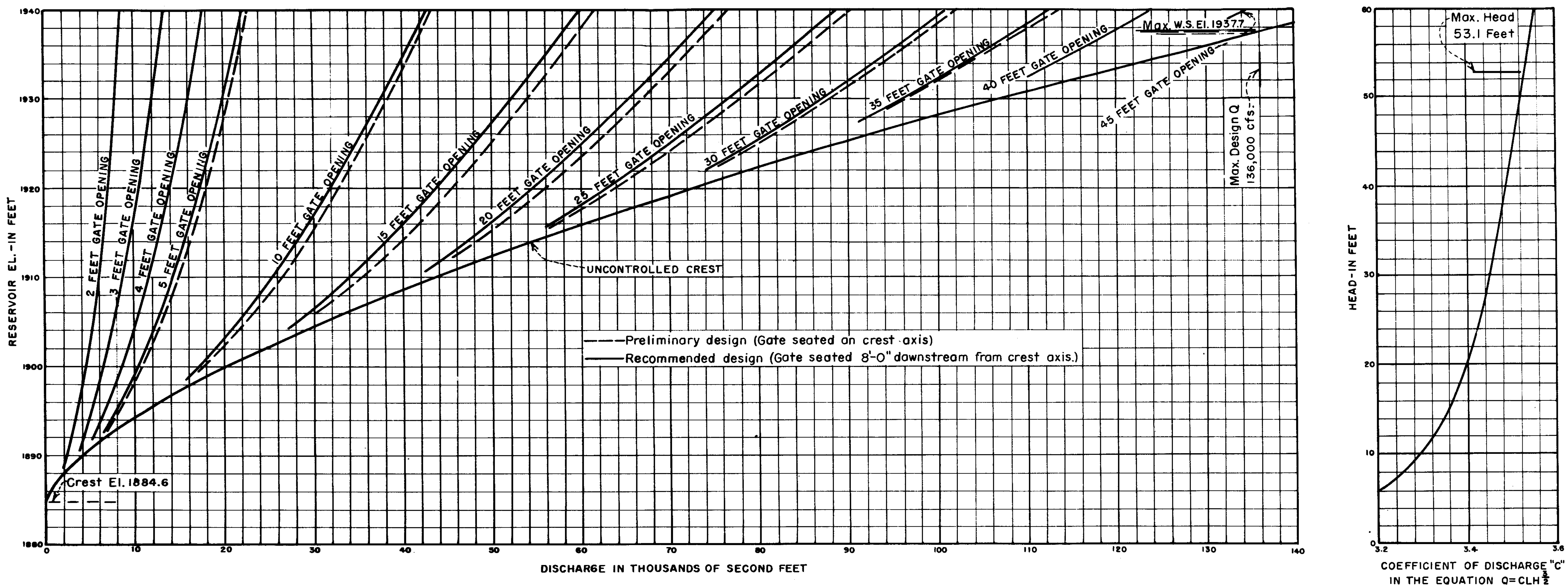


C. Scour pattern after 8 hour model erosion test. 2 hours 1/4 maximum flow, 2 hours 1/2 maximum flow, 2 hours 3/4 maximum flow and 2 hours at maximum. Note no failure of bank riprap

D. Test in (C) continued 4 more hours with maximum discharge. Note right bank riprap failed

WEBSTER DAM SPILLWAY
Recommended Stilling Basin Erosion Tests
1:54 SCALE MODEL

FIGURE 35
Report Hyd-390



NOTE: Gate opening is defined as the difference in elevation between the bottom of the gate and crest elevation 1884.6

WEBSTER DAM SPILLWAY
DISCHARGE CALIBRATION AND COEFFICIENT CURVES
1:54 SCALE MODEL