UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

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Technical Memorandum No. 290

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HYDRAULIC LABORATORY REPORT HYD. 1.1

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RESULTS OF HYDRAULIC MODEL STUDIES ON THE CLE ELUM DAM SPILLWAY. FINAL REPORT

by

E. W. LANE, RESEARCH ENGINEER

- - -

Denver, Colorado,
April 28, 1932

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DEPARTMENT OF THE INTERIOR

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MEMORANDUM TO CHIEF DESIGNING ENGINEER

SUBJECT: RESULTS OF HYDRAULIC MODEL STUDIES

ON THE CLE ELUM DAM SPILLWAY.

FINAL REPORT

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(E. W. Lane, Research Engineer)

Subject: Results of Hydraulic Model Studies on the Cle Elum Dam Spillway. Final Report.

The Cle Elum Dam

The Cle Elum Dam is being constructed to store water for the Yakima Storage Project in Washington. It will be of the earth-fill type, 130 ft. high and store 360,000 acre feet of water, making a lake 4440 acres in area. Spillway capacity for 40,000 sec. ft. will be provided. Ledge rock is too far beneath the surface to be used as a foundation for a spillway, and a trough type founded upon gravel was decided upon. Under maximum flow conditions the water would fall 110 ft. in this spillway. A general plan of the dam and spillway is shown on Figure 1. As an aid to developing an economical and safe design for these conditions, extensive model tests were undertaken.

Hydraulic Laboratory of the Colorado Agricultural College

By the generous permission of the Colorado Agricultural College, experiments for the design of the Cle Elum spillway were carried out in their hydraulic laboratory (Plate I-A) which has been described in detail in the Engineering News, Volume 70, page 662, Oct. 2, 1913.

Figure 2 is a drawing of the laboratory showing the location of the models. The flow was obtained from a reservoir of 30,000 cu. ft. capacity located upon a hill behind the laboratory (Plate I-B). The flow out of the reservoir was controlled by hand operated gates. From these gates the discharge passed into a weir box 19.5 ft. long, 10 ft. wide, and 7 ft. deep. In the side of this box 13 ft. upstream from the weir was a by-pass which was controlled by a movable crest, and another of smaller discharge controlled by a valve. Fine adjustments of the quantity discharged through the model were made by varying the flow through these by-passes. The head on the weir was observed by means of a float gauge similar to that developed at the Cornell University. The gauge was located in a stilling pool connected with a main channel by a pipe, as shown in Figure 2.

The discharge through the spillway model was measured over two types of weirs. During the first part of the work, a 90° V notch weir was used (Plate I-C). This had been previously volumetrically calibrated by the laboratory staff of the Colorado Agricultural College in the exact setting in which it was used and their results were therefore adopted. In order to accommodate the experiments on the models of the Hoover Dam spillways, it was necessary to use a weir of larger capacity and higher crest level. The latter part of the Cle Elum experiments were therefore conducted with a 2 ft. Cipolletti weir with its crest 1 ft. higher than the apex of the V notch. This weir had been previously calibrated by the college staff with a crest height above the channel floor of 1 ft. less than that at which it was used in these experiments. As the velocity head in both of these settings of the Cipolletti weir was negligible, only a few calibration observations on the weir in its higher setting were made. These agreed with those previously run by the college staff at the 1 ft. lower crest elevation and discharges were therefore based on a curve giving the coefficient for various heads as derived from both sets of observations.

From the weir the water passed through a diverting gate by means of which the flow could be rapidly deflected into the model or into a waste tank. From this gate a short channel 6 ft. wide, 17 ft. long and 8.5 ft. deep formed a supply channel for the model. As the direction of the flow in the model was at right angles to that in this channel, baffles were necessary to prevent boils and whirls in the water within the model. After considerable experimentation a set of baffles was evolved which would eliminate practically all of these disturbances. From the supply tank the water passed through the headgate structure of the model, down the trough and through the stilling pool. From the stilling pool it dropped to the bottom of the tank in which the model was constructed, and was pumped back to the supply basin upon the hill behind the laboratory.

Description of the Model.

Figure 3 is a drawing of the model of the Cle Elum spillway and Plate II-A shows the type of construction used. This model was set up in the calibration tank which is located inside the Hydraulic Laboratory building. The dimensions of the tank limited the scale of the model to 1/50 of the linear dimensions of the spillway of the dam. For this ratio the relation of the dimensions and quantities of the model and prototype are shown in the following tables. The symbol N will be used to express the ratio of the linear dimension of the prototype to that of the model; i.e. N = 50.

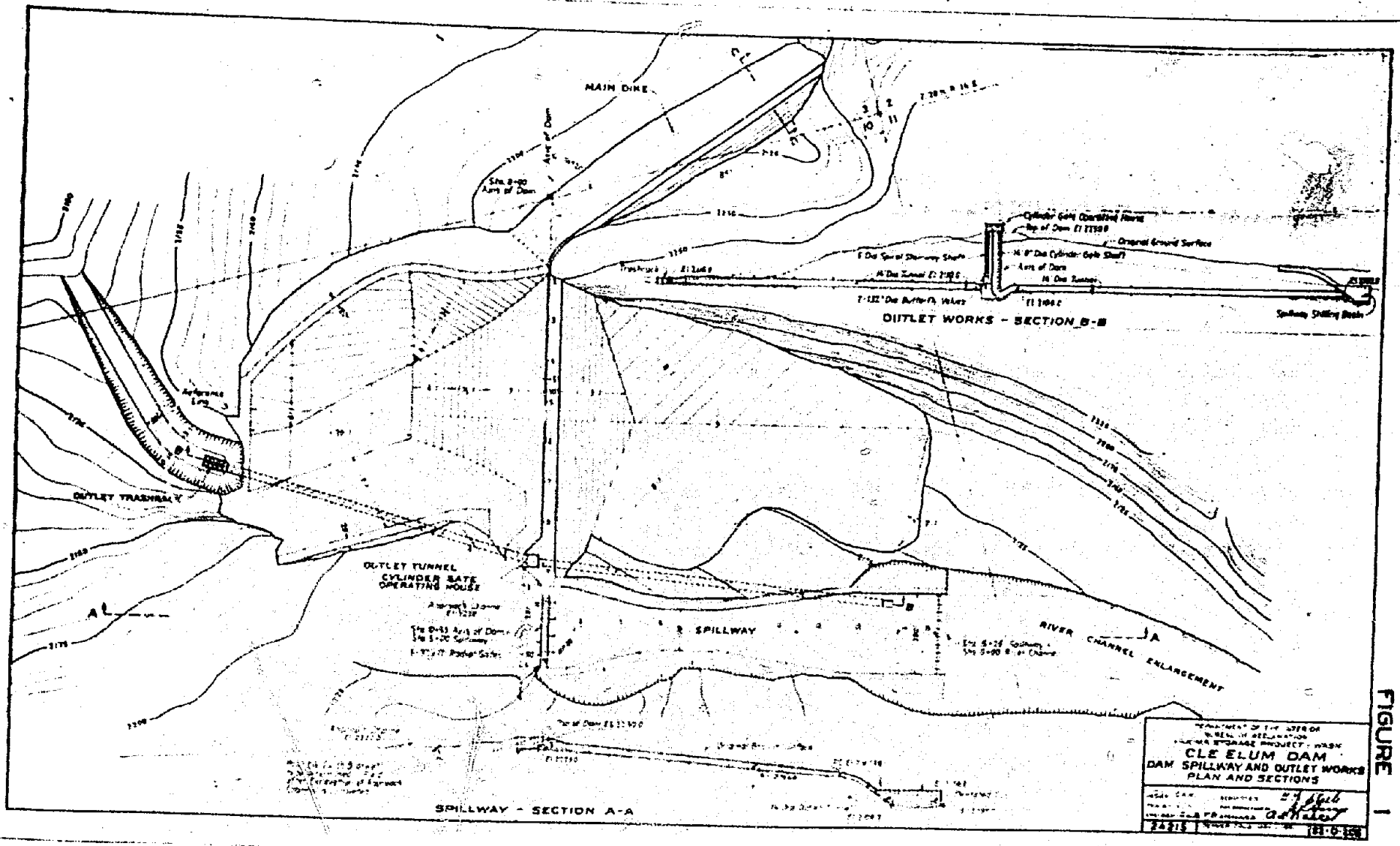

MAIN DIKE
OUTLET TRASHRACK
OUTLET TUNNEL
CYLINDER GATE
OPERATING HOUSE
SPILLWAY
RIVER CHANNEL ENLARGEMENT
OUTLET WORKS - SECTION B-B
SPILLWAY - SECTION A-A
CLE ELUM DAM
DAM SPILLWAY AND OUTLET WORKS
PLAN AND SECTIONS

FIGURE 1

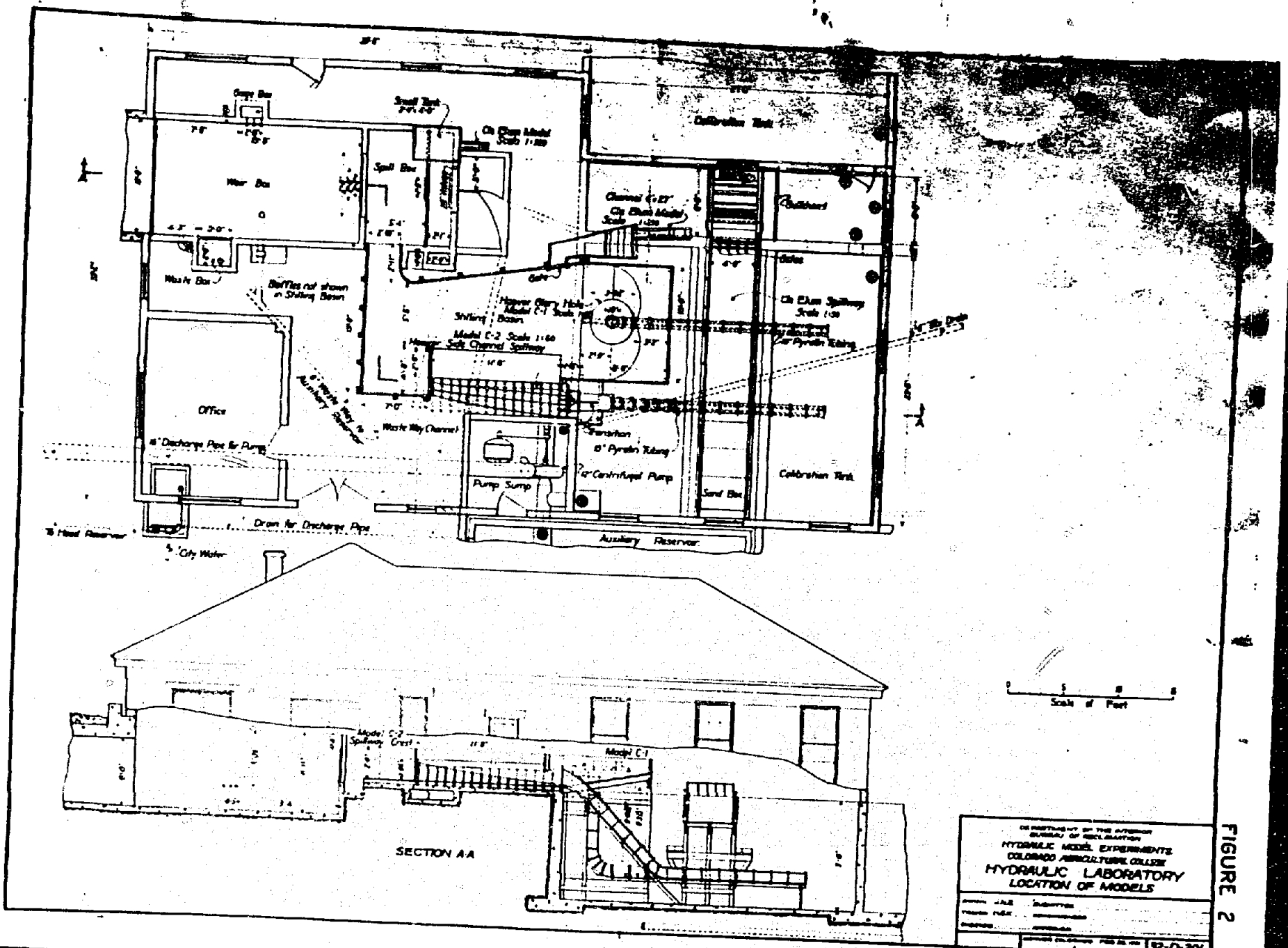

Gage Box
Small Tank
Cle Elum Model Scale 1:200
Calibration Tank
Weir Box
Spill Box
Channel C-27
Cle Elum Model Scale 1:200
Bulkhead
Gates
Waste Box
Baffles not shown in Stilling Basin
Gate
Hoover Glory Hole Model C-1 Scale 1:60
Stilling Basin
Model C-2 Scale 1:60
Hoover Side Channel Spillway
Cle Elum Spillway Scale 1:50
Pyrolin Tubing
Office
Waste Way Channel
Transition
10" Pyrolin Tubing
12" Centrifugal Pump
Pump Sump
Sand Box
Calibration Tank
16" Discharge Pipe for Pump
Drain for Discharge Pipe
To Head Reservoir
City Water
Auxiliary Reservoir
Model C-2 Spillway Crest
Model C-1
SECTION A-A
Scale of Feet
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
HYDRAULIC MODEL EXPERIMENTS
COLORADO AGRICULTURAL COLLEGE
HYDRAULIC LABORATORY
LOCATION OF MODELS
32-D-291

FIGURE 2

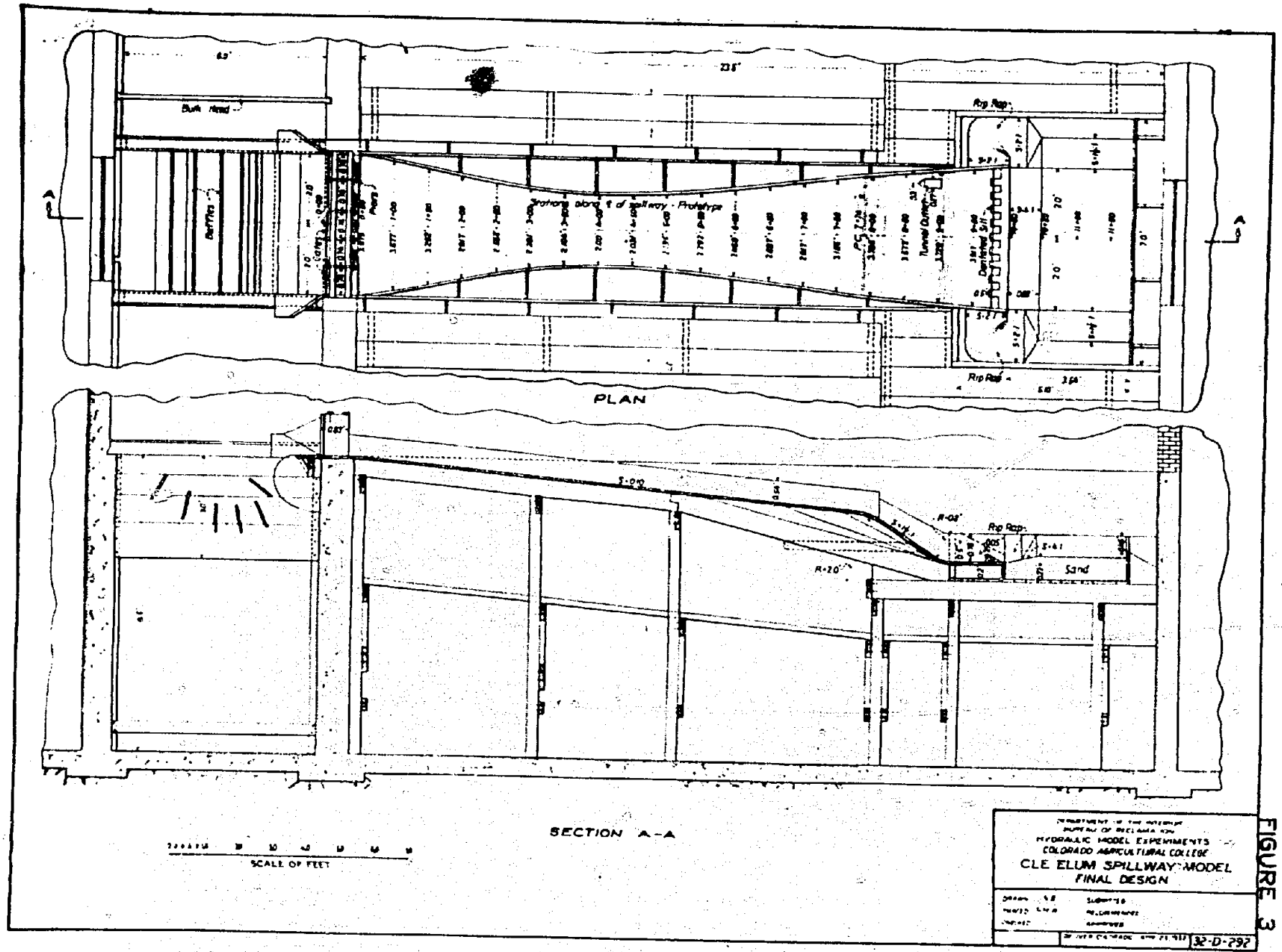
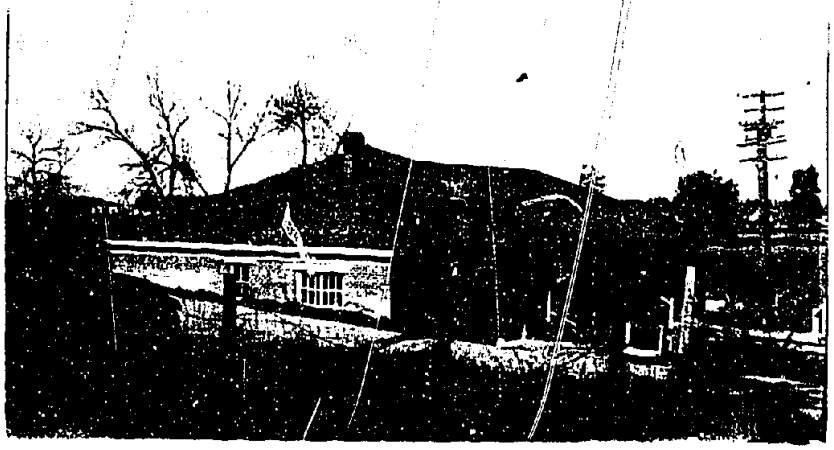
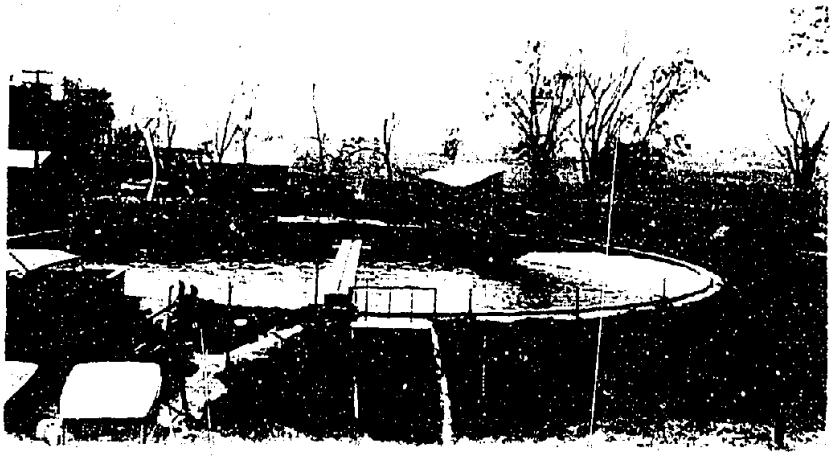

Bulk Head
Baffles
Piers
Stations along ℄ of spillway - Prototype
Tunnel Outlet
Rip Rap
PLAN
Rip Rap
Sand
SECTION A-A
SCALE OF FEET
HYDRAULIC MODEL EXPERIMENTS
COLORADO AGRICULTURAL COLLEGE
CLE ELUM SPILLWAY MODEL
FINAL DESIGN
32-D-292

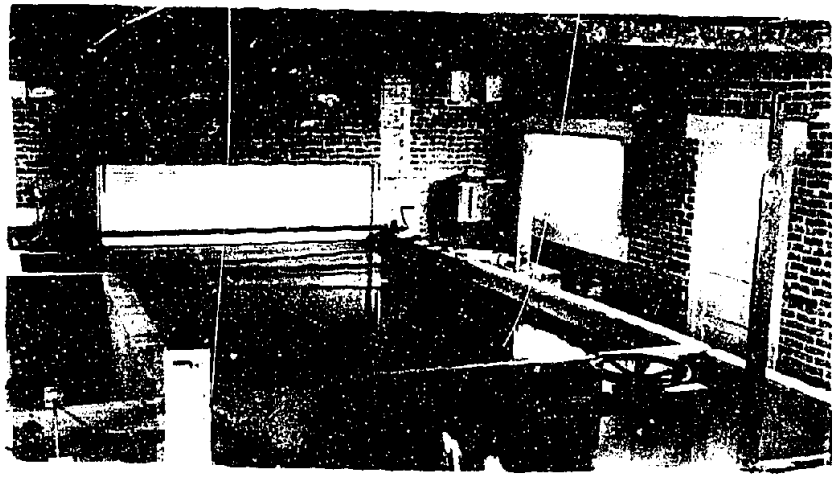
FIGURE 3



A-THE LABORATORY

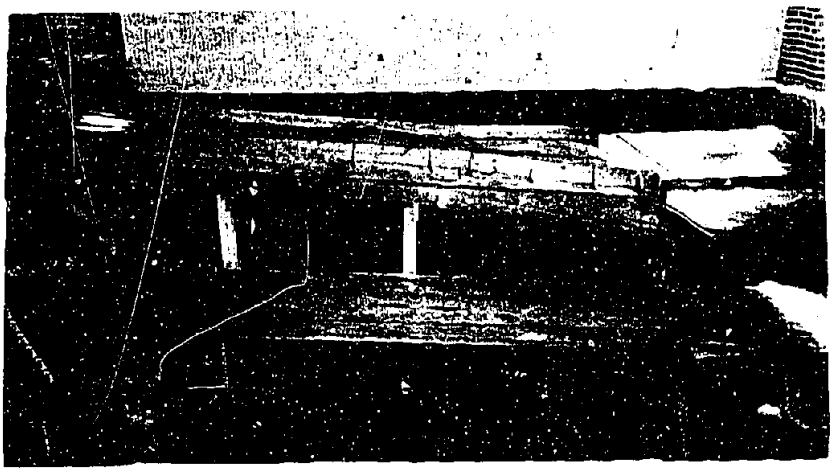


B-THE SUPPLY RESERVOIR FOR THE LABORATORY



THE WEIR BOX AND MEASURING WEIR

THE HYDRAULIC LABORATORY OF
THE COLORADO AGRICULTURAL COLLEGE



a-MODEL OF CLE ELUM SPILLWAY
ORIGINAL DESIGN-SCALE 1:50

b-USE OF THE POINT GAGE TO
DETERMINE WATER LEVEL

Ratio of Prototype to Model

Quantity	Formula	Value
Linear Dimensions	N	50
Head	N	50
Velocity	$\sqrt{N}$	7.071
Area	N^2	2500
Discharge	$N^{5/2}$	17,667
Time	$\sqrt{N}$	7.071

	Model	Prototype
Length (approximate)	20 ft.	1000 ft.
Max. Width	4.0 ft.	200 ft.
Fall	2.2 ft. (Approximate)	110 ft.
Velocity	11.6 ft. per sec.	82.0 ft. per sec.
Design Discharge	2.26 Sec. ft.	40,000 sec. ft.

The gate structure at the upper end of the spillway was made with wooden piers with 5 gates of 1/8" sheet steel, sliding in grooves in the gate piers. The trough or flume section was built on a sloping floor, 5' wide, supported by scaffolding with walkways along both sides. The floor was covered with galvanized iron laid as smoothly as possible to minimize friction. The sides of the flume were constructed of flexible boards resting on the galvanized iron floor. On account of the fact that the friction losses do not follow the law of geometric similarity, it was necessary to incline the floor more steeply in the model than in the prototype. The floor of the model was computed to give the same relative velocity as the prototype using a roughness coefficient in Manning's formula of 0.014 in the prototype and 0.009 for the smooth galvanized iron floor of the model. Blocks were placed under the supports of the flume floor in order to permit adjustments in the slope, should that be found necessary. Observations by means of the Pitot tube showed velocity in the model almost exactly proportional to those indicated by the computations for the prototype, and adjustments of the floor were therefore unnecessary. At the lower end of the trough section the inclination of the floor was changed by means of a vertical curve to a slope of 1½ H to 1 V, which led to the stilling pool. The model was so constructed that this slope could readily be changed to 2:1, 3:1 or 4:1.

Two types of stilling pool were used, the first of these was 4 ft. wide with vertical side walls corresponding to the 200 ft. width of the prototype. On one side, the wall was formed of plate glass in order that the action of the water within the pool might be observed. Downstream from the stilling pool was a pit of sand

confined between an extension of the vertical walls of the pool. Later a larger sand box was constructed in which the conditions more nearly reproduced those in the prototype. The height of the tail water in the stilling pool was controlled by means of a weir, the height of the crest of which could be adjusted by stop logs.

Field Covered by the Experiments

The design of the Cle Elum spillway from the hydraulic standpoint involved two major problems, each of which had a number of subdivisions. The first major problem was the design of the trough or flume in which the water was to be carried from the lake created by the dam to the stilling pool. The second problem was the determination of the design of the pool in which the energy of the falling water could be dissipated without causing damage to the dam or surroundings. The following outline shows the main subdivisions of these two problems which were investigated by means of the models.

Design of the Trough Section

(1) Original Design
- All gates open
- Various combinations of gates open and closed

(2) Flume with Parallel Sides
(3) Best Form for Various Throat Widths
(4) Use of Control Section at Throat
(5) Development of Final Design
- All gates open
- Various combinations of gates open, closed and partly open

(6) Analysis of Wave Action

Design of Stilling Pool

(1) Consistency of Scour Test Results
(2) Development of a Standard Procedure
(3) Best Alignment of Side Walls
(4) Best Slope Leading into Pool
(5) Curve vs. Angle at Bottom of Entrance Slope
(6) Best Floor Shape at End of Pool
(7) Best Form and Location of Baffles
(8) Best Balance of Length, Width, and Depth of Pool
(9) Scour at Partial Flows
(10) Best Form of Lower End of Side Walls
(11) Best Form of Bank Protection at Lower End
(12) Shape of Tunnel Entrance on Slope Leading into Pool
(13) Upward Pressure on the Floor

Experiments on Design of the Original Trough

A preliminary design for the trough of the Cle Elum spillway was worked out in the Denver office of the Bureau of Reclamation. The principal dimensions are shown on Figure 4 and Table I. Thorough studies were made in order to obtain the best form which could be determined without the aid of model studies. The results obtained on a model of this form illustrated clearly the limitations of this method of design and the necessity of model studies, if good designs of such structures are to be obtained. Upon turning the water into the model based on this original design, it was immediately evident that major changes would be required. Two large waves were formed in the trough crossing and recrossing it. This would have required high side walls on the prototype at certain places, and cause excessive loads in the spillway floor and concentrated flow entering the stilling pool in a manner which would have resulted in undesirable erosion. The nature of this wave produced with all the headgates open is shown on Plate III-A. On Figure 4 is plotted the results of readings taken by means of the point gauge (shown on Plate II-B). This figure shows the depth of the water and height of the waves at a number of cross sections along the flume.

The data on this figure is given in terms of the values on the actual spillway rather than on the model. In the following discussions and illustrations, unless otherwise noted, all dimensions and quantities will be expressed in terms of the corresponding prototype values, since this gives a picture which is more easy to visualize. The station numbers differ from those finally adopted on the prototype. Those used in this report are five stations less than those used on the contract drawings.

The conditions of flow produced by all of the possible combinations of headgates open and closed in the original design were also investigated. Observations were not made of the various possible combinations with gates partly open, but only of those in which the gates were either entirely open or entirely closed. Figure 7 shows the height and location of the crest of severe waves at the sides of the trough, produced by all of the 18 possible combinations of gates, open and closed. Plate III-B and C show the flow for two of the most severe cases. Although these points indicate the height of wave which would be necessary to provide for if the action of the prototype followed exactly that of the model, it would not be safe to design the structure on this basis because of the limitations of model experiments. One of these limitations is that the surface tension in the water is the same in both model and prototype. On this account the water in the model is relatively much more smooth than in the prototype and therefore waves might be expected to dash higher in proportion in the prototype than in the model. Another

TABLE I

Widths in Feet of Various Troughs Investigated for the Cle Elum Spillway

Station	Original Design	Parallel Sides	60-Foot Throat	80-Foot Throat	100' Throat Third Form	100' Throat Fourth Form	Final Design	100' Throat 150' Pool	100' Throat 120' Pool
0+00	200	200	200	200	200	200	200.0		
0+50	200	200	197	197	193	193	199.0		
1+00	190	200	182	180	182	182	183.8	184	185
1+50	164	200	164	162	168	168	164.6	160	165
2+00	138	200	142	140	148	148	145.8	148	147
2+50	122	200	115	120	128	128	128.1	129	129
3+00	120	200	92	101	110	110	114.0	114	114
3+50	120	200	69	86	101	101	102.2	101	102
4+00	120	200	57	77	98	98	100.0	97	97
4+50	120	200	57	75	100	100	101.5	99	99
5+00	120	200	65	77	103	105	106.8	103	102
5+50	120	200	79	82	110	113	114.6	109	106
6+00	131	200	99	90	115	123	123.4	116	109
6+50	152	200	122	103	129	135	134.4	122	112
7+00	171	200	145	113	140	146	145.8	150	116
7+50	191	200		139	149	168	157.6	136	119
8+00	200	200			158	175	169.2	142	121
8+50	200	200			170	185	178.6	145	121
9+00	200	200			181	192	186.4	145	121
9+50	200	200			191	196	195.8	145	121
10+00	200	200			200	200	200.0	145	121

limitation of model experiments is that no information on the amount of air which would be entrained in the water flowing down the spillway is given by them. Experience has shown that in thin sheets of water at high velocity large quantities of air may be taken up and the flow depth in the prototype may therefore be thicker than the tests on the model indicate. For this reason a liberal factor of safety would be necessary in designing the side walls using the results of these experiments. However, the experimental results have eliminated many of the unknown factors and are therefore valuable even though they do not completely solve the problem. Vanes were tried to improve the flow conditions in this flume but were unsuccessful. Copings on the side walls were tested to determine the possibility of turning out the crests of the waves and thus permitting the use of lower walls than would otherwise be required, but no satisfactory forms were developed.

Experiments on a Trough with Parallel Sides

Experiments were next made upon a trough with parallel sides 4 ft. apart, representing a 200 ft. width in the prototype. The flow in this trough was much smoother than that in the original design, although some wave action was still set up by the transition section at the headgates, and by the gate piers. Plate IV-A is a photograph of the flow through this trough corresponding to the 40,000 sec. ft. maximum discharge of the prototype. The flow for various gate combinations is shown on Plate IV-b and c. The depths of flow for various discharges in this flume with all gates open are shown quantitatively on Figure 5.

It was found that the conditions of flow in this flume were somewhat unstable. More flow was found to pass on the right side of the flume at the lower end than on the left side. An attempt was made to remedy this condition by closing somewhat the headgate on the right side of the flume. This however, increased the flow on the right side instead of decreasing it. To decrease the flow on the right side at the lower end of the flume it was necessary to partially close the gates on the left side. This would indicate that the greater flow obtained at the lower end of the right side with all gates open, was due to the floor at the gate structure being higher on the right side thus causing a greater flow through the gates on the left side. Measurements, however, showed that instead of being higher on the right side, the floor was actually 0.175 ft. (prototype scale) lower. No explanation of the unbalanced condition of flow was obtained, but as this type of flume was soon discarded it was not considered worthwhile to continue the investigation of this point further. These results indicated that the flow through the parallel sided flume was unstable, and that a condition of unequal flow might result from small irregularities such as a curve in the approach channel to the spillway.

FIGURE 4

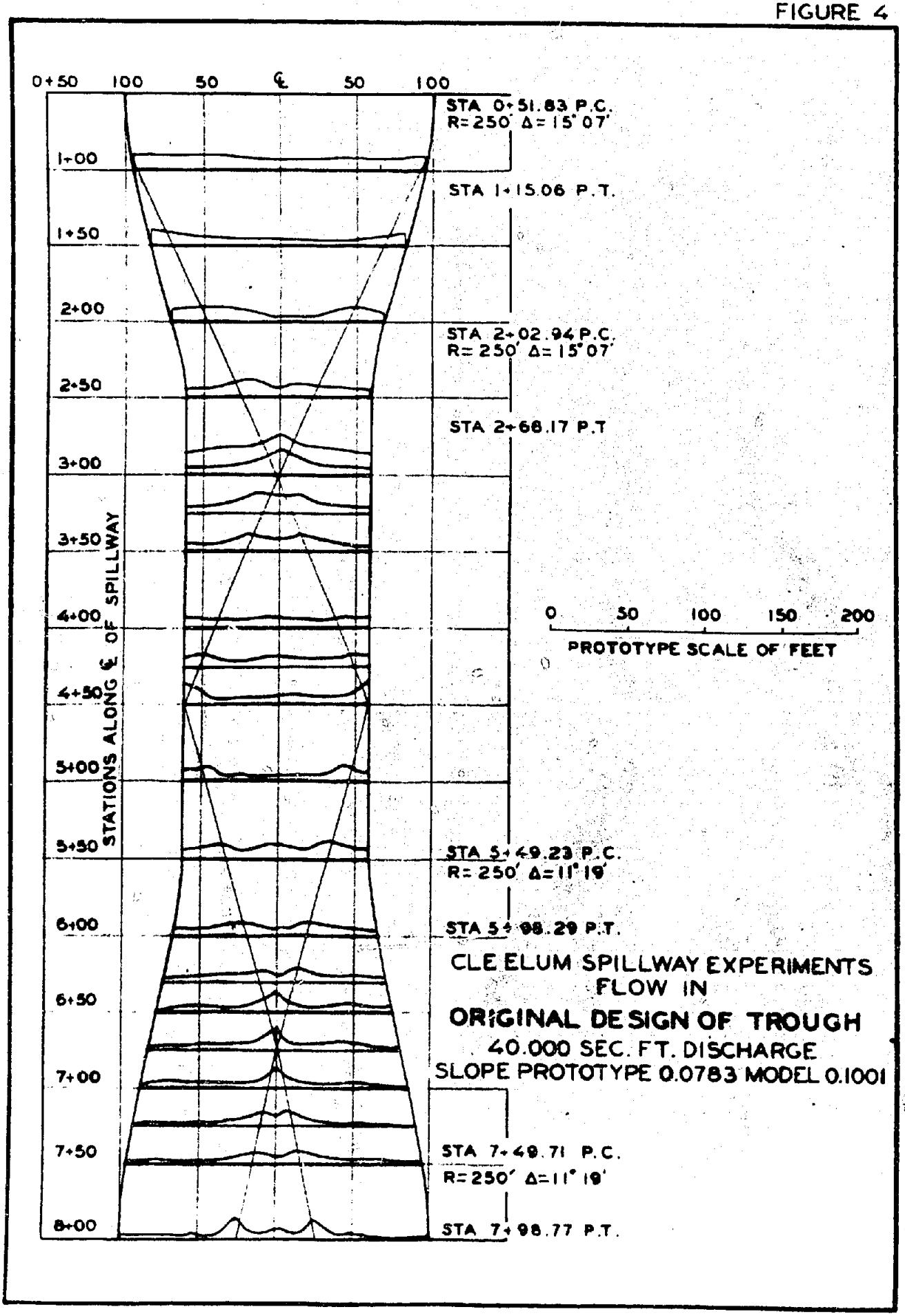
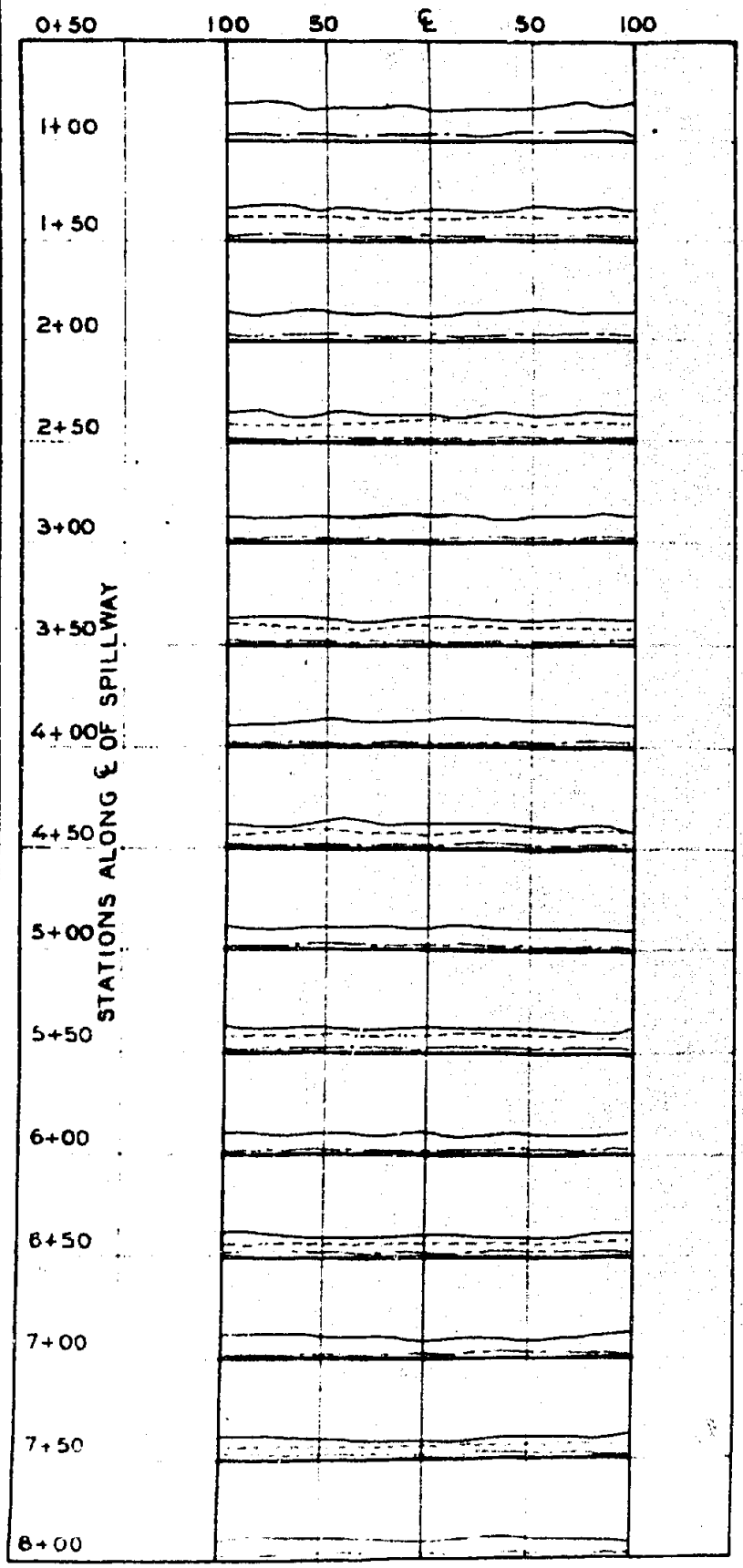

0+50
100
50
℄
50
100
STA 0+51.83 P.C.
R=250' Δ=15°07'
1+00
STA 1+15.06 P.T.
1+50
2+00
STA 2+02.94 P.C.
R=250' Δ=15°07'
2+50
STA 2+68.17 P.T
3+00
3+50
4+00
4+50
5+00
5+50
STATIONS ALONG ℄ OF SPILLWAY
STA 5+49.23 P.C.
R=250' Δ=11°19'
6+00
STA 5+98.29 P.T.
6+50
7+00
7+50
STA 7+49.71 P.C.
R=250' Δ=11°19'
8+00
STA 7+98.77 P.T.
0
50
100
150
200
PROTOTYPE SCALE OF FEET
CLE ELUM SPILLWAY EXPERIMENTS
FLOW IN
ORIGINAL DESIGN OF TROUGH
40.000 SEC. FT. DISCHARGE
SLOPE PROTOTYPE 0.0783 MODEL 0.1001

FIGURE 5



EXPLANATION

——— FLOW 140,000 SEC. FT.

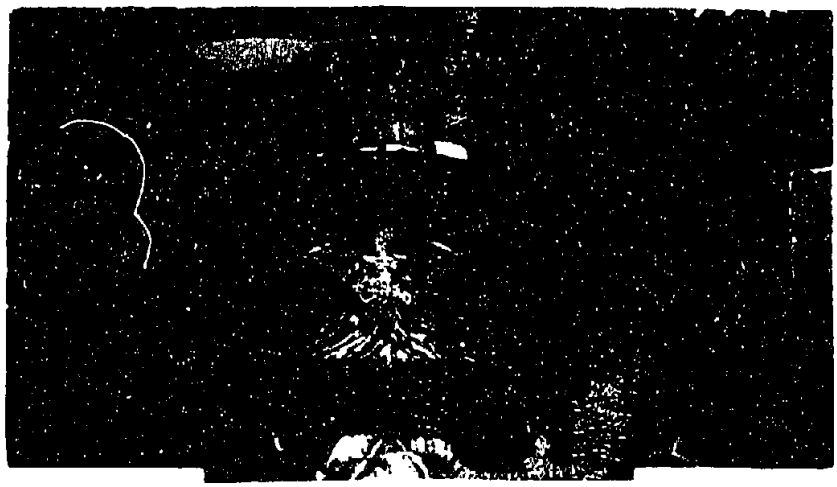
- - - - - - - FLOW 90,000 SEC. FT.

—— - —— FLOW 20,000 SEC. FT.

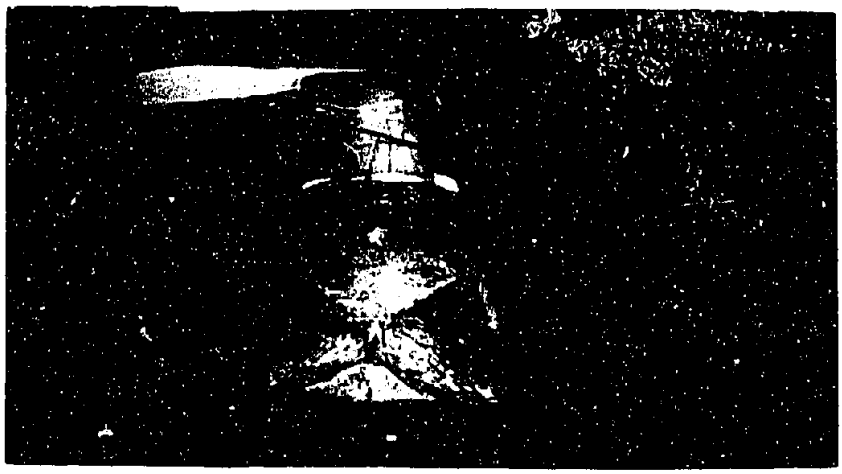
0 50 100 150

PROTOTYPE SCALE OF FEET

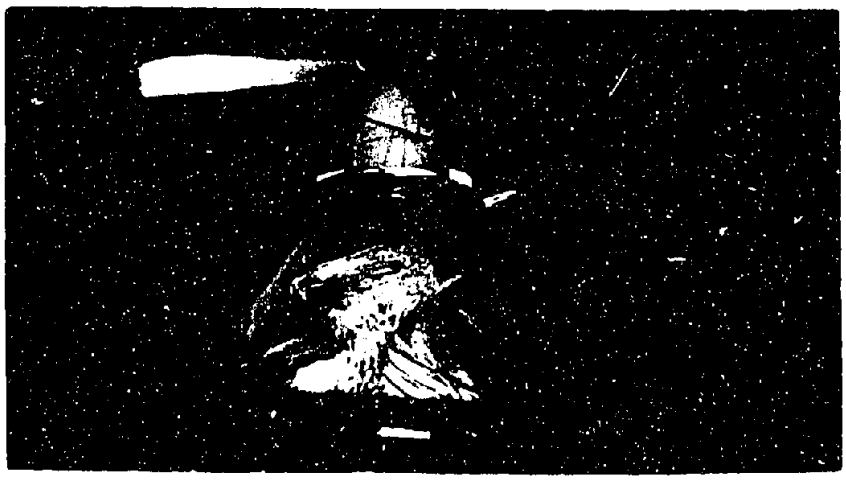
CLE ELUM
SPILLWAY EXPERIMENTS
FLOW IN
TROUGH WITH PARALLEL SIDES



a-FINAL DESIGN 40,000 SEC. [illegible]
LOOKING DOWNSTREAM

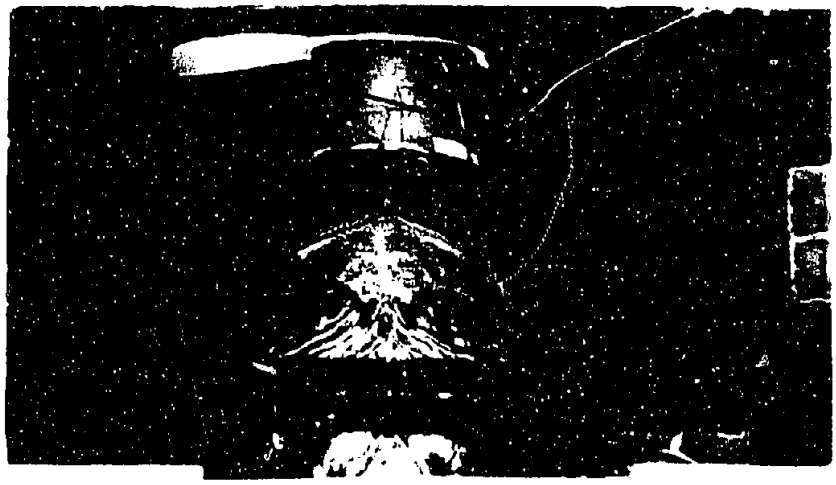


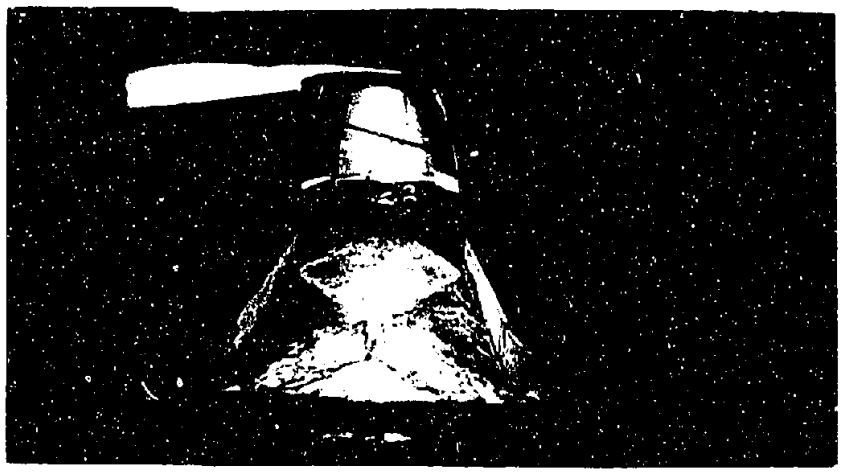
b-RESERVOIR AT MAXIMUM LEVEL
GATES 2,3 AND 4 OPEN

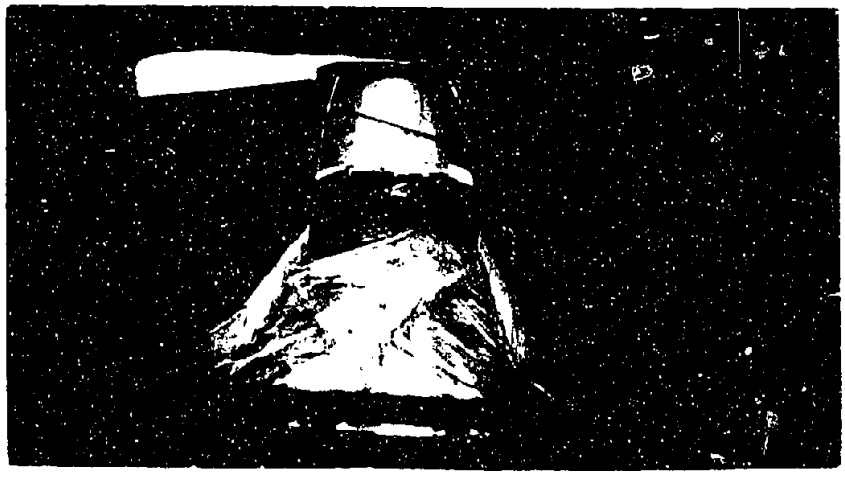


c-RESERVOIR AT MAXIMUM LEVEL
GATES 1,3,4 AND 5 OPEN

FLOW IN ORIGINAL DESIGN TROUGH







Development of a Trough with Narrow Throat and Uniform Bottom Gradient

In order to economize on concrete in the spillway floor, flume forms were developed with a narrow throat somewhat similar to the original design, but with much less abrupt convergence and divergence. By means of flexible sides, a great many shapes of flume were tried and the best ones were measured and recorded. Shapes with throat widths of 100, 80 and 60 feet for the prototype were developed. From the standpoint of flow in the trough these were about equally desirable. Greater throat widths were also tried but were not so successful, especially in the case of those approaching 200 ft. width. In these forms the waves set up by the transition at the entrance section above the gates did not travel entirely across the flume before the lower end was reached, and ridges of water entered the stilling pool and caused disturbances and excessive erosion. The sections developed expanded to 200 ft. width at the top of the steep drop. After a decision was made on the best width of throat to use, the shape was refined until a form was developed which expanded to the 200 ft. width at the lower end of the pool. The widths of these flumes are given in Table I. Plate V-A shows the shape of the 60 ft. throat width and the wave action with maximum discharge and all gates open. Plate V-B and C show the condition with some of the worst combinations of gates open and closed in the trough of 60 ft. width for a water level in the reservoir at the maximum safe elevation; i.e., the same as that for the 40,000 sec. ft. discharge with all gates open. These results are shown quantitatively on Figure 6. A comparison of 6-a with 6-b shows that the wave action in the flume is not materially influenced by the gate piers. The effect of gate piers is also shown by a comparison of Plates IX-A with IX-C and X-A with X-C. Figure 6-d gives the depths of flow for various cross sections for the flume of 80 ft. throat width, and 6-e for 100 ft. width. Scour tests in the stilling pool also showed these flumes to be preferable to the original design. A comparison of the scour caused by the two types is given on Figure 16.

Use of Control Section at Throat

The grade of the trough section of the Cle Elum spillway is to a certain extent controlled by the topography at the damsite. In order to build the spillway in a cut and have no part of it on a fill, it is necessary that it have a rather steep slope. As previously explained, wave action developed in the flow in this trough. Little difficulty was experienced in securing a satisfactory expanding portion, but the form of the contracting section at the upper end of the flume was found to have an important effect on the wave action set up. The flow throughout the flume with uniform gradient was below the critical depth. In flows at depths greater than the critical,

such waves are not set up. By making the contracting portion of the spillway on a sufficiently flat slope to produce depths greater than the critical depth, waves in this section could be eliminated. An attempt was therefore made to develop a flume with the control section at the most contracted portion, with a flat gradient upstream so that the flow would be below critical velocity in this section and a steep gradient downstream in the expanding section. Computations showed that such a form could be developed which would perform in this manner above any predetermined discharge, but below this discharge there would be two control sections, one at the gates and the other at the spillway throat, with a hydraulic jump between. If the minimum discharge at which the flume would perform with a single control section was less than the maximum discharge for which the spillway was designed, a larger gate structure would be required than for the flume of uniform gradient. It was therefore desirable to make the discharge capacity, at which the flume would operate with one control section, equal to the maximum flood discharge. A 1:250 spillway model was built with a 60 ft. throat width to meet these requirements in order to see how severe the conditions would be due to the hydraulic jump which would form between the two control sections at discharges less than the maximum. The results showed less desirable conditions in this flume than could be obtained in the flume of uniform gradient and therefore no further experiments on this type were performed. The conditions of flow in this flume are shown on Plate VI-A and B. A 1:250 model was built to act similarly with a 120 ft. throat, but the conditions of flow were not better than in the model with uniform gradient. By making the floor slope upward in the direction of flow to the throat section, the control could have been made to come at the throat for all conditions of discharge. Such a plan would have required much larger and more expensive gates and would have required the gate structure, which acts as a dam, to sustain a considerably higher head. This was not desirable as the 17 ft. head contemplated in the design adopted was believed to be as high as was safe for the foundation conditions existing at the spillway site.

Developments of the Final Trough Design

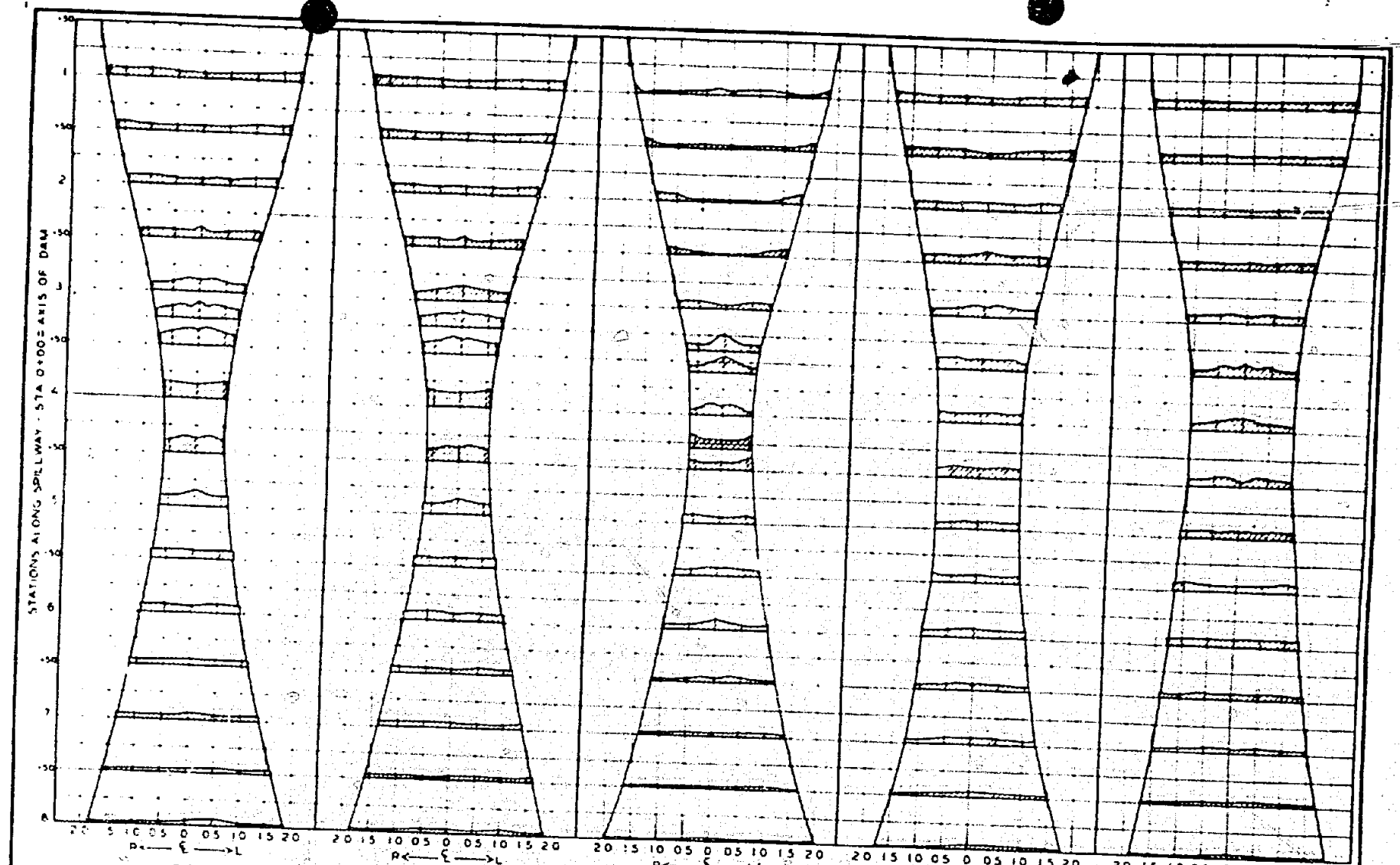
As previously stated, experiments showed that a trough on uniform gradient could be developed for 60, 80 and 100 ft. widths which would be equally desirable from the standpoint of flow in the trough but wider throats were not practicable. Cost estimates showed the higher walls required for the narrow throat about offset the saving in floor concrete which would result from a narrower throat, assuming the same thickness of floor in all cases. Since the narrower floors would have to be thicker on account of the greater pressure exerted upon them by the deeper water in the narrower sections, the

narrow widths were found to be less economical. Since the 100 ft. throat was cheapest and also seemed to produce less scour below the jump pool (See Figure 17) it was adopted.

This section was studied in detail and the final form shown in Figure 3 was adopted as best. The widths are also given in Table I. This form has a 100 ft. throat at station 4+00 and a gradually expanding section which continues to expand to the downstream end of the stilling pool. Plate VII shows conditions of flow in this flume. The wave action with a 40,000 sec. ft. flow is shown on Plate VII-A and VII-B. The greatest waves were formed near the center of the throat. They seemed to be little affected by the shape of the expanding section of the flume. Plate VII-C shows the formation with an expansion to 120 ft. stilling pool width instead of the 200 ft. of the final design. The shape of the contracting section seemed to be much more important in securing good flow conditions than that of the expanding section. With the shape of the contracting section as developed for the original design, various expanding shapes to the 120, 150 and 200 ft. stilling pool widths gave very similar results.

As in the case of the original design, experiments were made with the 18 possible combinations of gates open and closed. The wave heights along the side walls were observed with each of these combinations of gate openings. Measurements were also made with all the 18 possible combinations of gates open with all the possible combinations of the remaining gates partially open. Only one partial gate opening, 5.0 ft. in the prototype, was used. To have used all possible partially gate openings would have involved an infinite number of observations, and it is believed the one hundred and eleven combinations observed sufficiently cover the field. The conditions with several of the worst of those gate combinations are shown on Plate VIII. Figure 7 shows the height reached on the side walls of the flume by waves produced with various gate combinations on the final design, original design, and 60 ft. throat design. For the final design, the water surface along the side walls for the design discharge of 40,000 sec. ft. is also given. In order to facilitate comparison, a line is drawn at an elevation 15 ft. above the flume floor. All of the wave heights for the final design fell below this line but several waves of the original and 60 ft. throat designs reached above. As previously mentioned, a liberal factor of safety is necessary to estimate the wave heights from these observations on account of surface tension and entrained air effects.

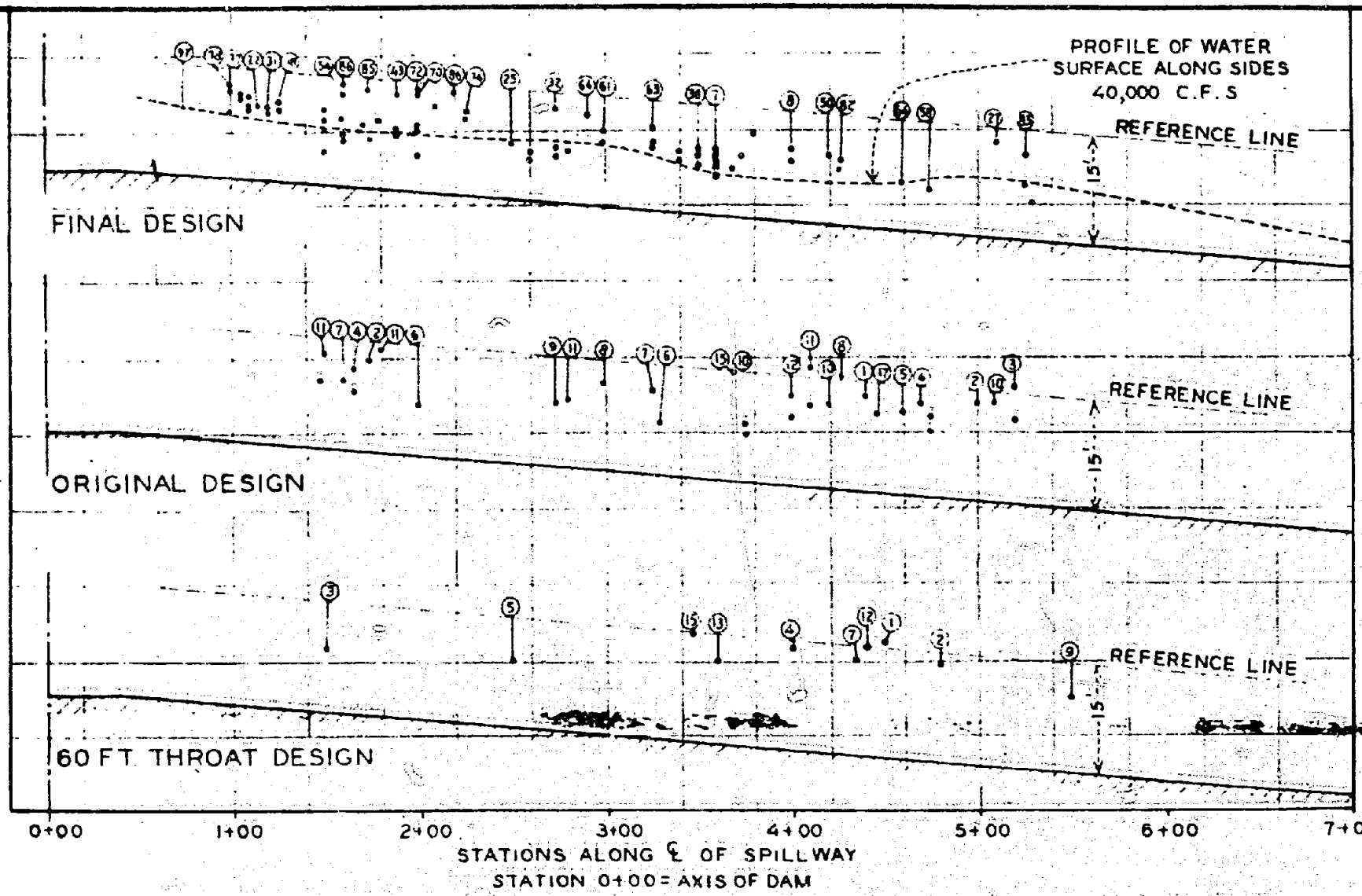
In order to permit a comparison of the conditions of flow in the model trough and those in the prototype, pictures were taken for 10,000 and 20,000 sec. ft. discharges under various conditions, since floods producing such discharges will occasionally occur, but the chance of a 40,000 sec. ft. flood is negligible. These are shown on



CLE ELUM SPILLWAY EXPERIMENTS
DEPTHS OF FLOW IN
FLUMES OF VARIOUS SHAPES

32-D-293

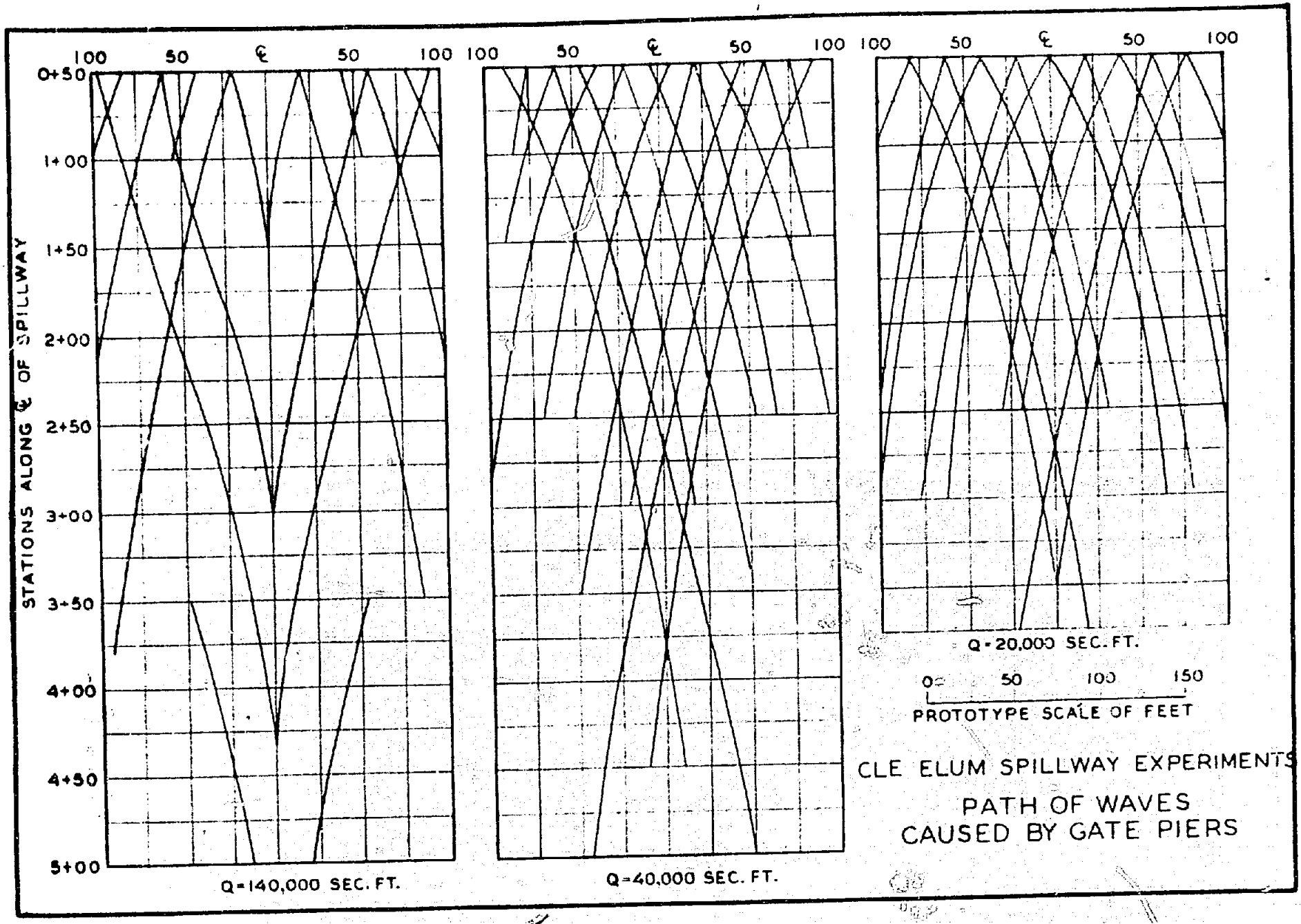
FIGURE 5



0 10 20 30 40
VERTICAL SCALE OF FEET FOR WAVE HEIGHT

CLE ELUM SPILLWAY EXPERIMENTS
HEIGHT OF WAVES AT SIDE WALLS
FOR DIFFERENT GATE COMBINATIONS

FIGURE 7

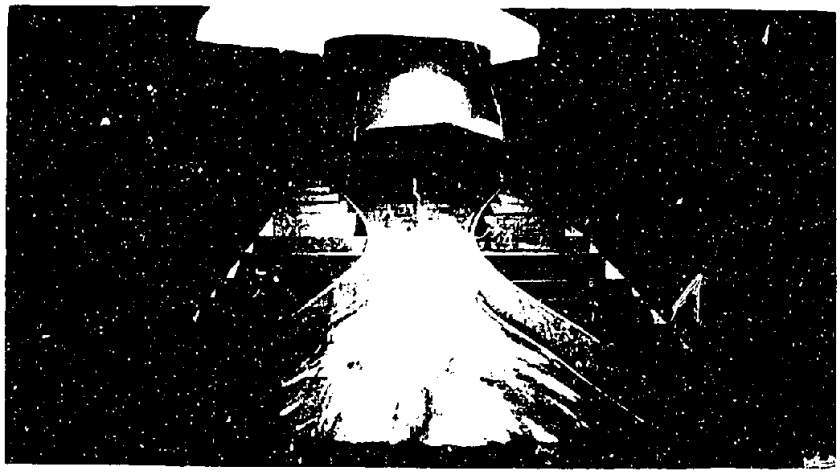


CLE ELUM SPILLWAY EXPERIMENTS

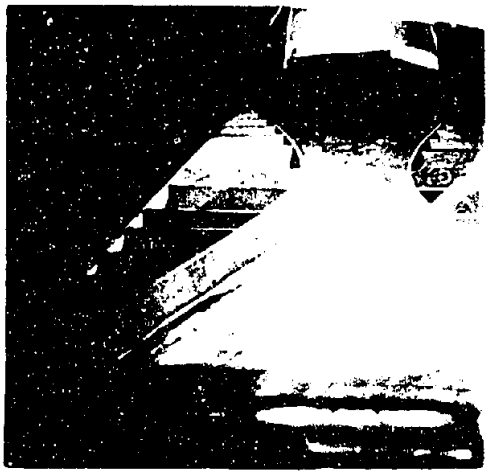
PATH OF WAVES
CAUSED BY GATE PIERS

FIGURE 8

a-40,000 SECOND FEET
ALL GATES OPEN

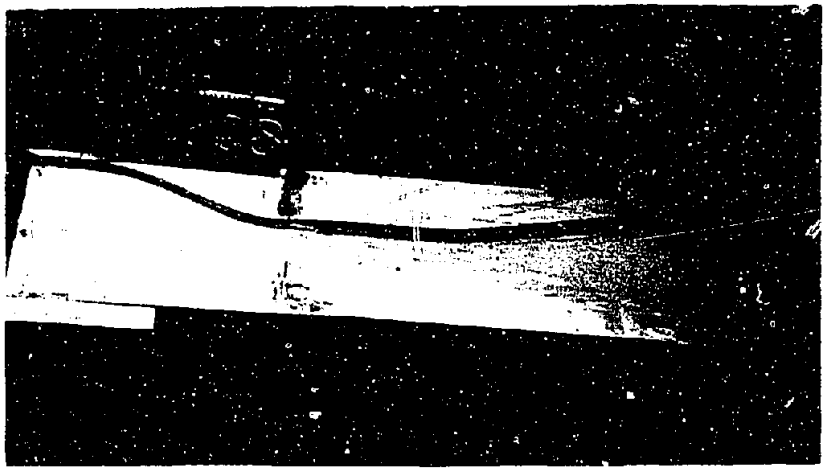


b-RESERVOIR AT MAXIMUM LEVEL
GATES 1,4 AND 5 OPEN

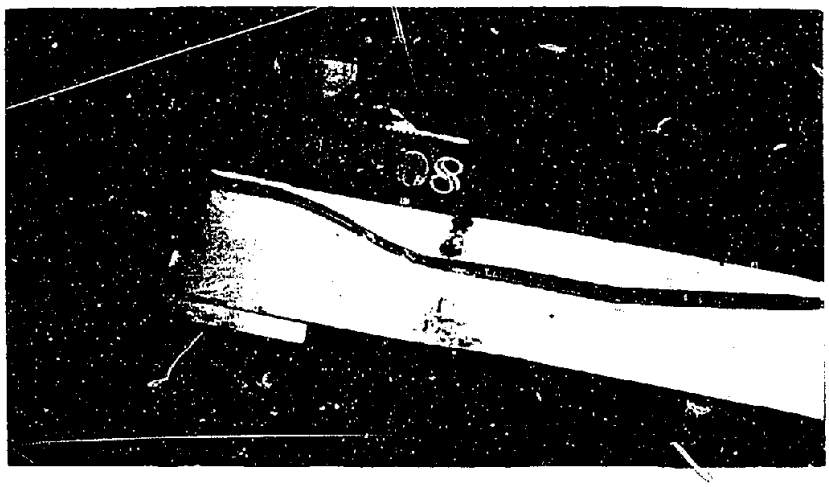


c-RESERVOIR AT MAXIMUM LEVEL
GATES [illegible] AND [illegible] OPEN

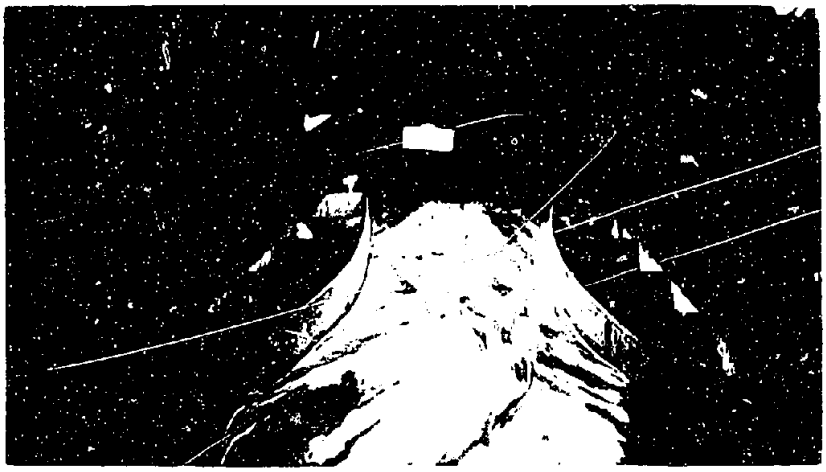
FLOW IN TROUGH [illegible]



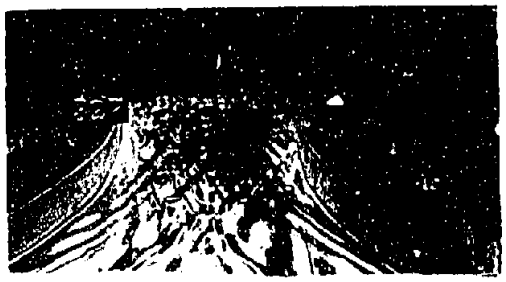
A. CONTROL SECTION AT THE THROAT



B. HYDRAULIC JUMP BETWEEN
CONTROL SECTIONS

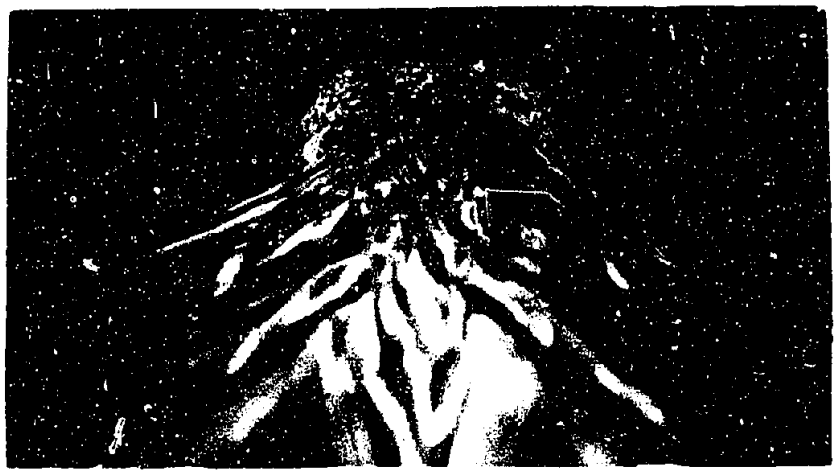


C. FINAL DESIGN WITH BAFFLE GATE
COMBINATION

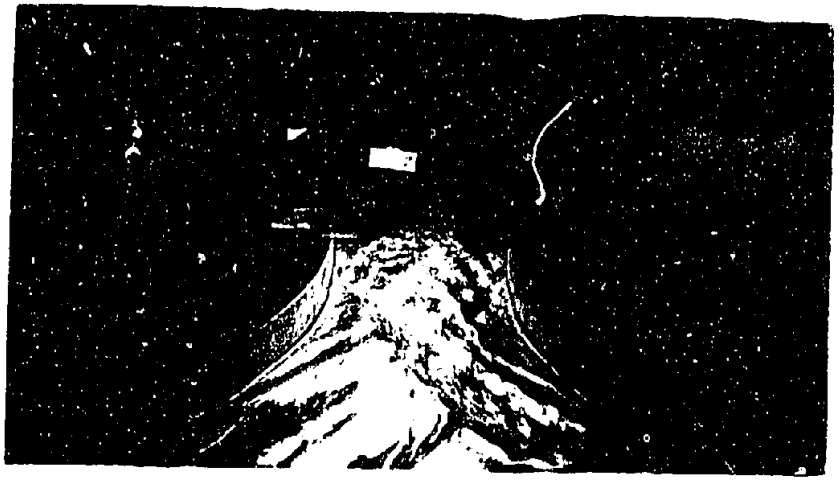


A-40,000 SECOND FEET
ALL GATES OPEN

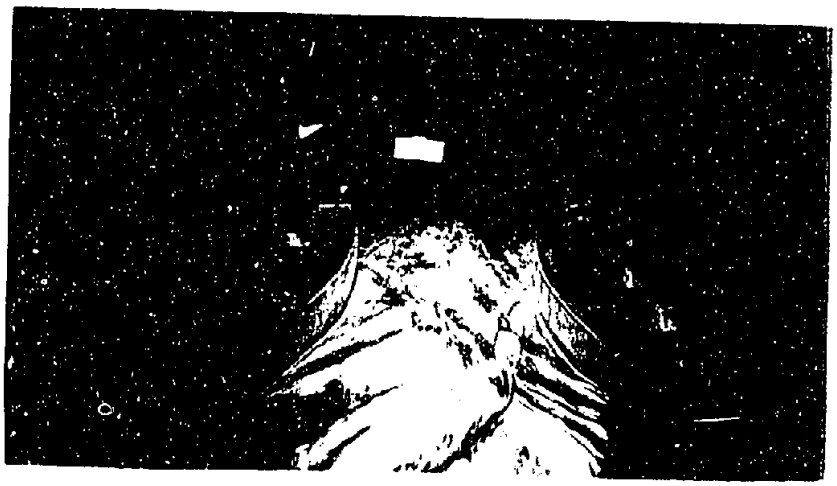
B-FINAL DESIGN 40,000 SEC FT FLOW
LOOKING UPSTREAM



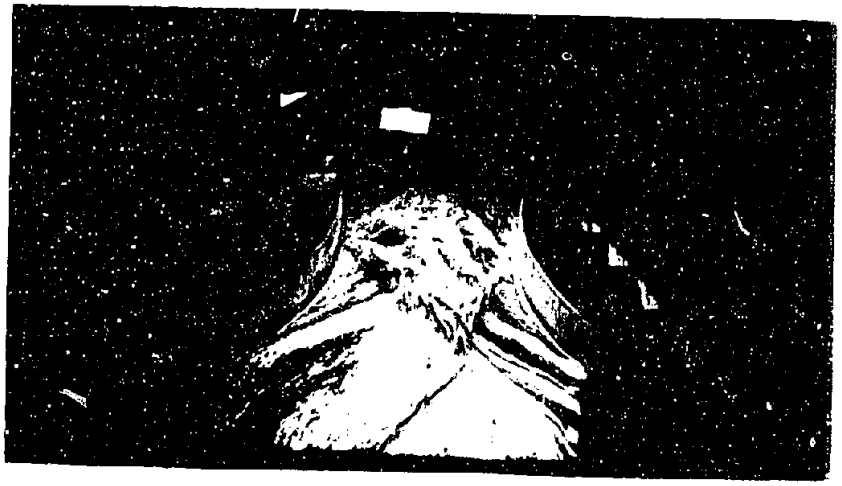
C-DESIGN [illegible]
STILLING POOL [illegible]
40,000 SECOND [illegible]



GATES [illegible] AND [illegible] WIDE OPEN.
GATES [illegible] AND [illegible] OPEN [illegible]

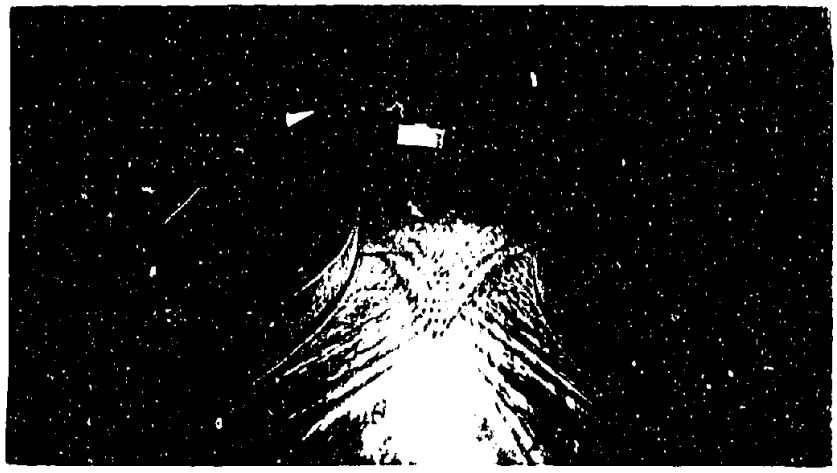


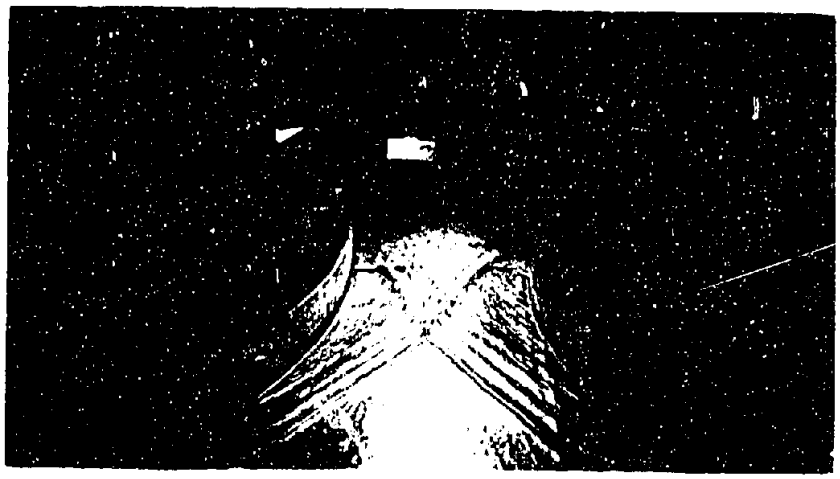
[illegible]

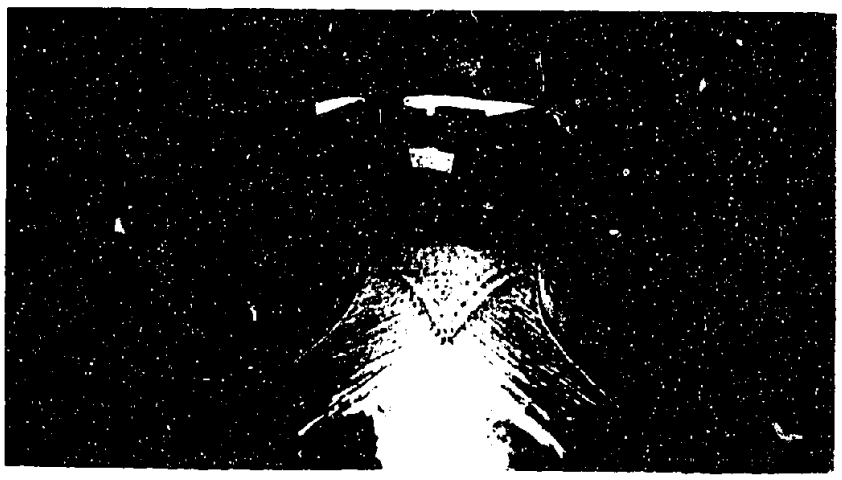


[illegible]

[illegible]
[illegible]







Plates IX and X. The first picture of each plate shows the conditions with all the gates entirely open. In order to obtain the greatest possible storage however, the gates may be set to keep the water to the maximum flow line by holding them partially open. As the best conditions of flow in the flume would result when all gates were open the same amount, it is probable that they would be set in this manner. The second picture on each plate shows the results for these conditions. It is intended to delay the installation of the piers and gates of this spillway until the irrigation demand reaches a magnitude which will require them. The flow conditions with no piers or gates are shown in the third picture.

Analysis of Wave Formations

The experiments showed that water is difficult to control when the flow is below the critical velocity, on account of the wave formations set up. That the ridges formed were waves and not currents of water was proven by introducing a solution of potassium per manganate into the streams. This color did not follow the waves at all, but flowed in the general direction of the flume. This is shown for the 1:250 model in Plate 50-A. Formations of such waves in depths below the critical has been observed by other experimenters (Hydraulic Laboratory Practice, Freeman, pages 126 & 169). In order to more fully understand the cause and action of these waves, an analysis was made of the waves set up by the gate piers in the flume with parallel sides, where conditions for analysis were favorable. It was assumed that those waves move at right angles to the flume, but the diagonal direction is given by the component due to the velocity of the water moving down the trough. The path of the ripples for three discharges were traced out as shown on Figure 8. The cross sectional area was measured at a number of points along the flume for each of three discharges, and from these areas and the discharge determined by the weir, the mean velocity at each cross section was computed. To reduce errors of observation, a curve of velocities along the flume was plotted, as shown on Figure 9 and from the velocities thus determined and the discharge, a mean depth curve was constructed. From the velocity and the angle between the path of the wave and direction of the flow of the water, the velocity of the wave moving across the flume was computed. In Figure 10 those velocities are plotted against the depth of flow at the point at which they occurred. The formula of the velocity of wave travel in water is $V = \sqrt{g D}$, where V is the velocity, g is the acceleration due to gravity and D the depth of the water. The relation of the cross velocity V to the $\sqrt{g D}$ is shown for a discharge of 40,000 second feet to be approximately 0.75 $\sqrt{g D}$ and for a discharge of 20,000 second feet, approximately 0.70 $\sqrt{g D}$. Experiments by Whitney M. Forland on models of beaches have shown that waves in very shallow water do not reach velocities equal to $\sqrt{g D}$. His results were

comparable with those shown in Figure 10 for the 20,000, & 40,000 discharges. In order to test greater depths a run was made at the maximum discharge capacity of the apparatus, which corresponded to a flow of 140,000 second feet. This showed (Figure 10) velocities practically equal to $\sqrt{gD}$. The variation of the observed results at various points in any run is no doubt partly due to the fact that the mean velocities and depths in the cross sections were used in the computations instead of the actual depths and velocities at each point, as the latter were not observed.

It must be admitted that the assumption that the waves set up by the piers move at right angles to the direction of flow in the flume does not rest on a very secure basis. No particular reason is evident why the impulse due to the piers might not set up waves at some other angle. However as the results worked out on the basis of the right angle flow give velocities which agree with those of Borland for the shallow depths and with the theoretical velocities at greater depths, the assumption made seems to be sufficiently established to form the basis of a working hypothesis.

As the depth in the prototype would be sufficient to cause waves with velocities equal to $\sqrt{gD}$ and in the model at the design capacity reached only 0.75 $\sqrt{gD}$, the results of this study throw some doubts on the sufficiency of the determinations of the best shape of flume. Since slight changes in the flume shape, especially in the contracting section make considerable difference in the flow conditions, the difference between the cross velocity in the model equal to 0.75 x $\sqrt{gD}$ and those in the prototype which would probably be equal to the $\sqrt{gD}$ might be sufficient to somewhat alter the conditions of flow in the flume. This indicates the desirability of conducting model experiments of this portion of the flume on sufficient scale to give cross velocities for the waves equal to the $\sqrt{gD}$. Such models might be cheaply constructed at the laboratory on the Uncompahgre Project.

Stilling Pool Experiments

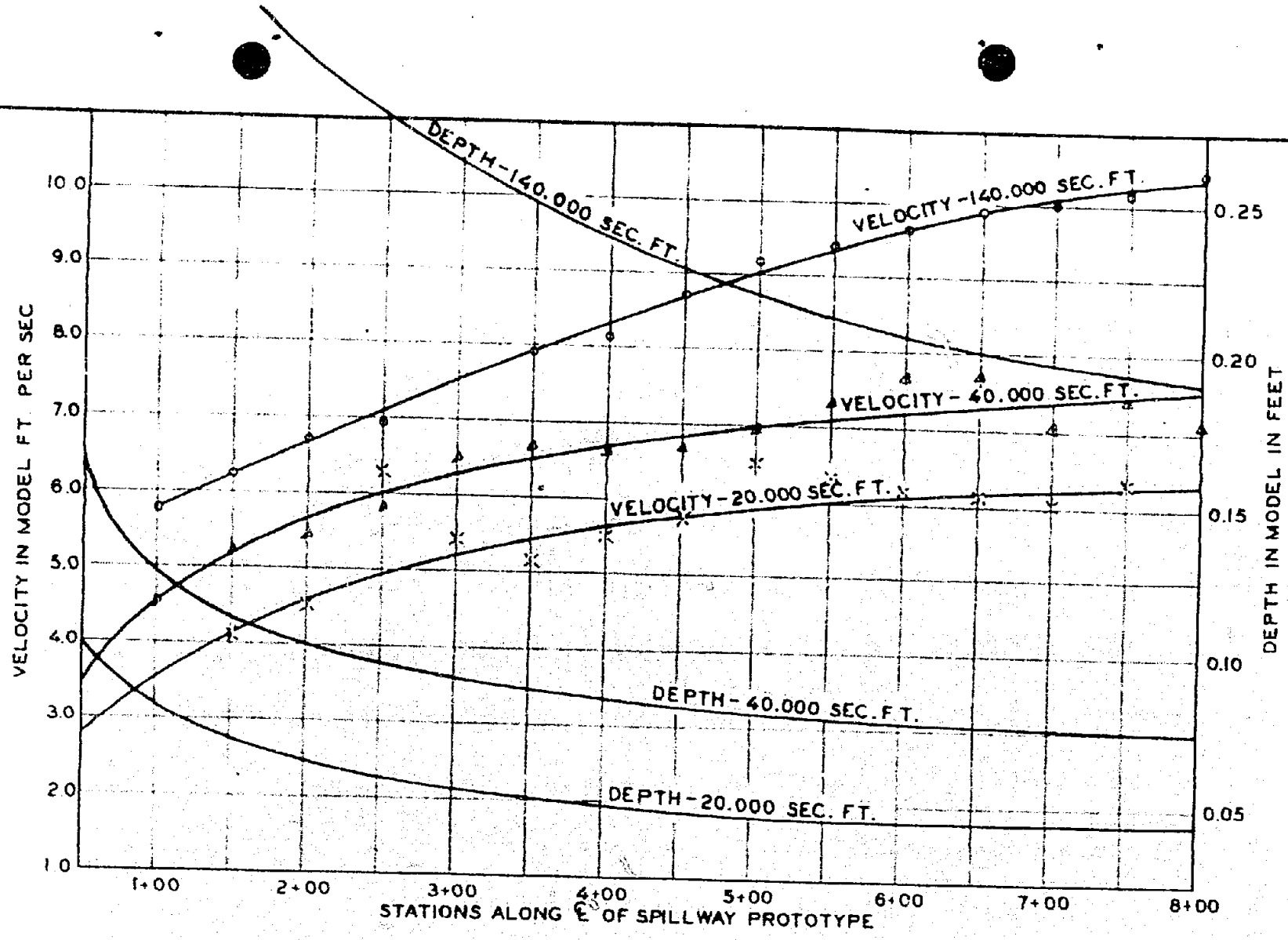
To dissipate the energy of the water at the bottom of a spillway one of two following devices or a combination of them in ordinarily used. (1) hydraulic jump pool and (2) piers or baffles. Only the first of these was tested as the second was believed to be unsuitable for the foundation conditions existing at Cle Elum.

The hydraulic jump pool will be the most expensive part of the Cle Elum spillway. With any practical width of the flume the side walls of the pool must be high and costly retaining walls. The floor must be of thick concrete in order to withstand the vibration from the very turbulent water in the hydraulic jump and to resist the un-

balanced upward pressure which a jump produces. In the trough section the excavation will add little to the cost for it will be used in the dam. The excavation from the pool however will be largely from below ground water level, and will require extensive pumping on account of the porous composition of the soil. It is evident therefore that the greatest savings which could be made by hydraulic model tests would be in the design of the stilling pool, and extensive experiments were undertaken on this portion of the spillway. The effectiveness of the stilling pool was determined by means of the scour produced in a bin of sand downstream from the solid portion of the model which represented the concrete floor and walls of the jump pool. The sand was screened river sand, largely composed of quartz. A mechanical analysis curve of it is given in Figure 11. The first part of the tests was made with a bin whose sides formed a continuation of the sides of the stilling pool. In order to observe and photograph the scour, the nature of the currents formed, and the hydraulic jump, one of the sides of the stilling pool and sand bin was formed of plate glass. The width of the pool and sand box was 4.0 ft., corresponding to 200 ft. in the prototype. The bottom of the sand bin corresponded to elevation 2074 and was about 23 ft. below the usual level of the bottom of the stilling pool. This depth was found to be amply sufficient. The lower end of the sand box was 275 ft. downstream from the junction of the pool floor and the entrance slope. As the length of the pool floor varied, the length of the sand bin was also variable, ranging from 96 to 301 ft.

Very valuable results were obtained with this form of sand bin but it had the disadvantage that the vertical non-erodable sides of the sand bin did not exactly represent the conditions which would exist in the prototype, as the banks downstream from the concrete stilling pool would be sloping and subject to erosion. It was found that the vertical sides gave rise to eddies and whirls along them which resulted in more severe scour along the sides of the pool than was likely to occur in the prototype. This scour is noticeable in many of the pictures showing the sand in this type of bin after a run, taking the form of a small channel or valley at the foot of the vertical glass wall. The action is believed to be set up by the restraining effect of the walls on eddies. Away from the walls similar eddies rotate harmlessly in the surrounding water, but when they come in contact with the walls, they cannot continue to rotate and are partially diverted downward, coming in contact with the sand bottom and causing more scour than would normally occur.

In order to be free from this effect and also to investigate the scour around the ends of the stilling pool walls, a larger sand bin was constructed in which the sloping earth banks of the prototype below the stilling pool could be more accurately reproduced and eliminated the false indications caused by the vertical side walls.



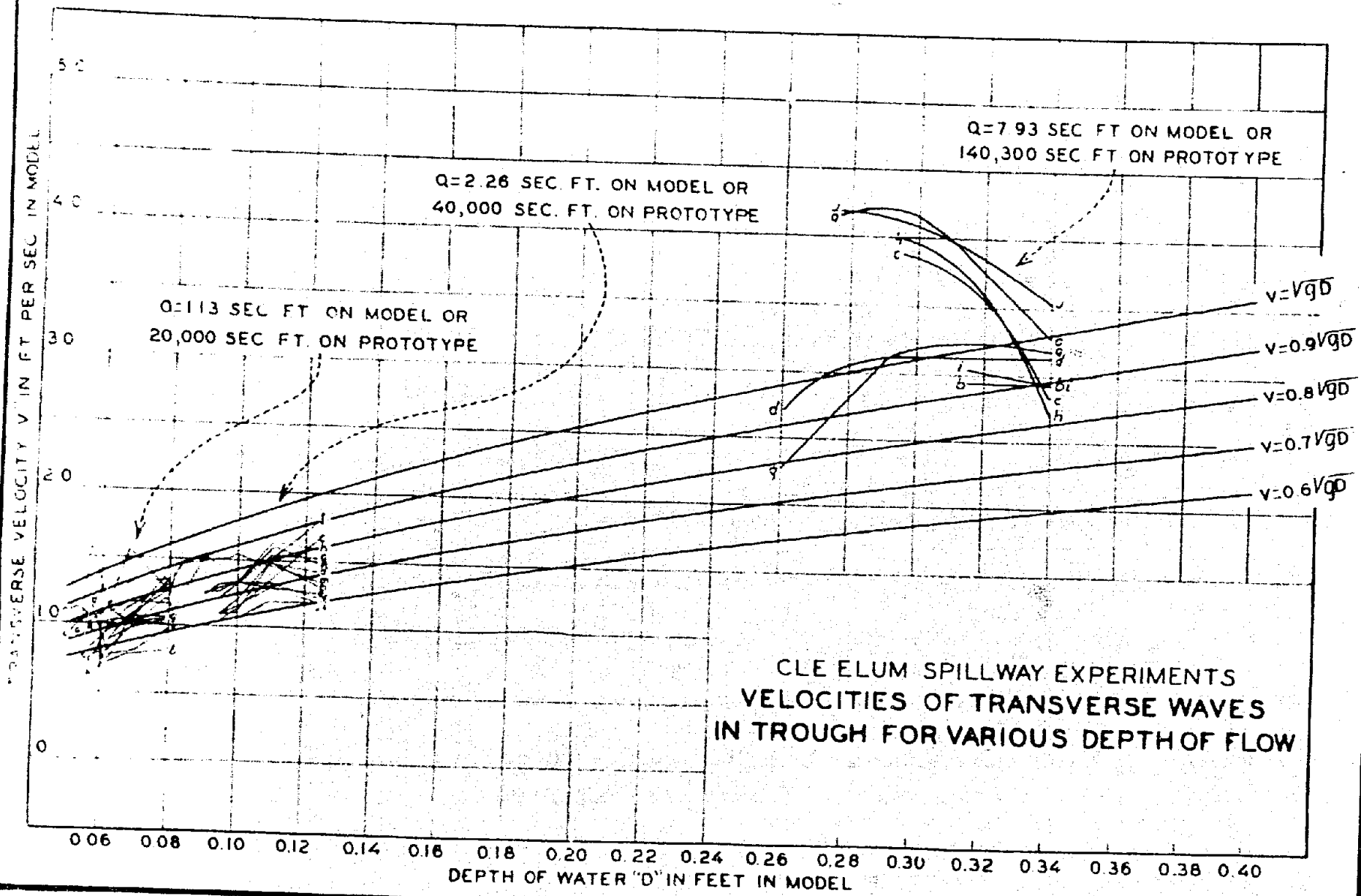
CLE ELUM SPILLWAY EXPERIMENTS

DEPTH AND VELOCITIES OF FLOW

TROUGH WITH PARALLEL SIDES

200 FT. WIDE

FIGURE 9



CLE ELUM SPILLWAY EXPERIMENTS
VELOCITIES OF TRANSVERSE WAVES
IN TROUGH FOR VARIOUS DEPTH OF FLOW

FIGURE 10

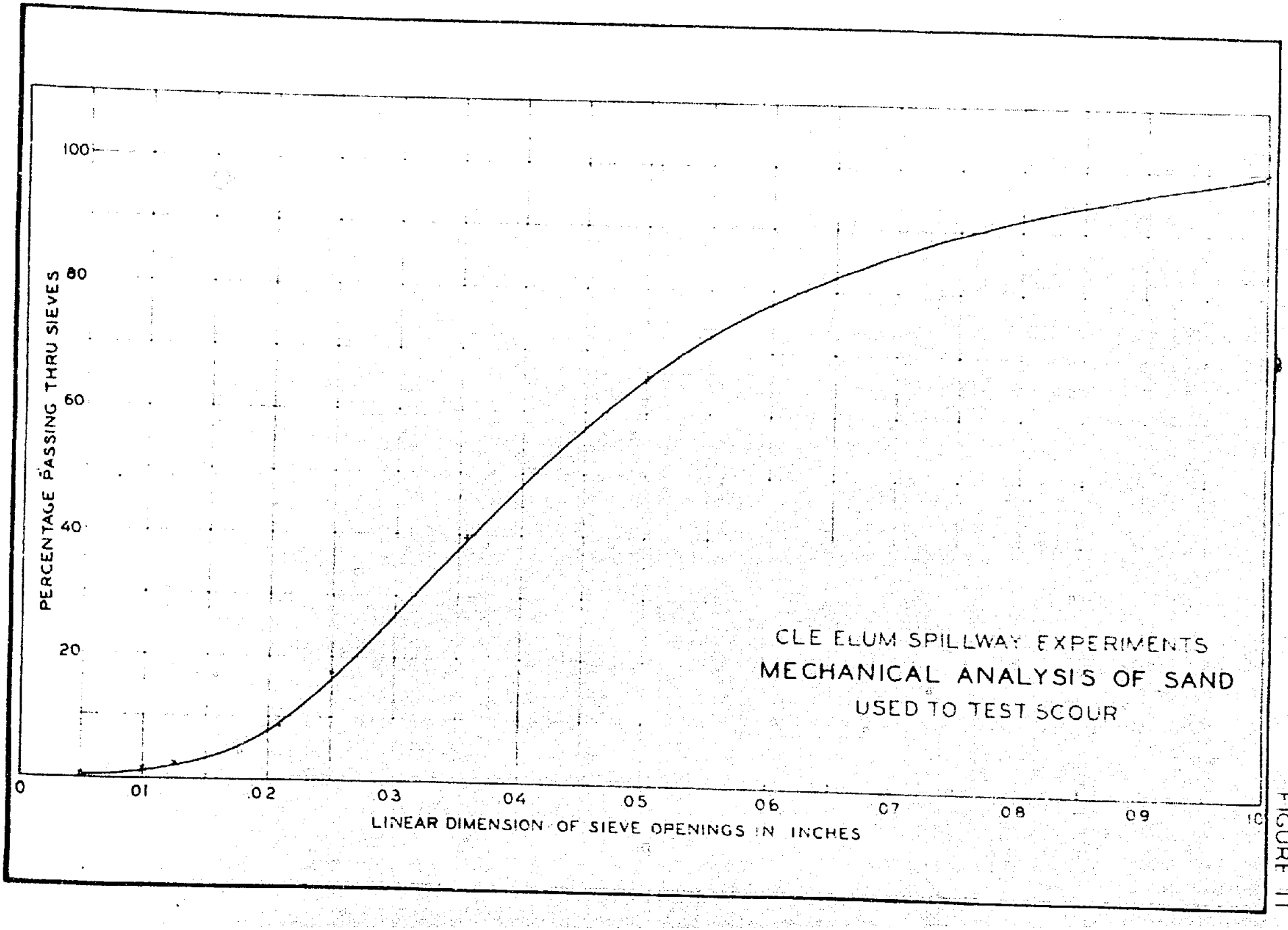
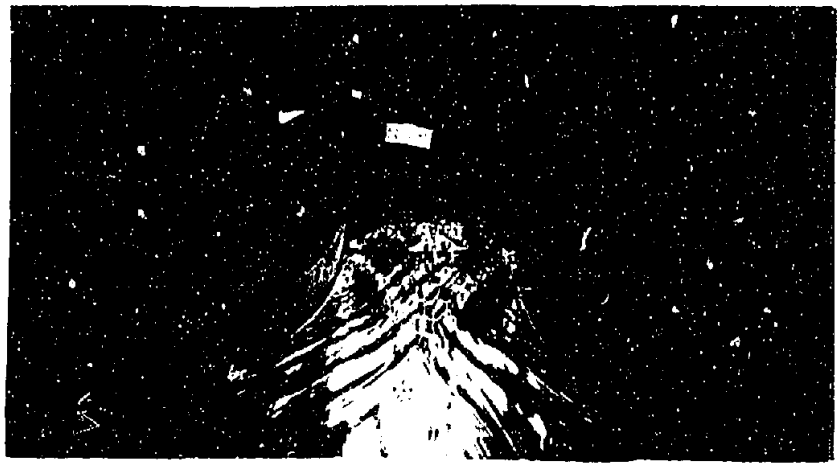
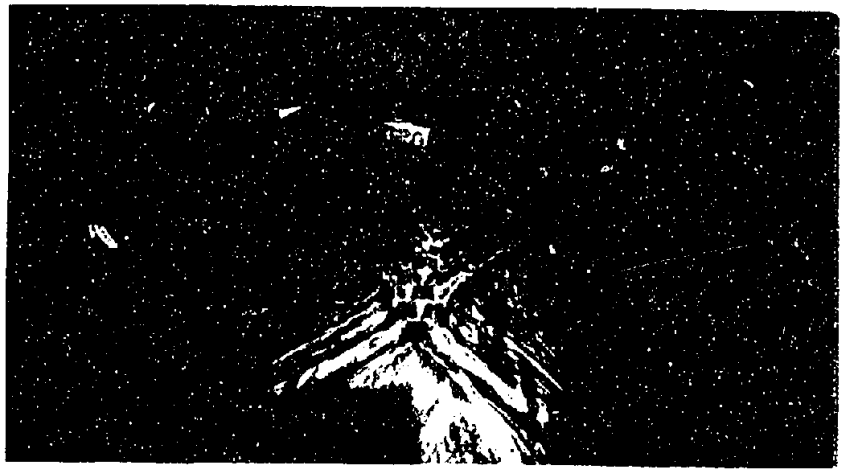

CLE ELUM SPILLWAY EXPERIMENTS
MECHANICAL ANALYSIS OF SAND
USED TO TEST SCOUR
PERCENTAGE PASSING THRU SIEVES
100
80
60
40
20
0
.01
.02
.03
.04
.05
.06
.07
.08
.09
.10
LINEAR DIMENSION OF SIEVE OPENINGS IN INCHES

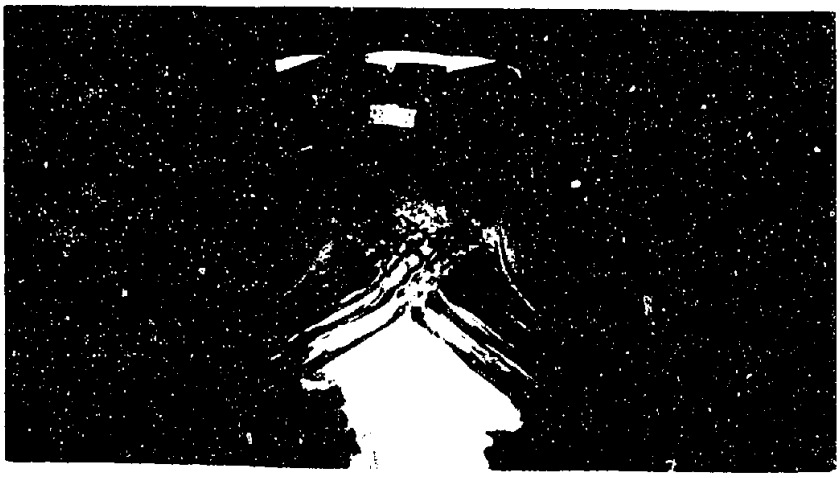
FIGURE 11



A. ALL GATES WIDE OPEN

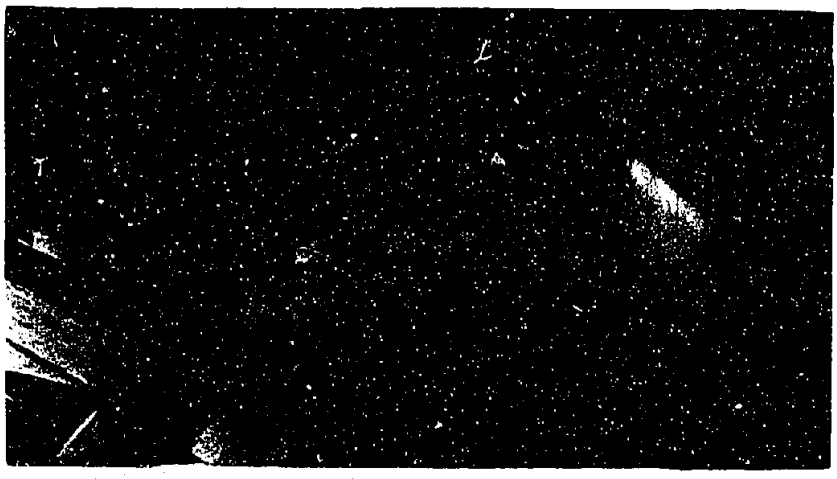


B. RESERVOIR AT MAXIMUM [illegible]
ALL GATES OPEN THE SAME AMOUNT



C. [illegible] GATES OF RIGHT

FLOW IN FINAL DESIGN THROUGH 20,000 SEC FT



SAND WAVE FORMED DURING
EARLY PART OF RUN

FORMATION OF SAND BAR BEYOND STILLING POOL

The results obtained with this sand box are believed to be a much more accurate representation of what would take place in the prototype than those in the bin with vertical walls.

The length of both of these bins was too short to obtain a perfect picture of the scour produced. When water started to flow through the spillway, it scoured first at the lower edge of the spillway pool, rolling the sand back in the form of a sand wave, similar to those formed in a river (See Plate XI). This continued to move downstream until it came within the influence of the weir controlling the tailwater level. Part of the sand was lifted over the weir and the remainder piled up against it. If the sand box had been longer, the greater part of the sand forming the wave would have come to rest in a bar similar to that in Plate XI across the channel a short distance further downstream than the tailwater weir, where the boils and eddies had subsided. The result of the formation of such a bar slightly further downstream than the tailwater weir would differ little from that produced by the weir used, and it is therefore believed that the results obtained are sufficiently close for practical purposes.

Consistency of Scour Test Results

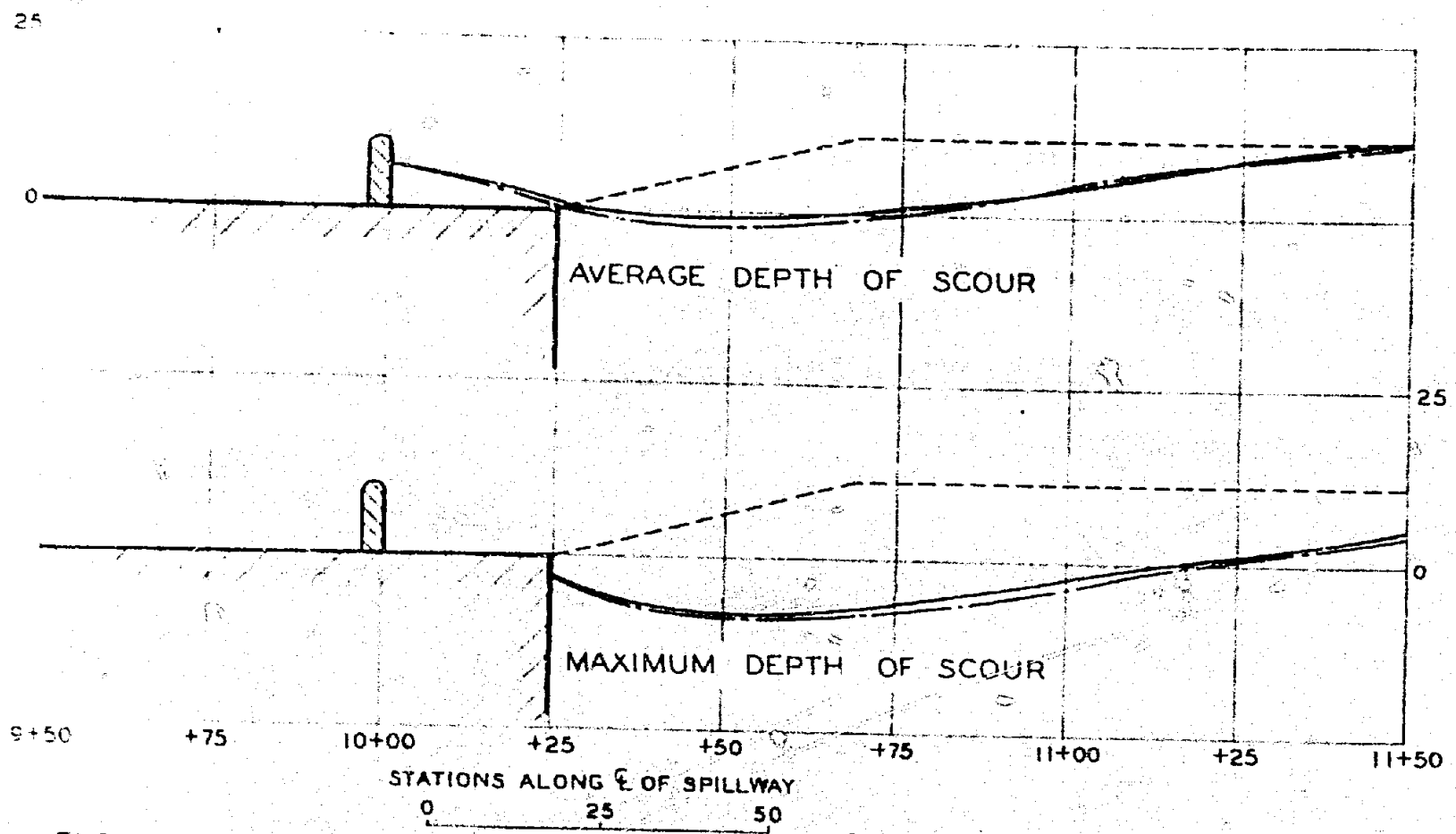
In the minds of some there may be a doubt as to how consistent results could be obtained in a scour pool, as used in these experiments. When the work was started it was not certain that the results obtained in one run could be closely duplicated in another. Unless results could be so duplicated the scour pool might lead to unwarranted conclusions. In order to throw light on this point several scour runs were made. The first of these consisted of two runs made under nearly identical conditions. At the time of these tests the effect of the condition of compactness of the sand was not appreciated and no especial effort was made to secure identical conditions of the sand in the bin. The results of these are shown in Figure 12. This diagram shows the level of the sand in the sand box before the run and the average elevation at any cross section after the run. It also gives a line representing the maximum depth of scour, which was drawn by plotting at the location of each cross section the elevation of the lowest point in that section, and joining these plotted points by a line. On account of the effect of the vertical walls before mentioned, the weight which can be attached to this maximum depth is uncertain. The results of the tests given in Figure 12 show a very close agreement, indicating that, within amply close limits, the results obtained on one test can be duplicated in a repetition of that test, even if the sand conditions are not exactly duplicated. It is thought that the slight difference which existed between the results of the two runs shown on Figure 12 may have been due to a different degree of compactness of the sand. A striking example of consistency

of the scour tests is shown on Plates XXXVIII to XL, where the effect of equal increments of change in pool floor length showed consistent changes in scour.

In order to determine how far differences in compactness of the sand could influence the results, three runs were made. In one of these the sand was put in damp and as loosely as possible. In another it was put in tightly packed and in a third it was placed in the manner ordinarily used. The results are shown in Figure 13. They show very little difference between the runs with the sand as ordinarily placed and that tightly packed. Somewhat more difference exists between the results of these two runs and that for the loosely placed sand. It is believed that this difference is due to the bulking of the loosely placed damp sand, which could be seen to settle considerably when the water was admitted to the sand pool. In the ordinary setup most of the sand in the pool had been settled by water from the previous run, and only a relatively small amount of sand was added. Ordinarily this was somewhat compacted and therefore settled little, but even if placed as loosely as possible the surface would not settle appreciably. It is therefore believed that although the results of scour runs may differ very slightly due to difference in compactness of the sand, this difference would ordinarily be too small to be considered, and under the worst possible conditions would not lead to serious errors.

Developing Standard Procedure.

One of the first questions arising in the use of the sand bin for determining scour was the length of runs necessary. If short runs were made they might not give the results which would take place in the prototype in a flood. The time ratio between the prototype and the model is $\sqrt{50} : \sqrt{1}$, or 7.071:1. A half hour run on the model would therefore represent only about 3½ hours on the prototype. To get runs of duration corresponding to a large flood require such a length as to seriously retard the progress of the tests. A run was therefore made to determine the minimum length which could be used. This was of two hours duration, measurements being made at the end of each 30-minute period to determine the progress of the scour. The results are shown on Figure 14. It was found that most of the scour took place in the first 30 minutes and that very little change occurred after 60 minutes. It was therefore decided to use 60-minute runs. The above tests were made in the pool with the glass side. On the final design a test of 5 hours duration, corresponding to over 35 hours on the prototype was made. Figure 15 shows a profile down the center of the channel below the stilling pool after runs of various duration. Although some movement of the bottom was still going on at the end of the test, erosion near the stilling pool, where it would be dangerous, was negligible.



EXPLANATION

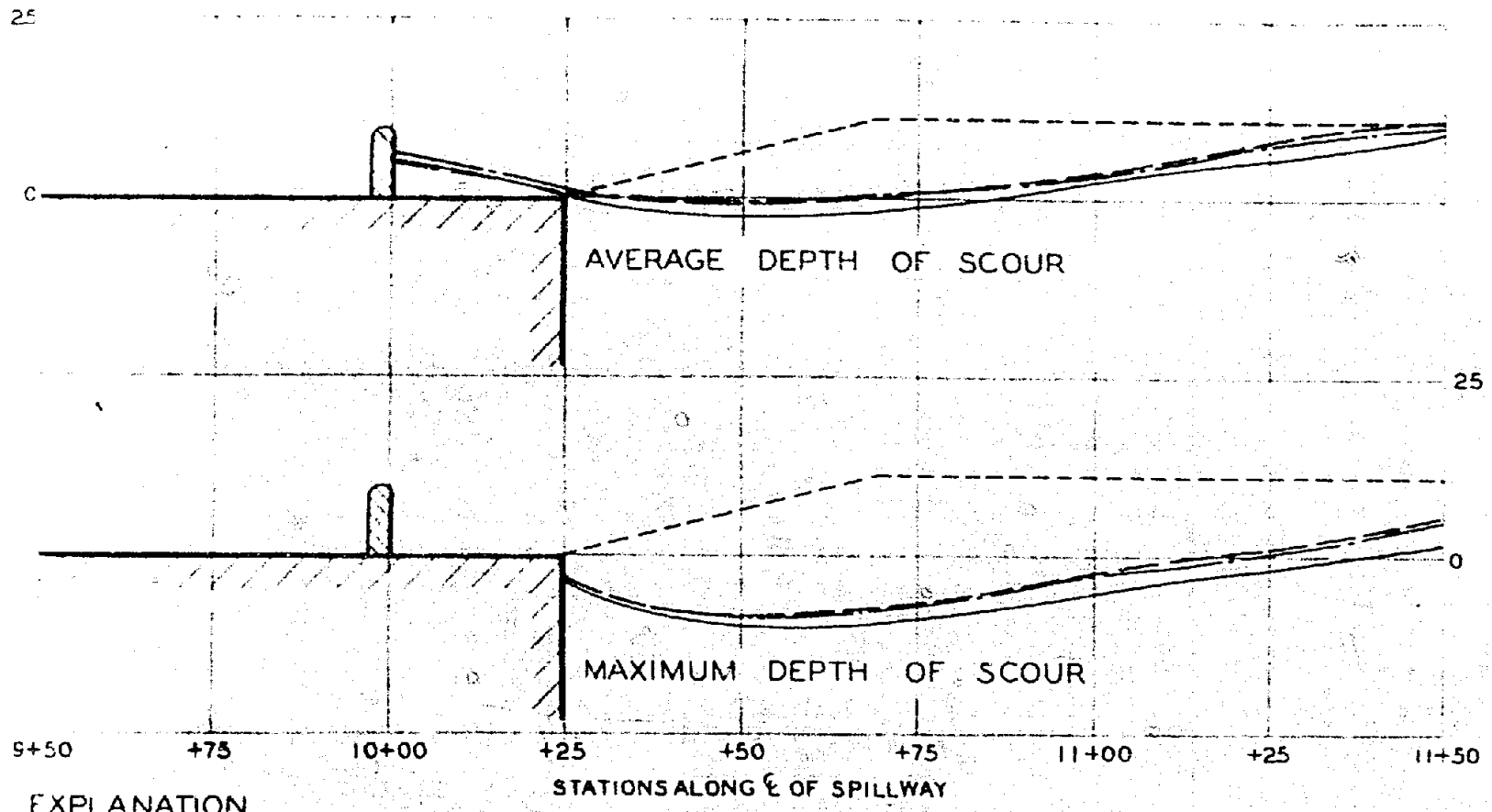
- - - - - - - SAND BEFORE RUN

—— · —— RUN NO. 1

———— RUN NO. 2

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
COMPARISON OF RESULTS OF TWO IDENTICAL RUNS
TROUGH SLOPE 4:1

FIGURE 12



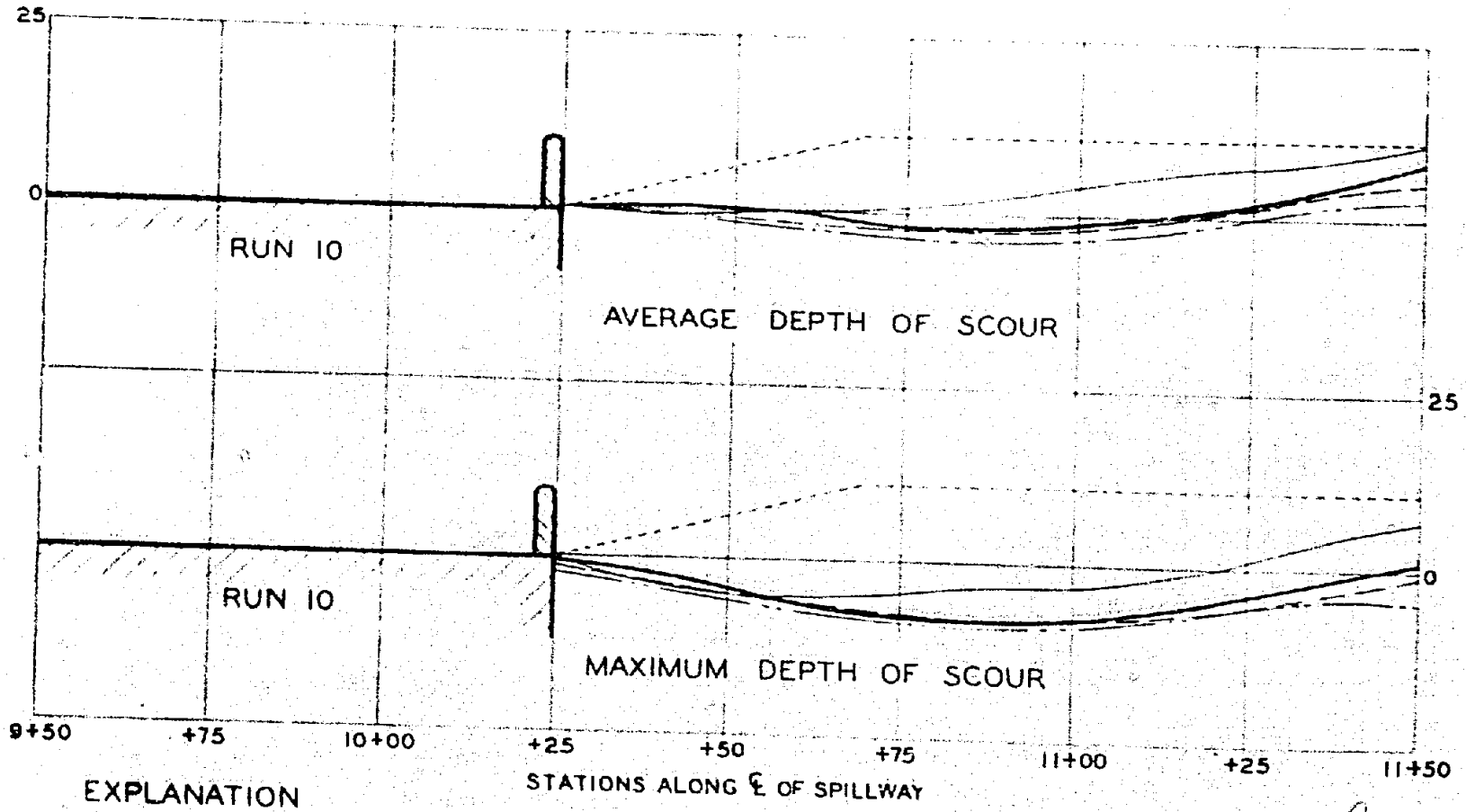
EXPLANATION

- - - - - - SAND BEFORE RUN
——— SCOUR LOOSELY PACKED SAND
— — — SCOUR NORMALY PACKED SAND
—·—·— SCOUR TIGHTLY PACKED SAND

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF SAND CONDITIONS
TROUGH SLOPE 4:1

FIGURE 13



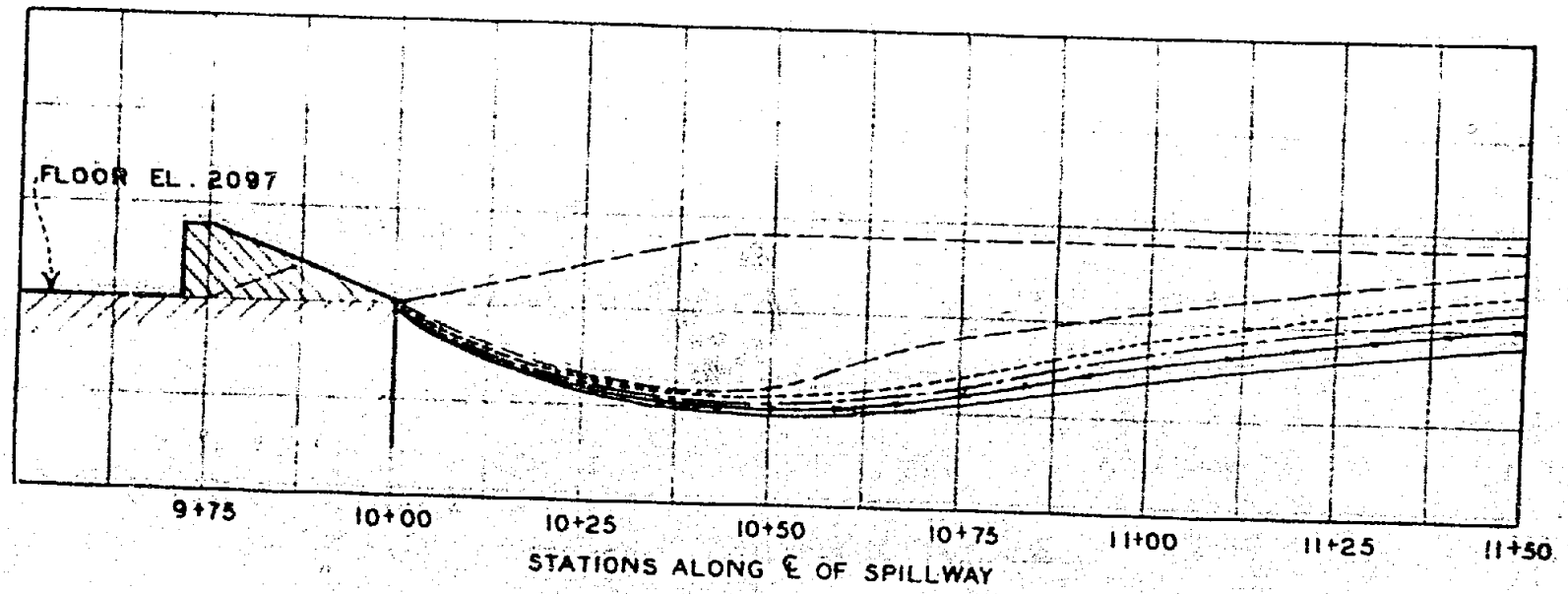
EXPLANATION

- SAND BEFORE RUN
- DEPTH 30 MIN. RUN
- DEPTH 60 MIN. RUN
- DEPTH 90 MIN. RUN
- DEPTH 120 MIN. RUN

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF LENGTH OF RUN

FIGURE 14



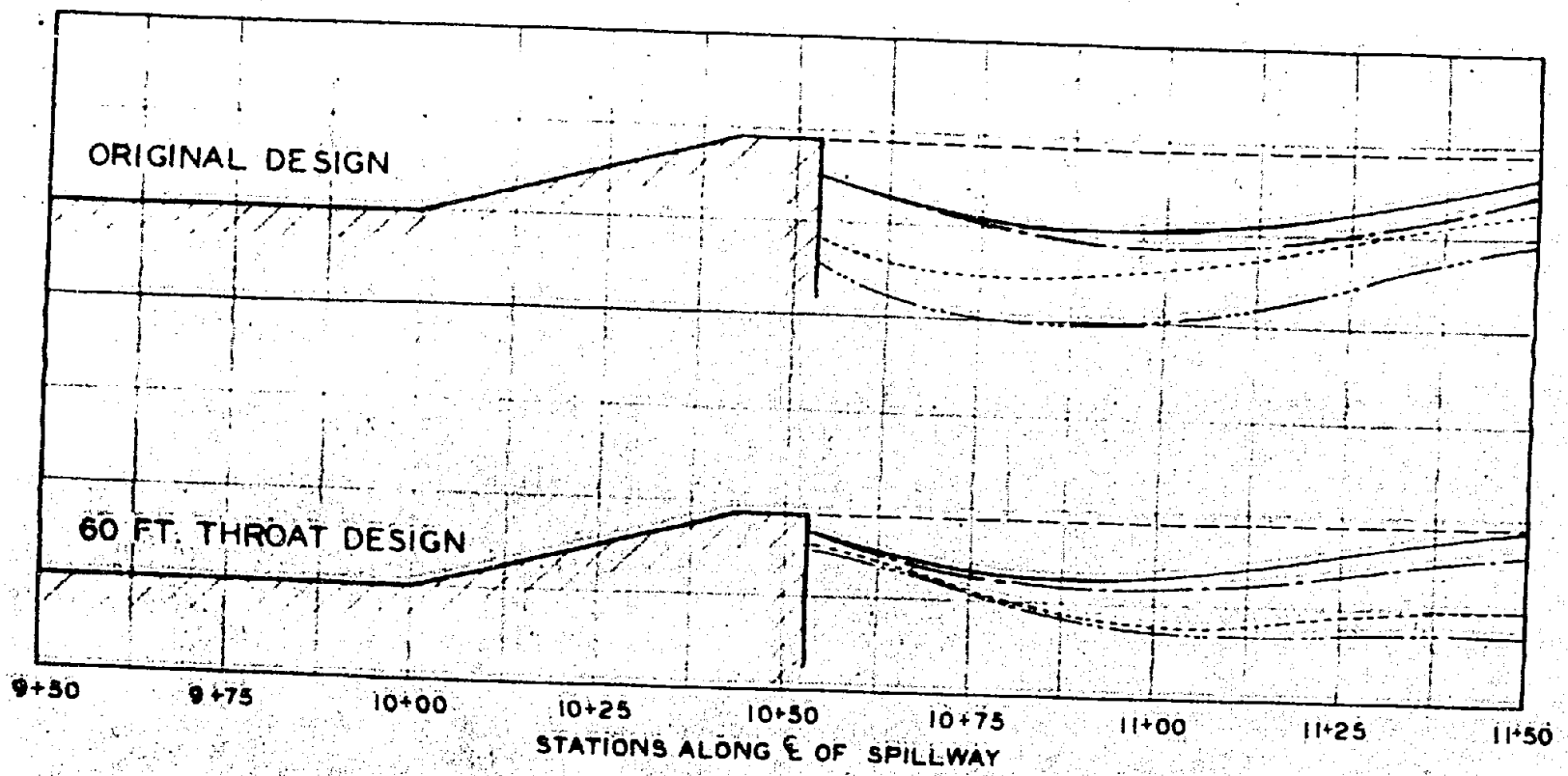
EXPLANATION

- WATER RUN 60 MIN
- WATER RUN 120 MIN
- WATER RUN 180 MIN
- WATER RUN 240 MIN
- WATER RUN 300 MIN.

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF RUNS OF VARIOUS LENGTHS
ON THE FINAL DESIGN

FIGURE 15



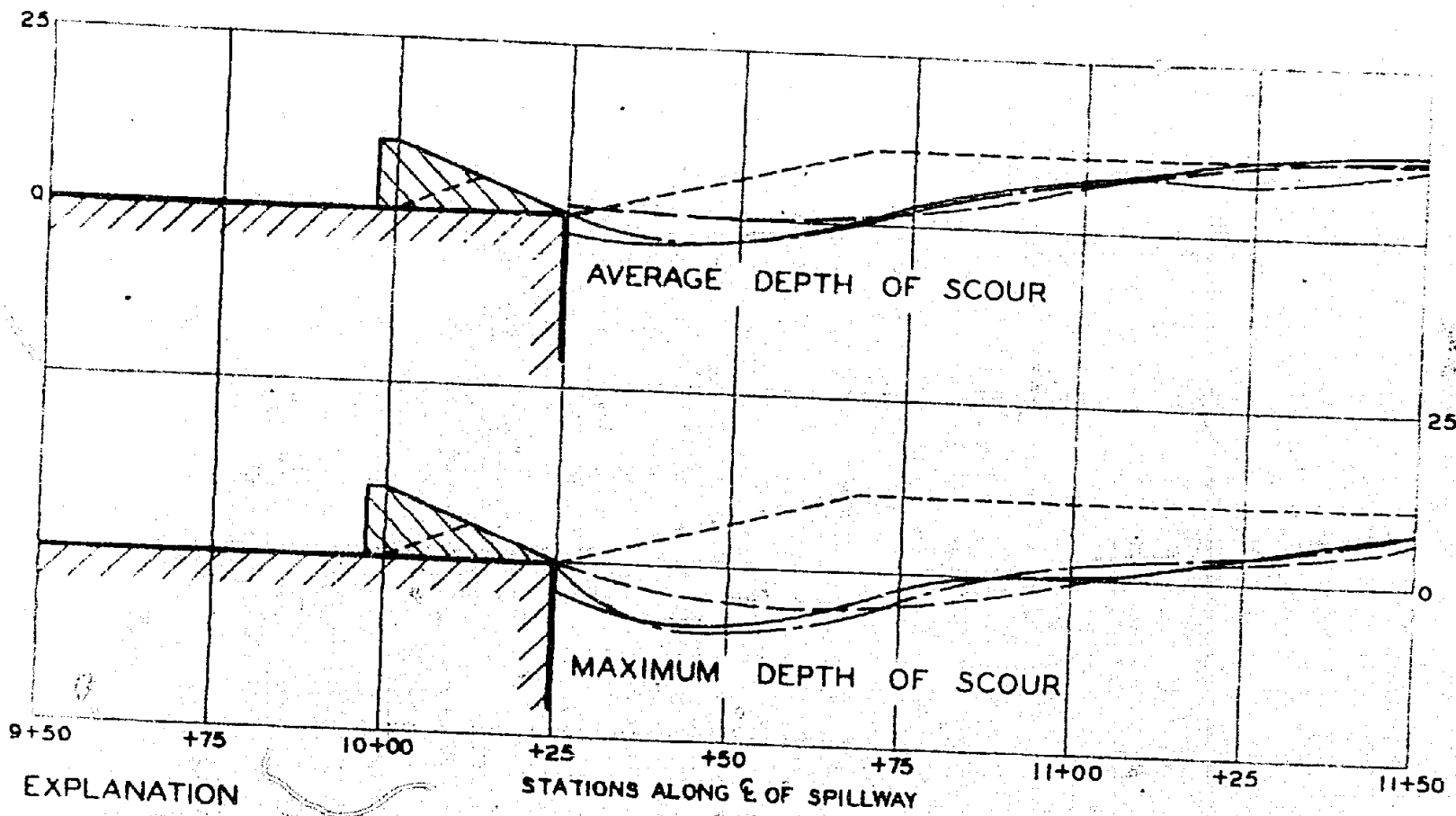
EXPLANATION

- SAND BEFORE
- AVERAGE 30 MIN RUN
- AVERAGE 60 MIN RUN
- MAXIMUM 30 MIN RUN
- MAXIMUM 60 MIN RUN

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF TROUGH SHAPE
ORIGINAL POOL DESIGN

FIGURE 16



EXPLANATION

- - - - - - SAND BEFORE RUN
——— STRAIGHT SIDES - 200 FT. WIDTH
— — — CURVED SIDES - 100 FT. THROAT
—·— CURVED SIDES - 80 FT. THROAT

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF TROUGH SHAPE
DENTATED SILL - LEVEL BOTTOM

FIGURE 17

The shape of the trough was found to have considerable effect on the scour which would take place below the stilling pool. An extreme case of this is shown in Figure 16, which gives the scour with the original design of flume as compared with that developed for a 60 ft. throat. Figure 17 shows comparative runs made with the 60 and 100 ft. throat widths and with the flume with parallel sides (i.e.) 200 foot width throughout. This shows appreciable difference in the results secured with the different flume sections, the best one being the 100 ft. throat adopted for the final design. These experiments show that rigid comparisons cannot be made of the scour resulting from various conditions in the stilling pool where different flume shapes were used. For this reason all comparisons of stilling pool results in this report, unless otherwise stated, were made with the same type of flume.

Best shape of Floor at End of the Pool

The stilling pool of the original design had a level bottom approximately 100 ft. in length at an elevation 32 ft. below the tailwater level for a 40,000 sec. ft. discharge, which was the computed depth necessary to keep the hydraulic jump within the stilling pool. At the end of the level portion of the pool bottom, the floor sloped upward on a 4:1 slope for 44 ft. to the elevations of the bottom of the leadoff channel, beyond which point the floor was level for 6 ft.

Experiments made with this form of floor showed considerable scour. Plate XII shows the result using the flume as originally designed and Plate XIII using that with the 60 ft. throat. In both cases the sand at the downstream edge of the pool floor was removed to a considerable depth, and in the prototype would have exposed the cutoff wall beneath. The results of these experiments are shown quantitatively on Figure 16. This was an undesirable condition and an attempt was made to reduce the scour by placing a dentated sill at the top of the slope. The dentated sill was invented and patented by Dr. Rohbeck of Karlsruhe, Germany, and has been considerably used as a means of protecting foundations from scour. The result of this test is shown on Plate XIV. It shows that although the scour was reduced somewhat it was still severe. The depth of scour is shown on Figure 18 (Run 2). Substituting a plain sill for the dentated one gave somewhat less desirable scour as shown on Plate XV and Figure 19, Run 3.

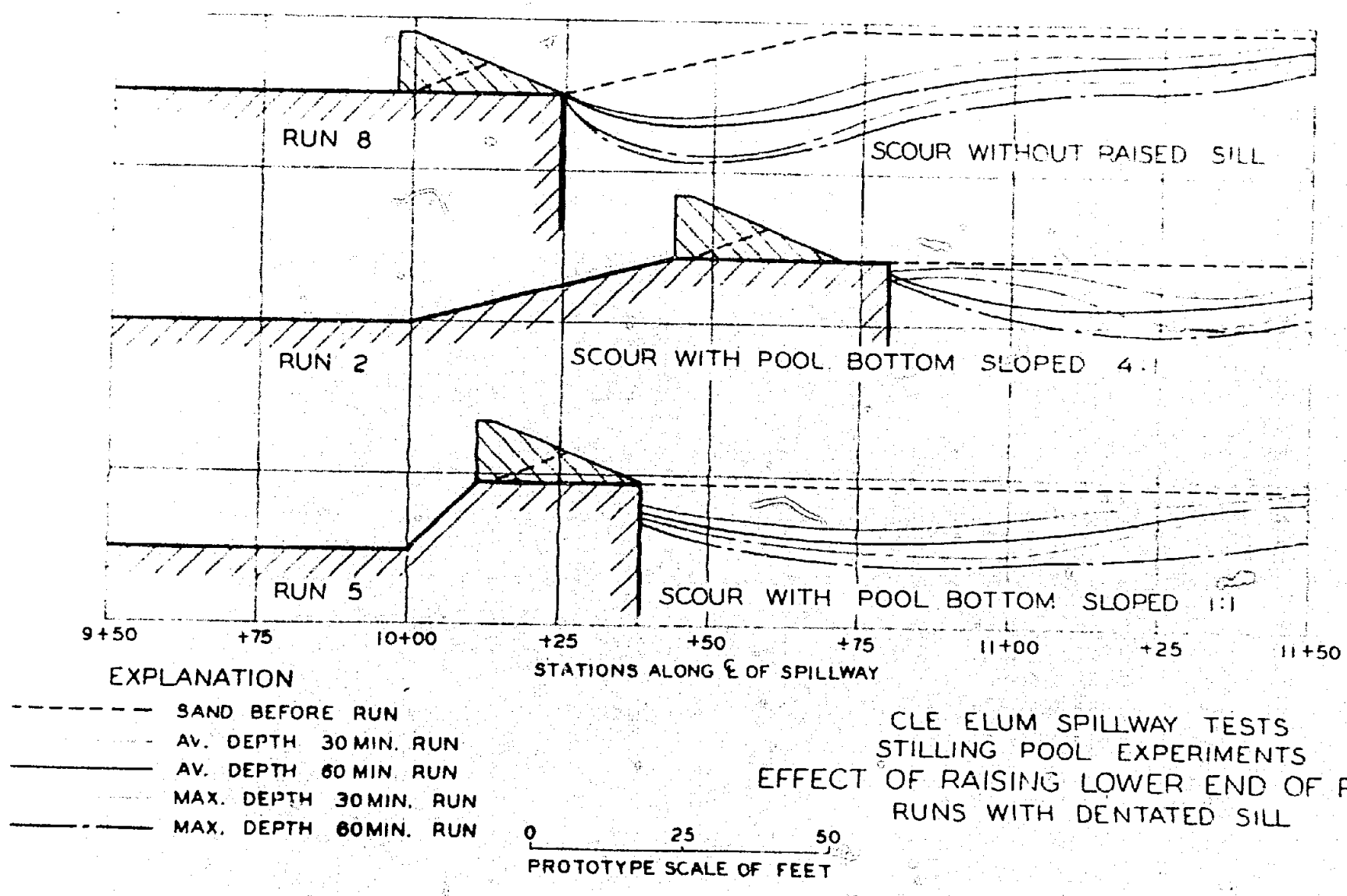
The 4:1 slope at the downstream end of the pool was then changed to a 1:1 slope keeping the length of the level portion of the floor the same. With a dentated sill at the top of the slope the scour was more severe than with the 1:4 slope, as shown on Plate XVI and Figure 18 (Run 5). With a plain sill at the top of the slope instead of the dentated sill the scour was especially severe, as shown

on Plate XVII and Figure .) (Run 6). With the sill moved 25 ft. downstream and the apron extended 25 ft. the scour was still severe, as shown on Plate XVIII and Figure 19 (Run 4).

As a result of these tests, it was concluded that any slope upward in the floor of the pool bottom was undesirable from the standpoing of scour and it was decided to build the pool floor level and make the 11 ft. rise to the level of the bottom of the lead-off channel in the earth section immediately downstream from the stilling pool. Another factor influencing this decision was the possibility of retrogression of the channel level due to scour of the banks and bed of the newly excavated leadoff channel. In an ordinary channel when movement of the material of the bed takes place, the material moved from one locality is replaced by that brought down from upstream points. Where a dam is constructed, however, the movement from above is usually cut off and the result is a lowering of the riverbed which in some cases has caused considerable damage to dams. The effect of lowering the tailwater on the scour with the pool floor at the lower level is indicated on Figure 20. It shows that somewhat more scour would occur. The experiments made on the final design discussed later, show that the maximum scour with the lowered tailwater, as shown on Figure 20 was an effect produced by the glass sides and would not occur with the prototype conditions.

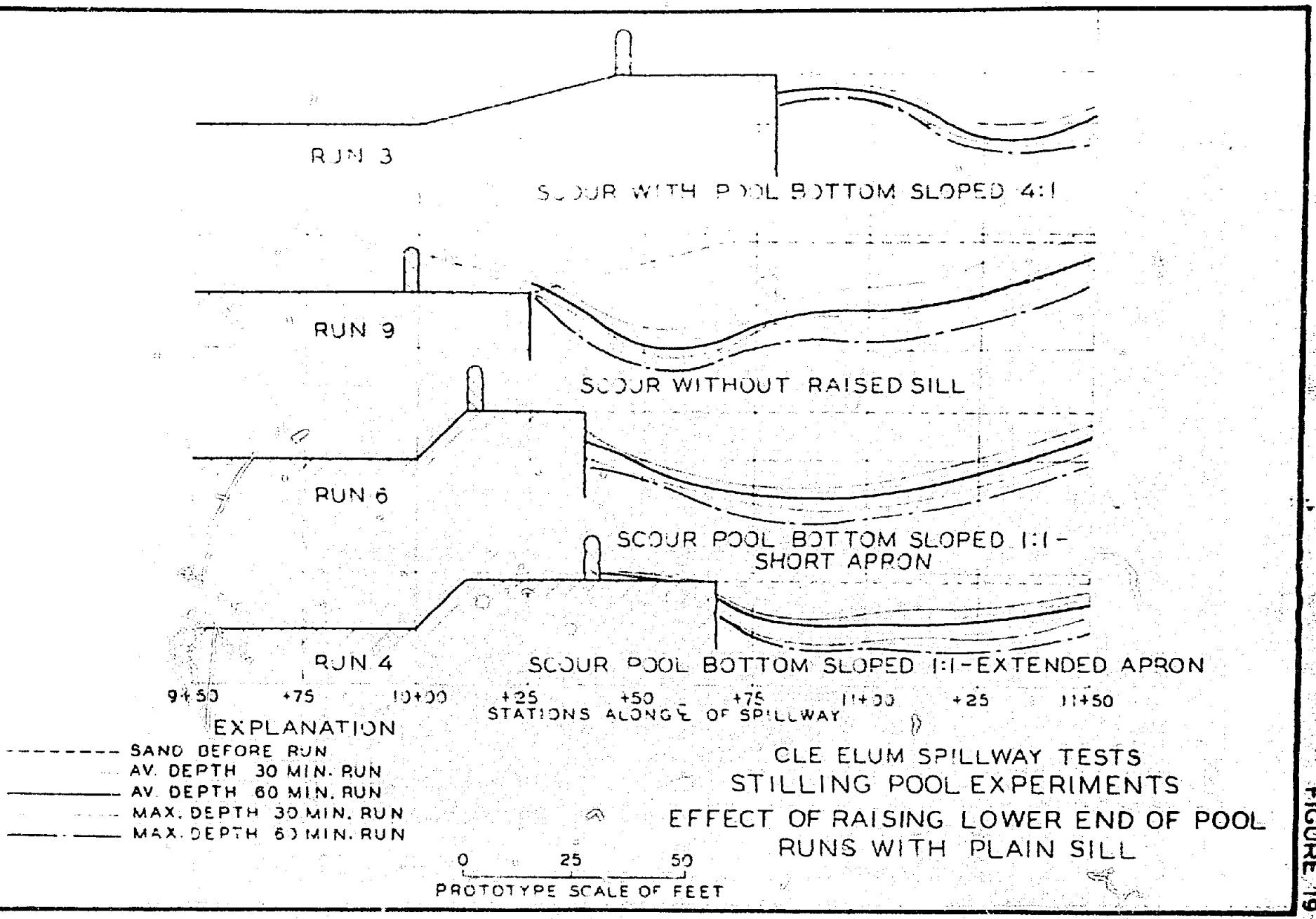
A plain floor at the lower level, without a sill at the end was not satisfactory as shown by the scour test on Plate XIX and Figure 24 ($1\frac{1}{2}$:1 slope). Very satisfactory results were obtained however with the dentated sill (Plate XX and Figure 22), the plain sill at the end of the apron (Plate XXI and Figure 23) and the plain sill 25 ft. upstream from the end of the apron (Plate XXII and Figure 26).

When the experiments showed that it was desirable to have the pool floor level and at an elevation below the bottom of the lead-off channel, the question arose as to whether the transition from the floor level to channel level could be made abrputly or would have to be made gradually. To have made it gradually would have required more excavation. Tests were therefore made with a level sand bottom below the end of the pool and with one sloping up abruptly using a dentated sill. The results of those tests are shown on Plates XX and XXIII and a quantitative comparison is given on Figure 21. The results indicated that although the scour was somewhat greater at the downstream edge of the pool floor when the slope was abrupt, it was not sufficiently severe to justify the additional excavation and the slope up at the end of the apron was therefore adopted.



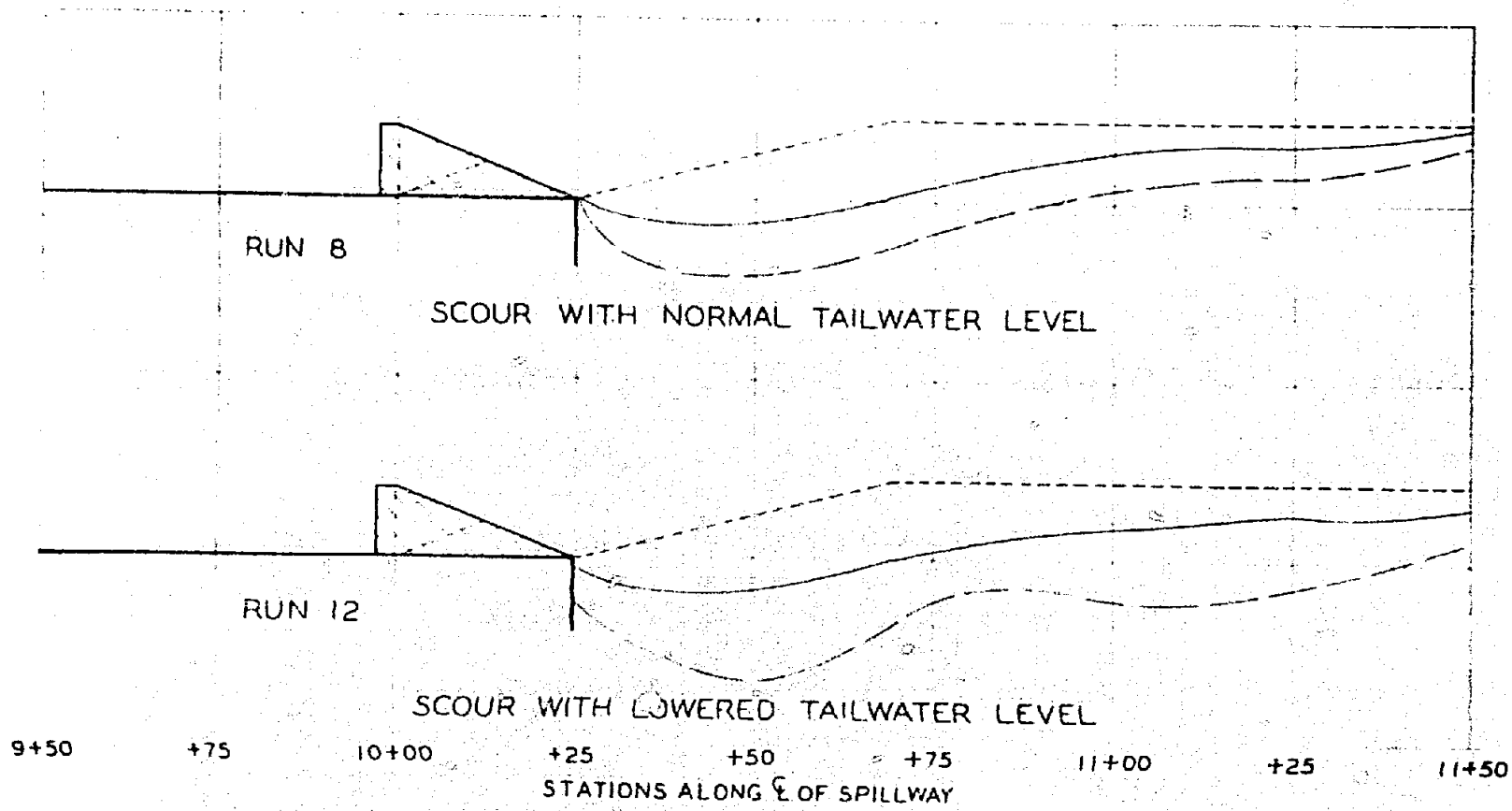
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF RAISING LOWER END OF POOL
RUNS WITH DENTATED SILL

FIGURE 18



CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF RAISING LOWER END OF POOL
RUNS WITH PLAIN SILL

FIGURE 19



EXPLANATION

- - - - - - - - SAND BEFORE RUN

———— AV. DEPTH 30 MIN RUN

— — — — MAX DEPTH 30 MIN RUN

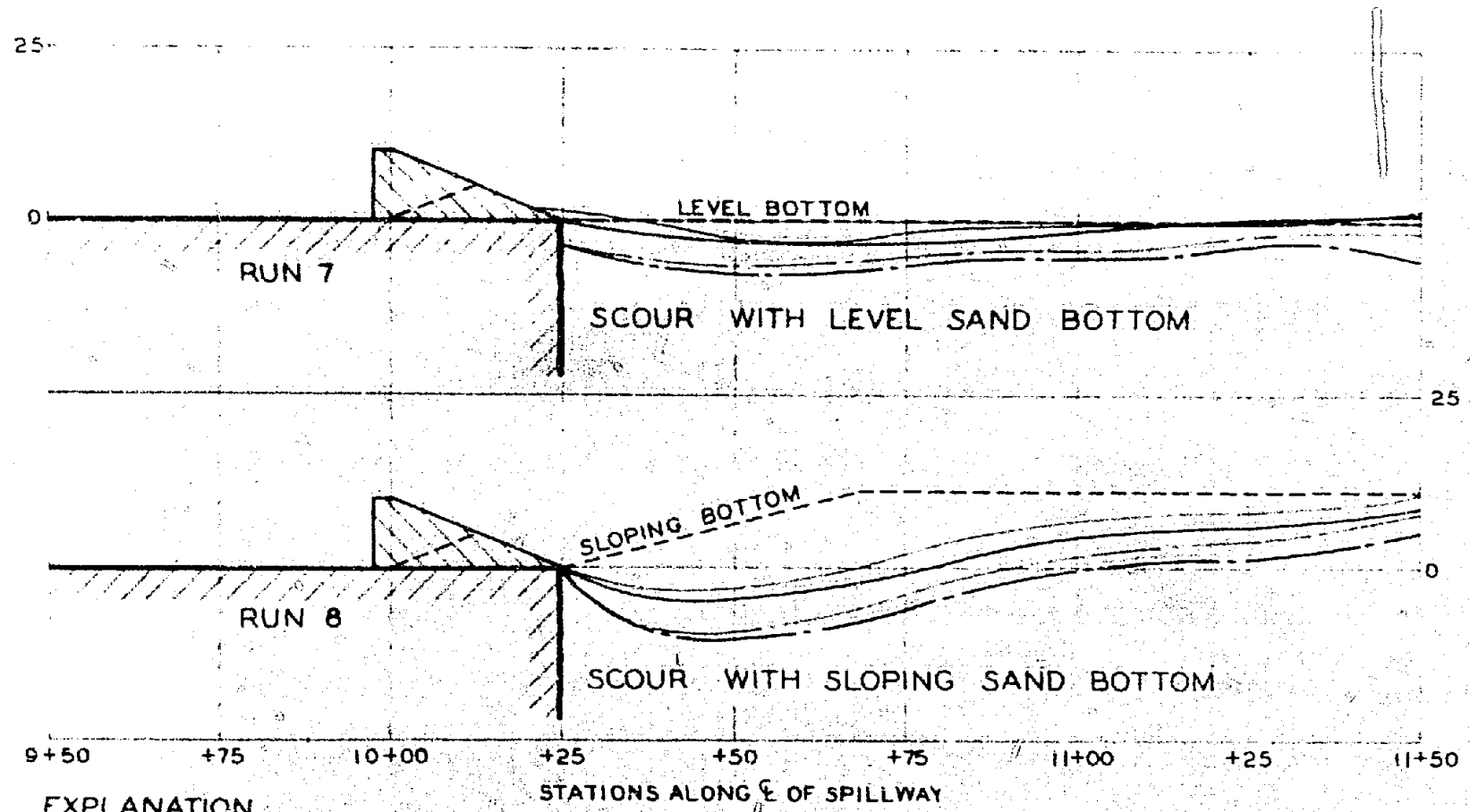
0 25 50

PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS

EFFECT OF TAILWATER HEIGHT ON SCOUR

FIGURE 20



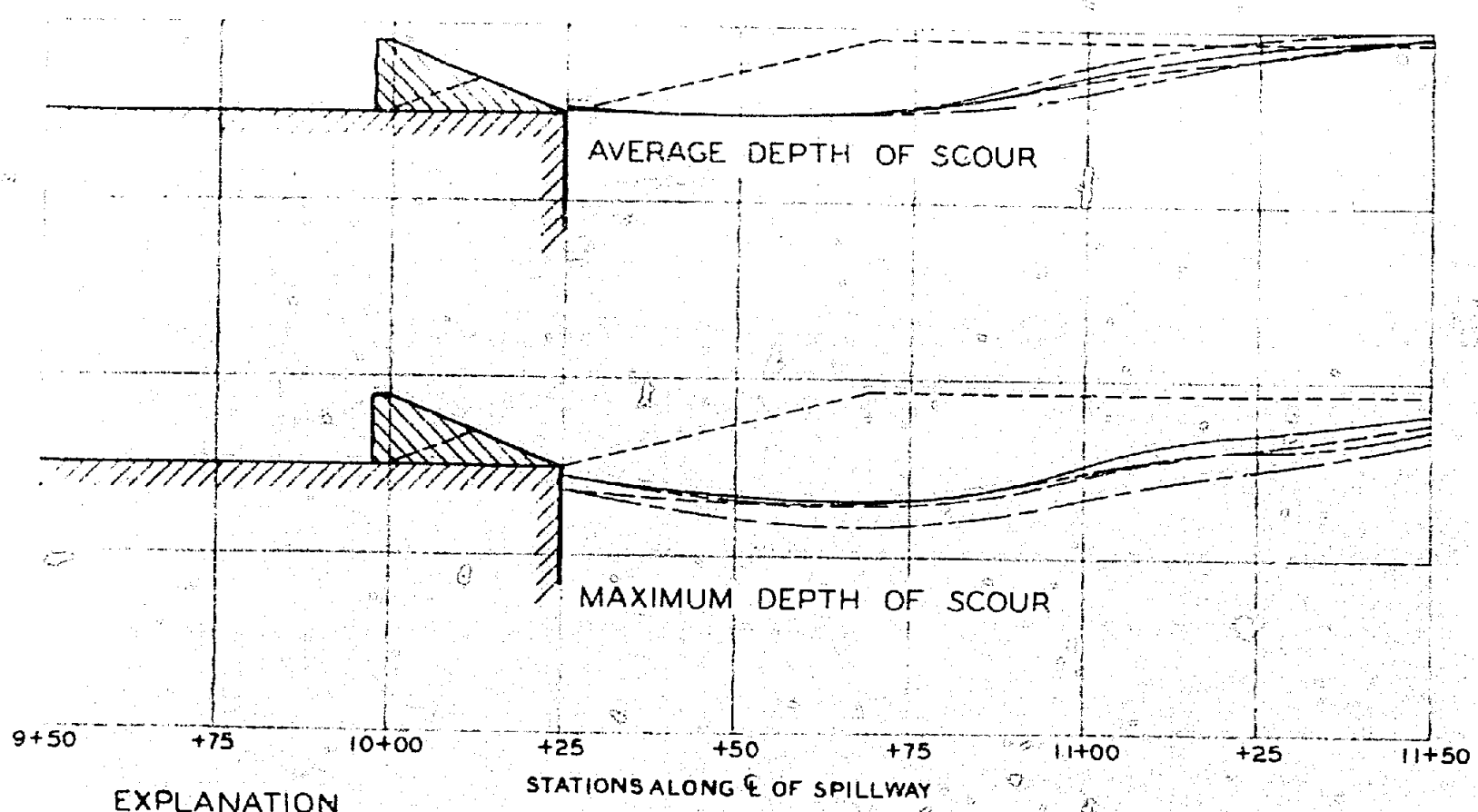
EXPLANATION

- SAND BEFORE RUN
- AV. DEPTH 30 MIN. RUN
- AV. DEPTH 60 MIN. RUN
- MAX. DEPTH 30 MIN. RUN
- MAX. DEPTH 60 MIN. RUN

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EEFECT OF SLOPING SAND BOTTOM

FIGURE 21



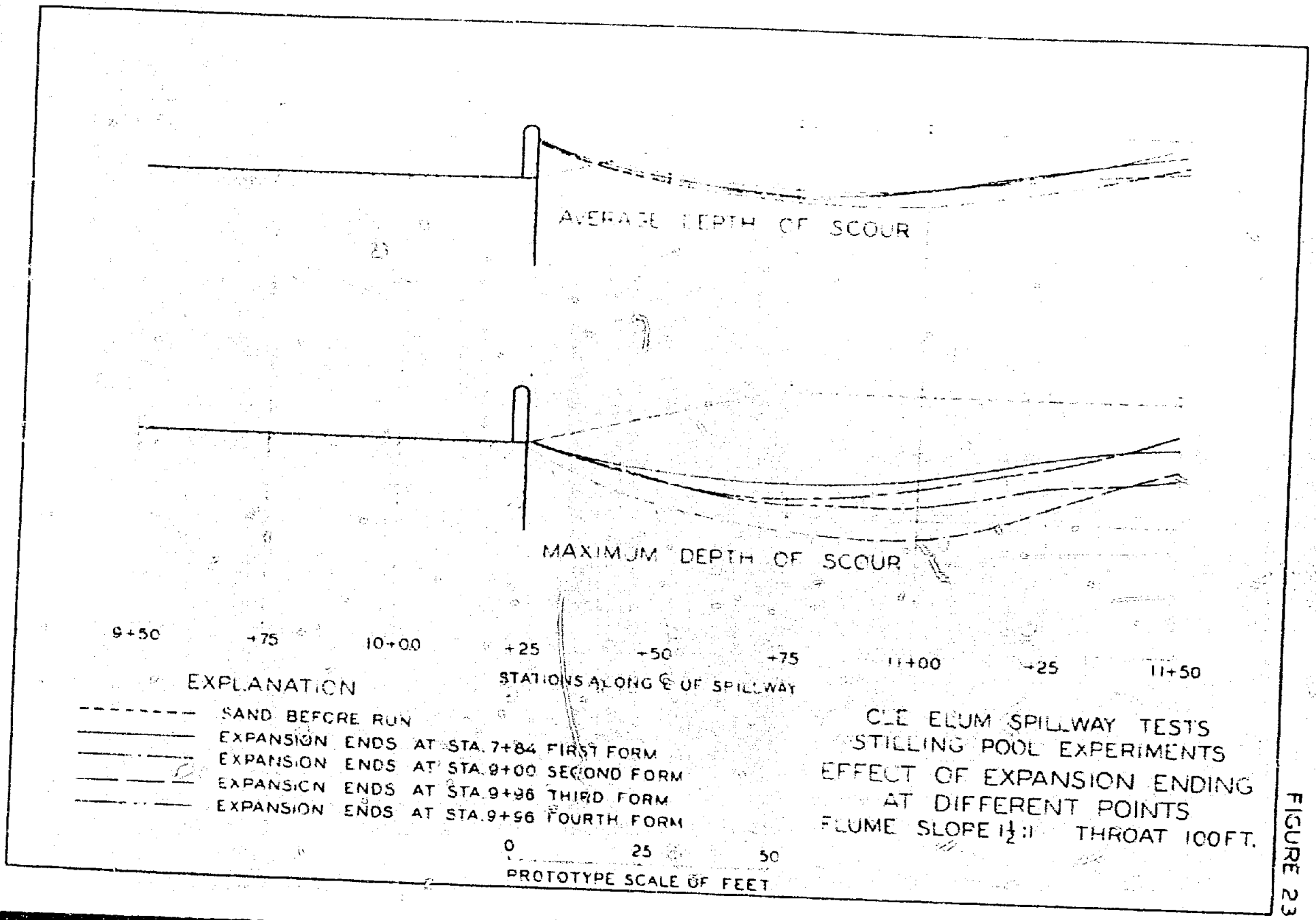
EXPLANATION

- - - - - - - SAND BEFORE RUN
— — — — EXPANSION ENDS AT STA. 7+84 FIRST FORM
———— EXPANSION ENDS AT STA. 9+00 SECOND FORM
—·—·— EXPANSION ENDS AT STA. 9+96 THIRD FORM
—··—··— EXPANSION ENDS AT STA. 9+96 FOURTH FORM

0 25 50
PROTOTYPE SCALE OF FEET

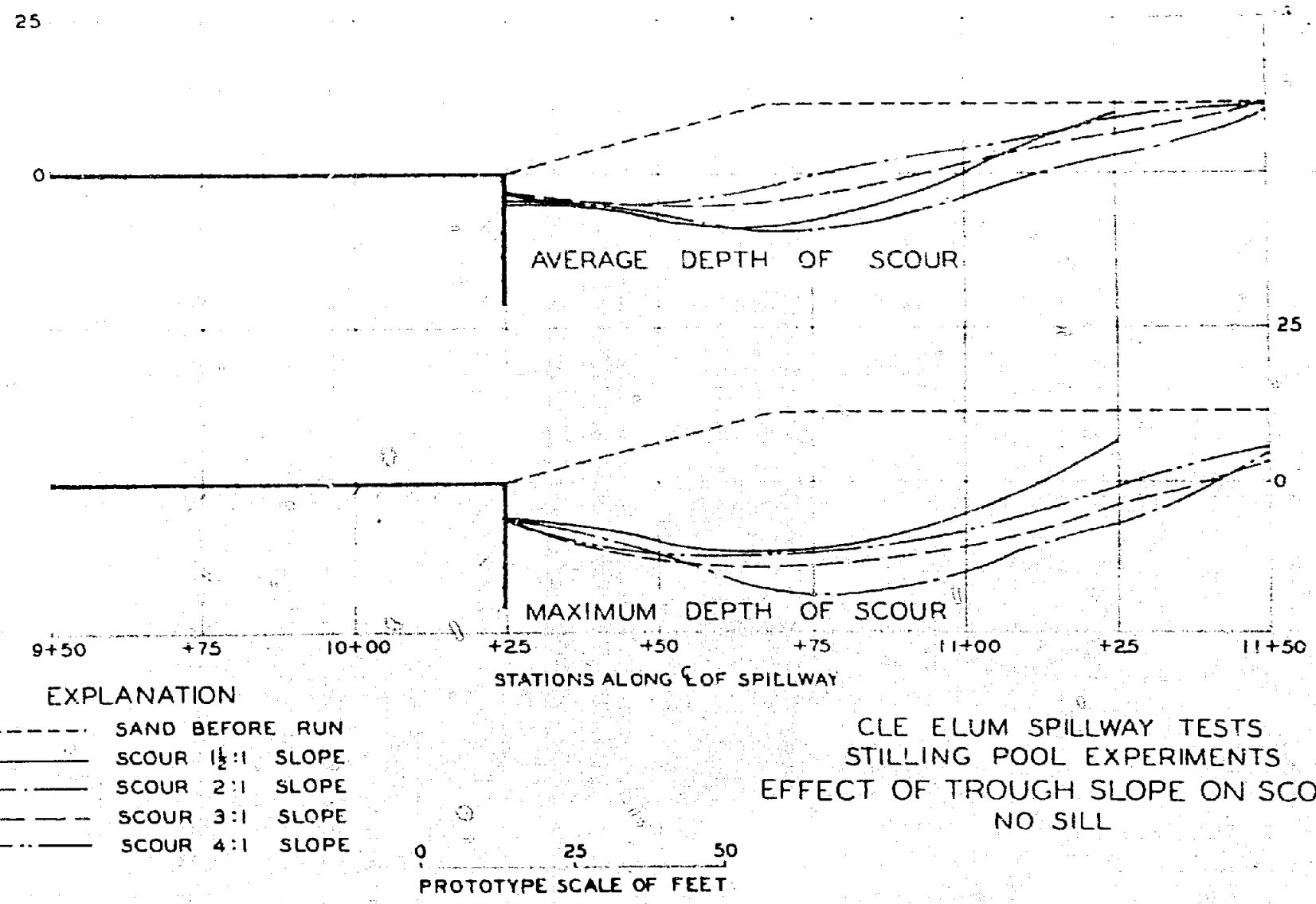
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF EXPANSION ENDING
AT DIFFERENT POINTS
FLUME SLOPE $1\frac{1}{2}$:1 THROAT 100 FT.

FIGURE 22



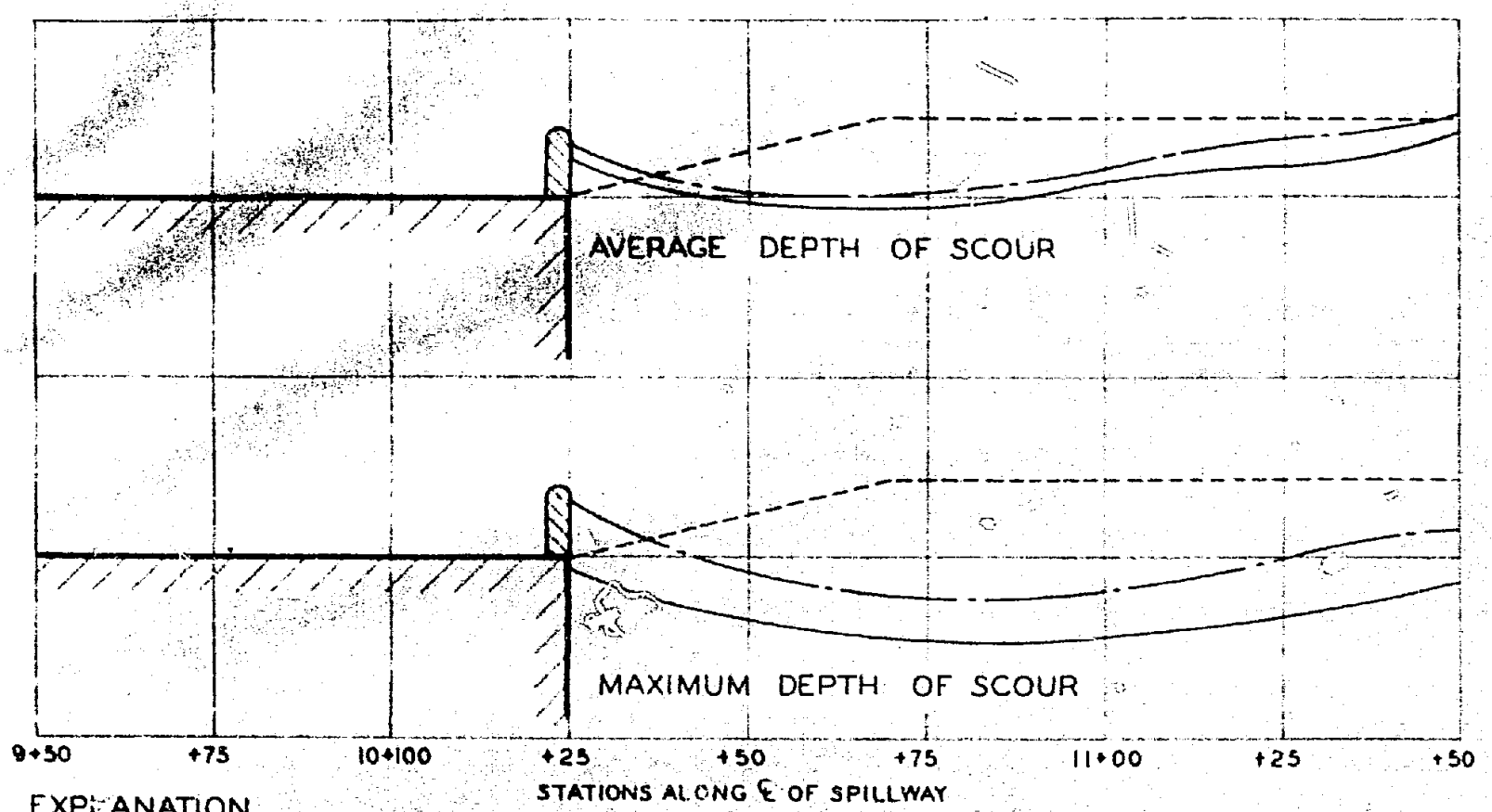
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF EXPANSION ENDING
AT DIFFERENT POINTS
FLUME SLOPE 1½:1 THROAT 100 FT.

FIGURE 23



CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF TROUGH SLOPE ON SCOUR
NO SILL

FIGURE 24



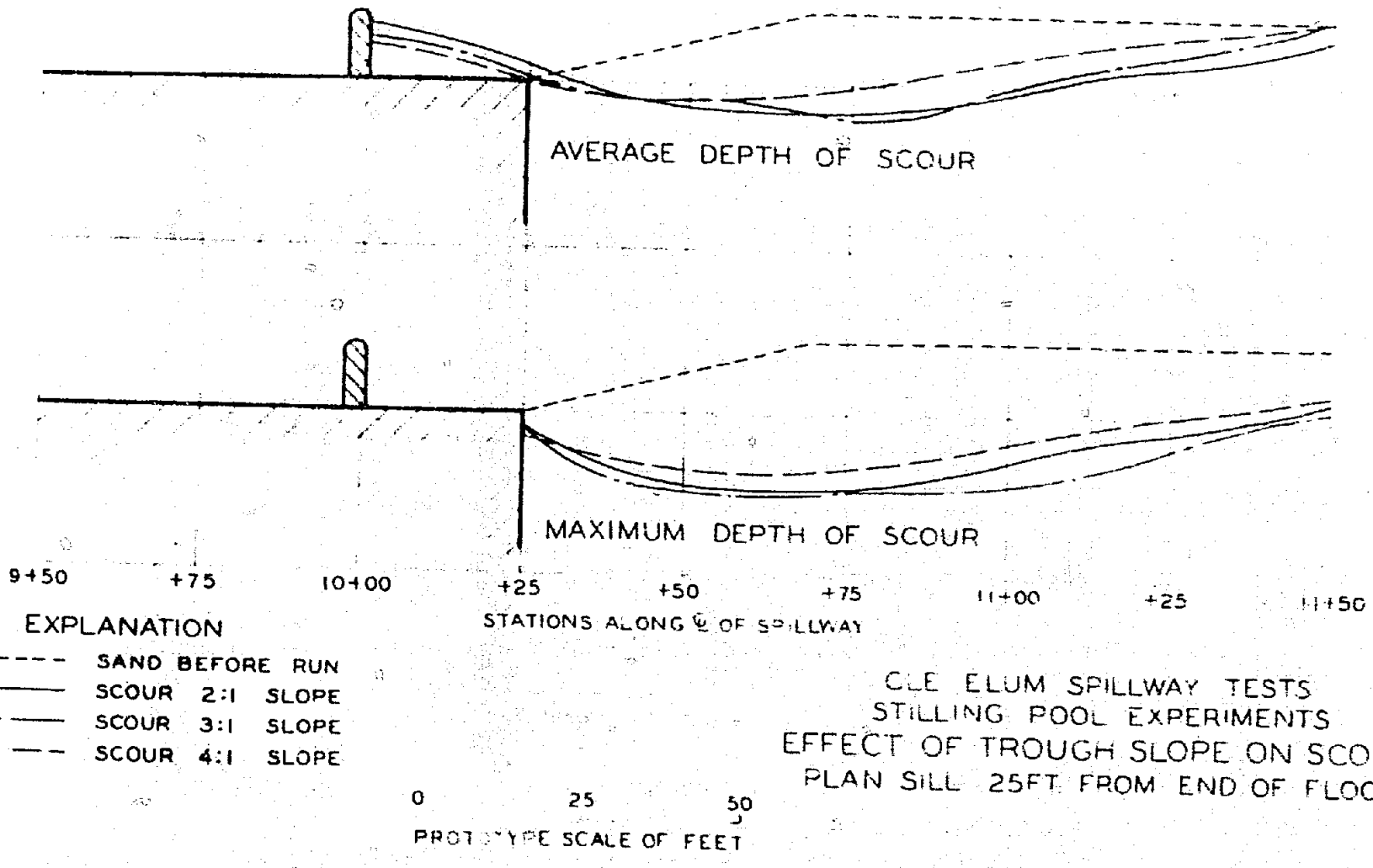
EXPLANATION

- - - - - - - SAND BEFORE RUN
——— SCOUR 3:1 SLOPE
—·— SCOUR 4:1 SLOPE

0 25 50
PROTOTYPE SCALE OF FEET

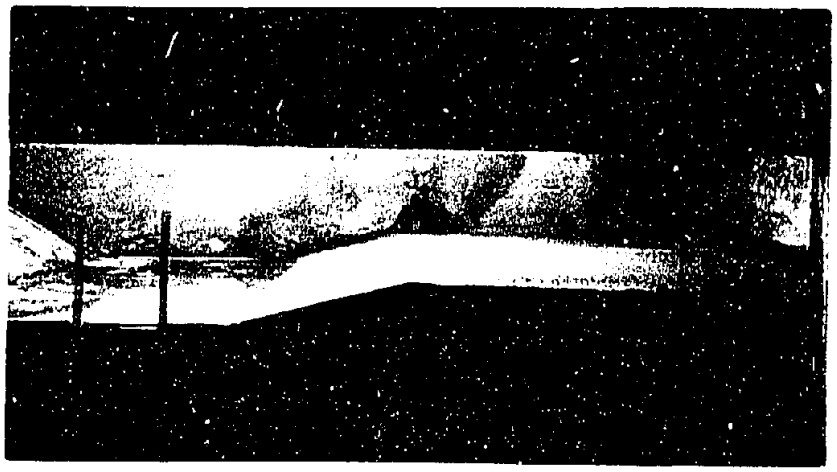
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF TROUGH SLOPE ON SCOUR
PLAIN SILL AT END OF FLOOR

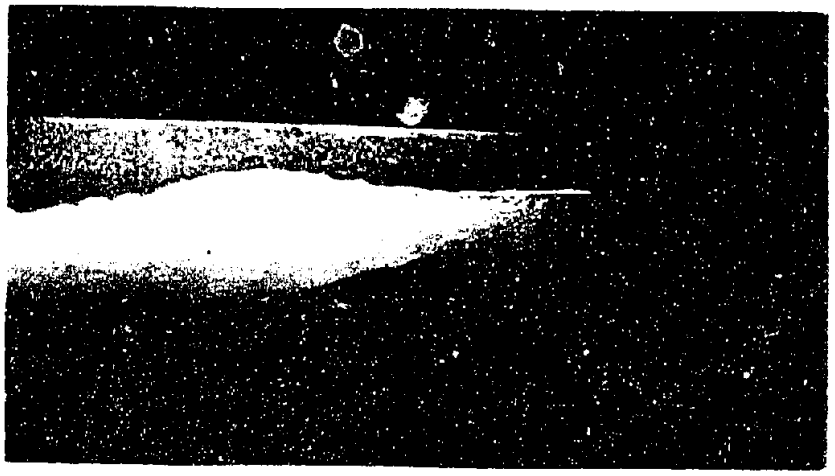
FIGURE 25



CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF TROUGH SLOPE ON SCOUR
PLAN SILL 25FT FROM END OF FLOOR

FIGURE 26





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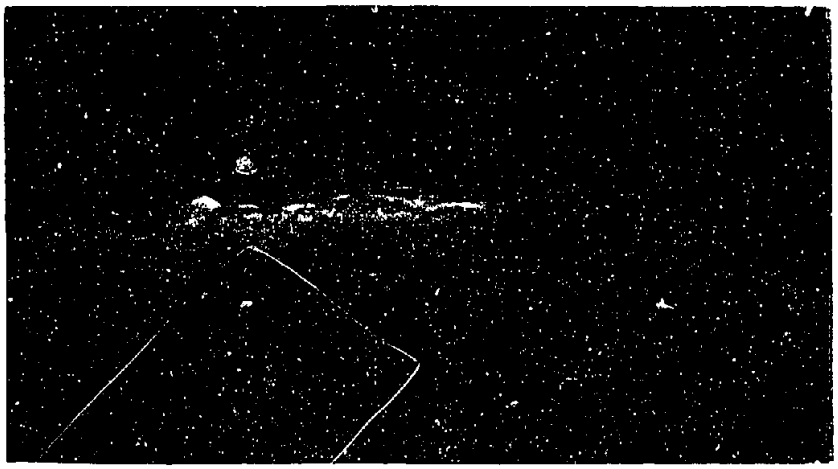

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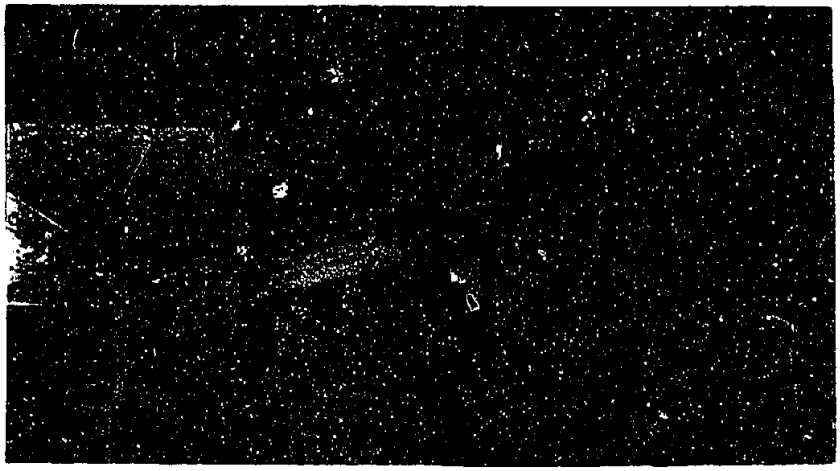
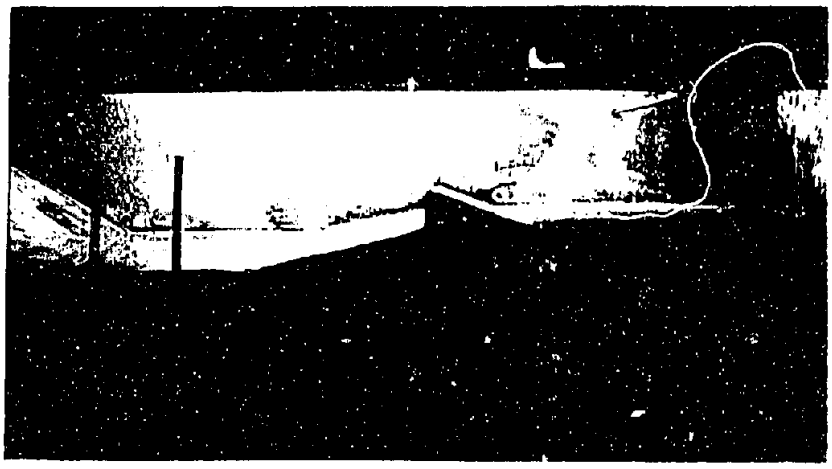
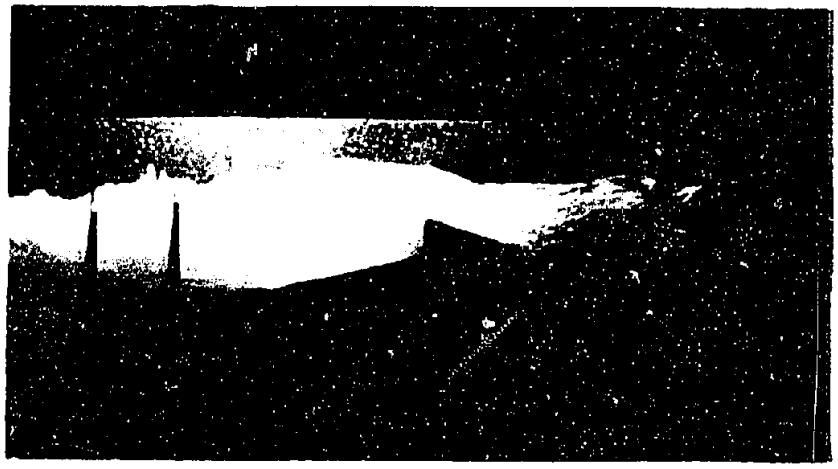


[illegible]

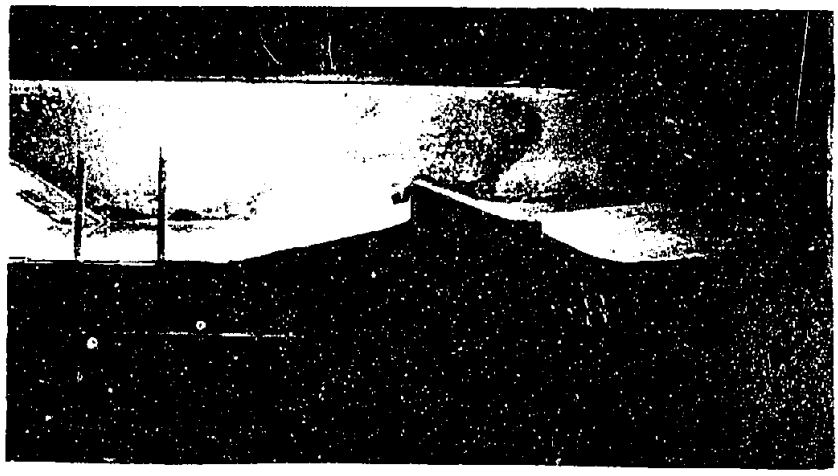
SCOUR WITH ORIGINAL POOL DESIGN
AND 60 FOOT TROUGH THROAT



A-BEFORE RUN

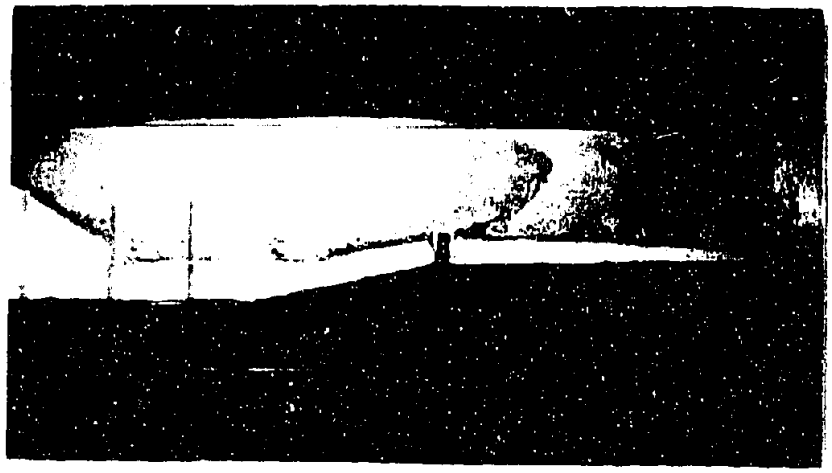


B-DURING RUN

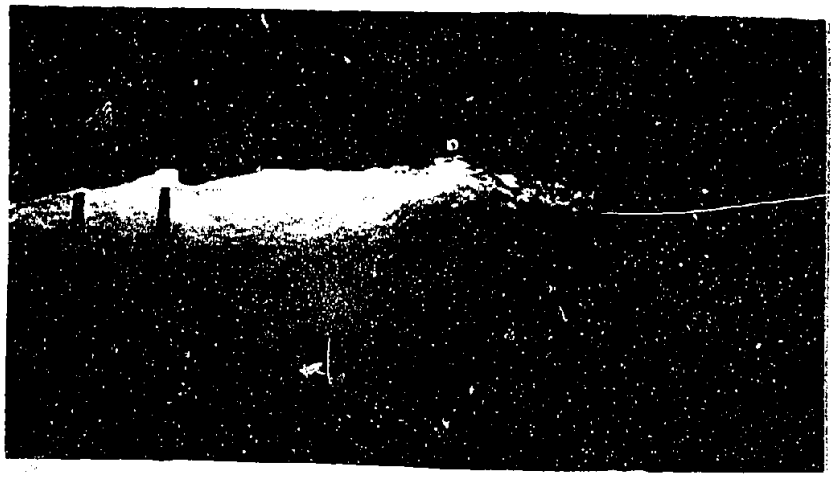


C-AFTER RUN

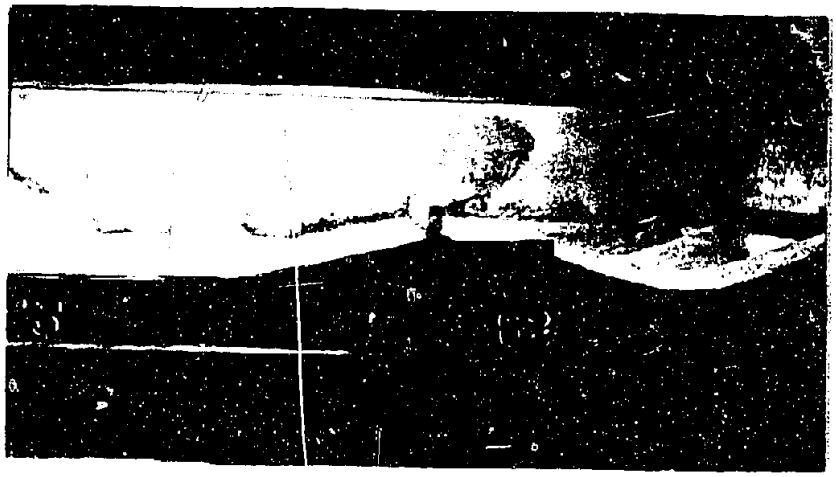
FLOOR WITH DENTATES [illegible]
FLOOR OF ORIGINAL POOL DESIGN



A-BEFORE RUN

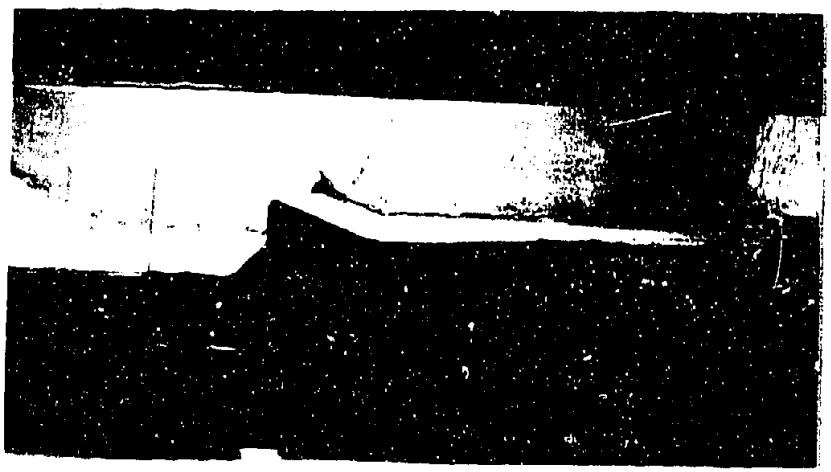


B-DURING RUN

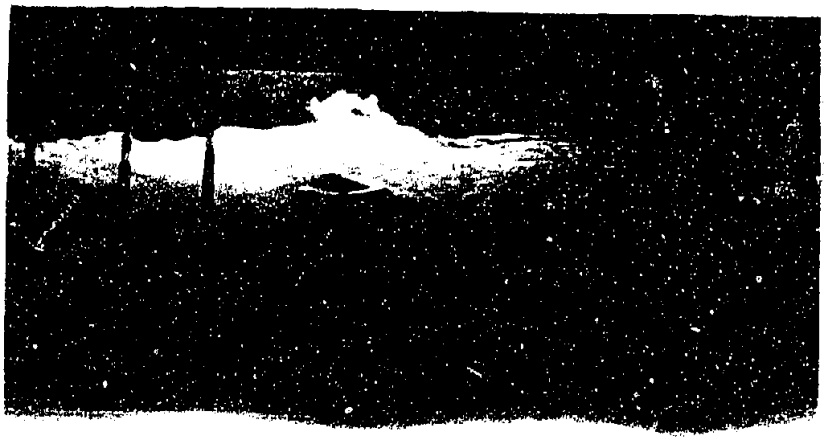


C-AFTER RUN

SCOUR WITH PLAIN SILL AT TOP OF A SLOPING
FLOOR OF ORIGINAL POOL DESIGN



[illegible]



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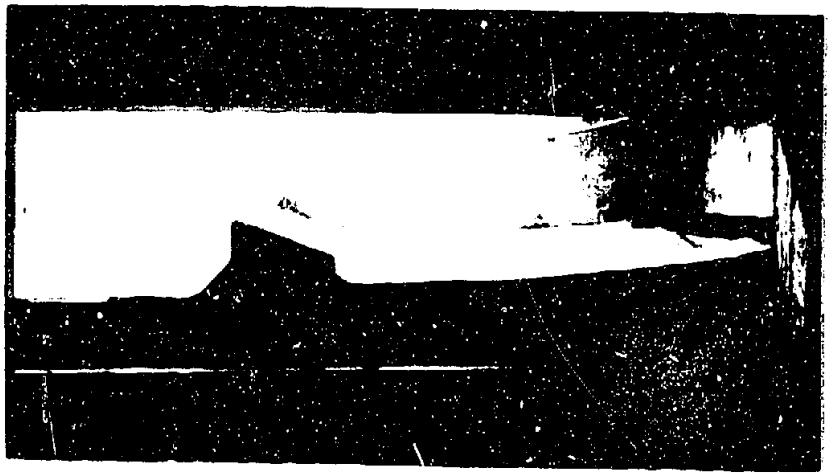
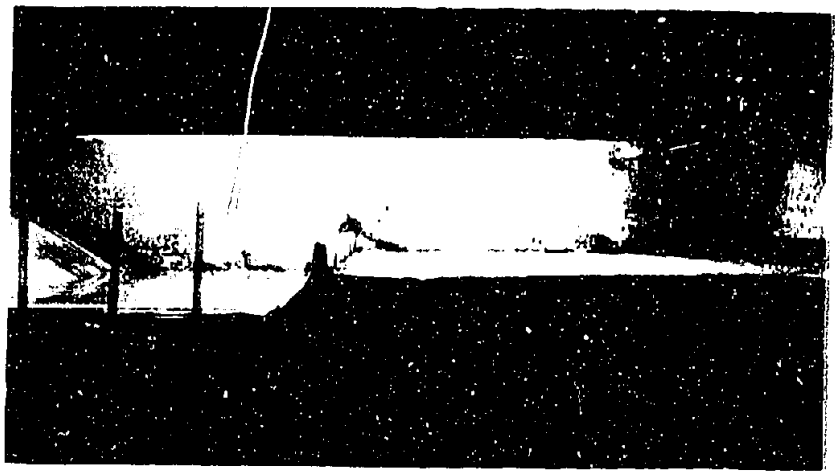


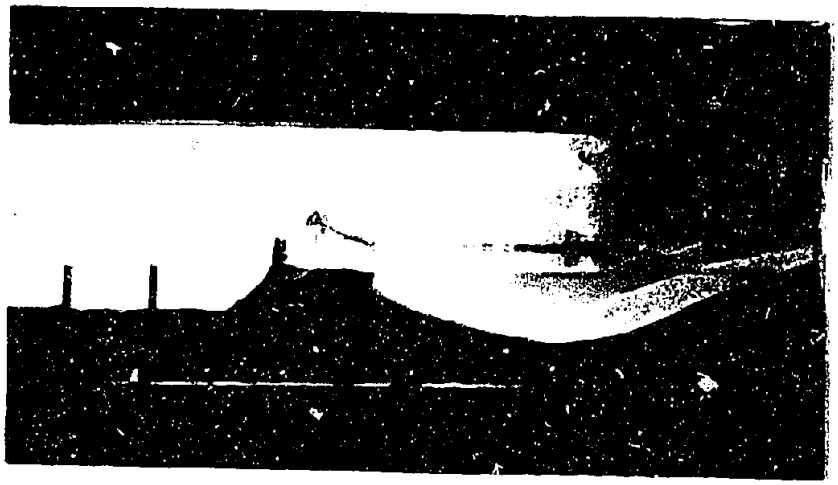
[illegible]

[illegible]
[illegible]



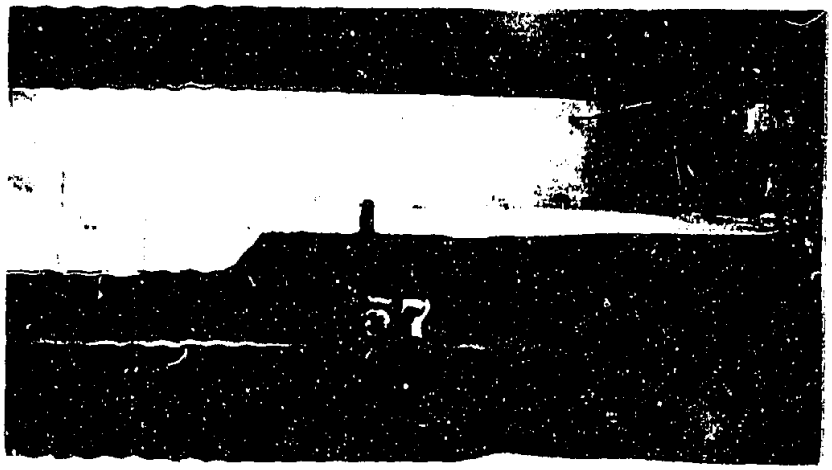
a-BEFORE RUN

b-DURING RUN

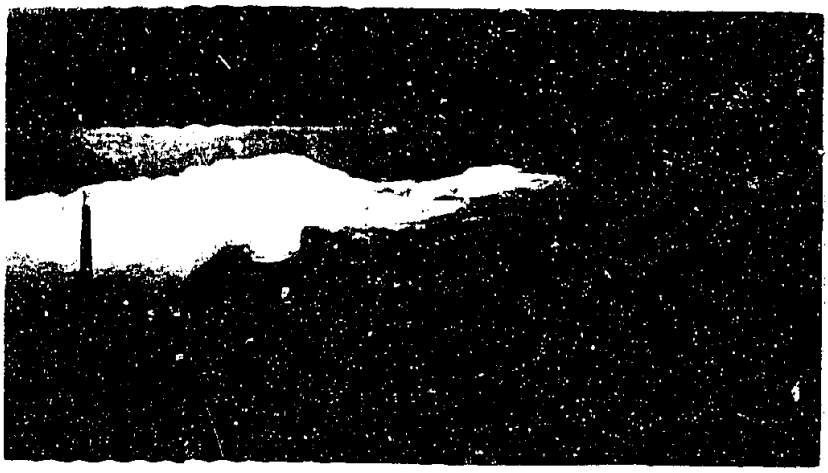


c-AFTER RUN

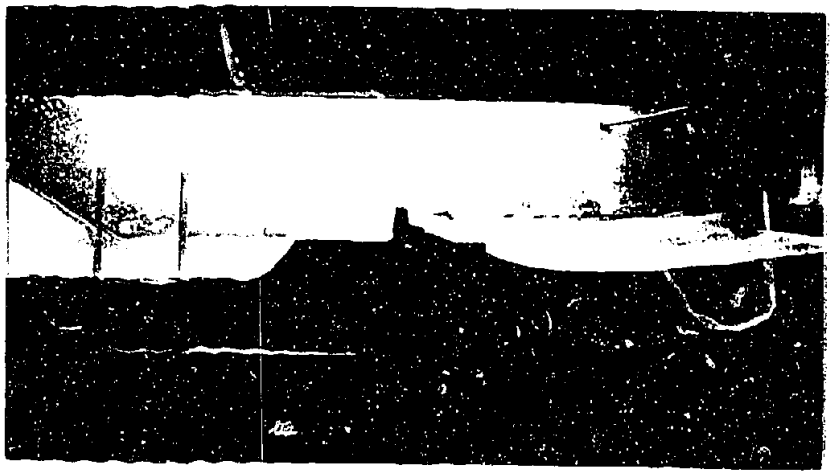
SCOUR WITH PLANE SURFACE OF
1 SLOPE OF POOL FLOOR



A-BEFORE RUN

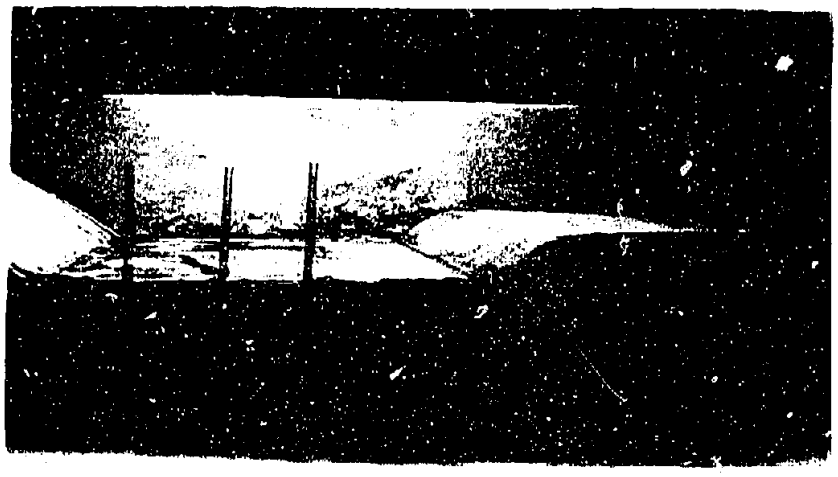


B-DURING RUN

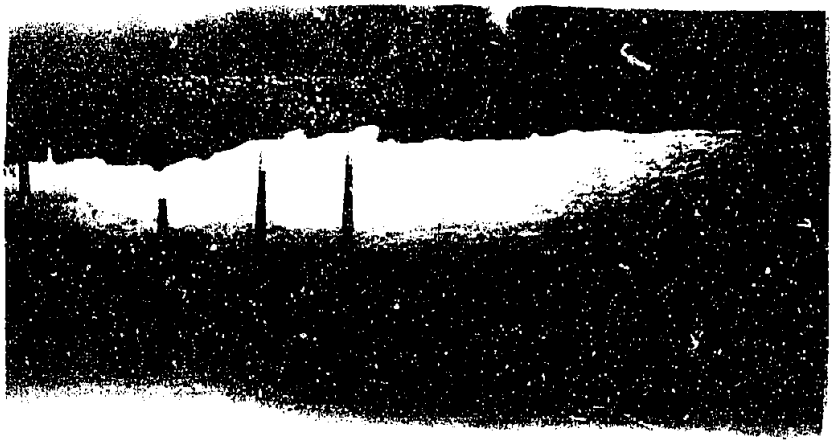


C-AFTER RUN

SCOUR WITH PLAIN SILL IN CENTER OF FLOOR
AT TOE OF 1:1 SLOPE



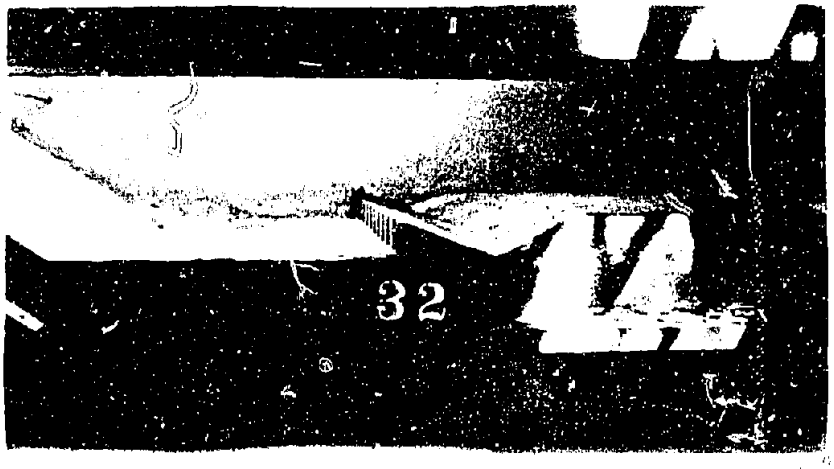
A-BEFORE RUN



B-DURING RUN

C-AFTER RUN

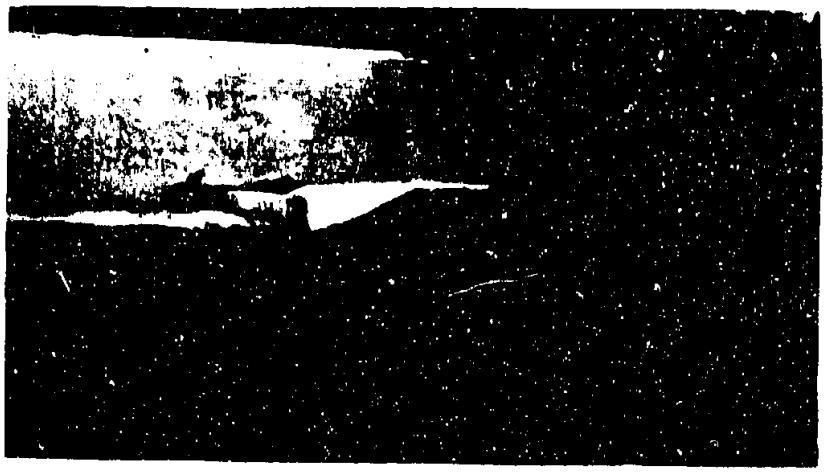
SCOUR WITH LEVEL POOL BOTTOM AND NO SILL

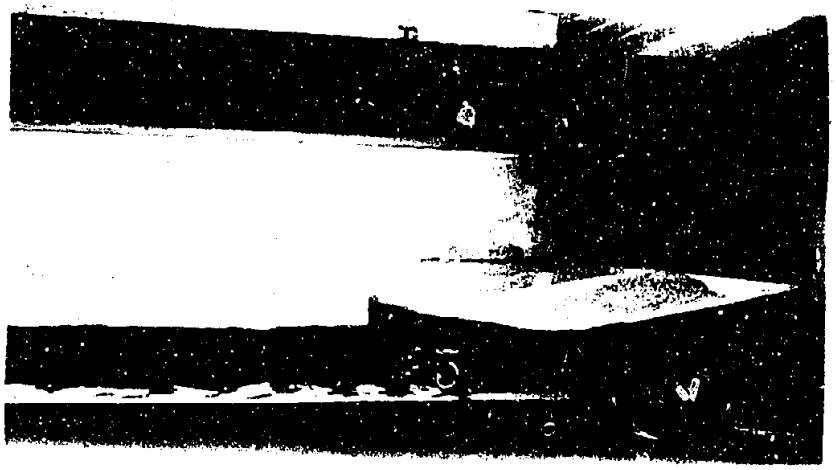


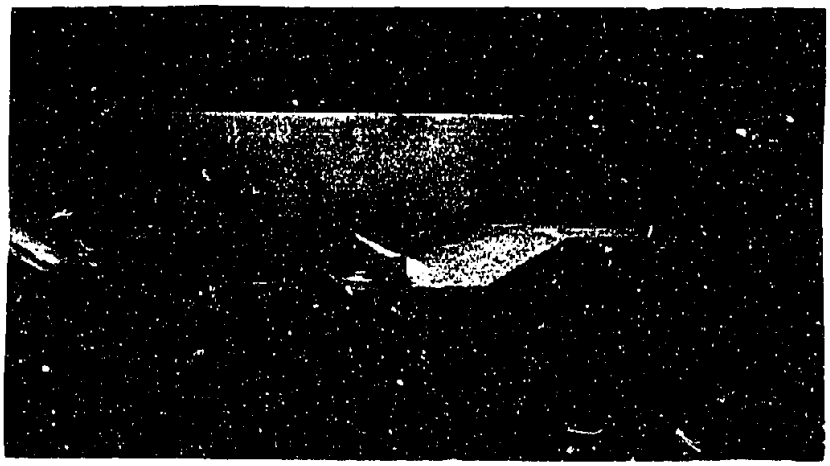
A-BEFORE RUN

B-AFTER RUN

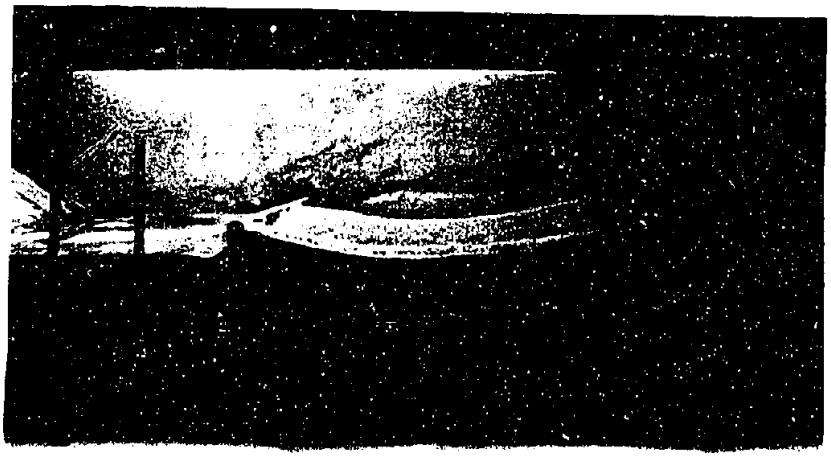
SCOUR WITH LEVEL POOL BOTTOM AND DENTATED SILL





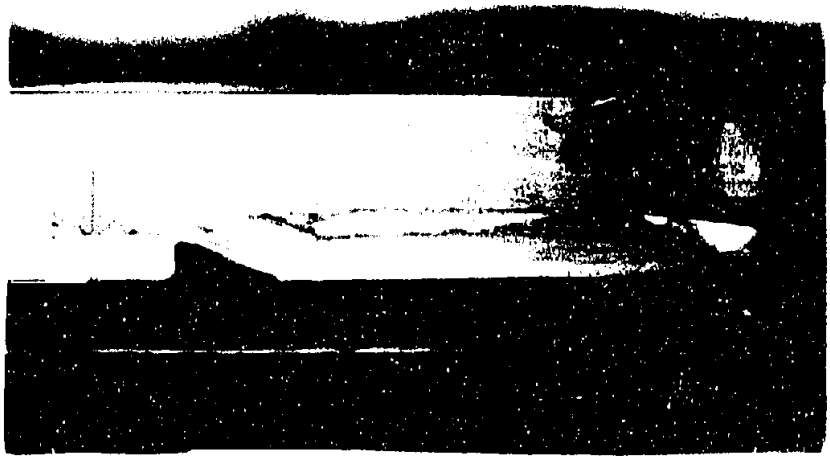


A-BEFORE RUN

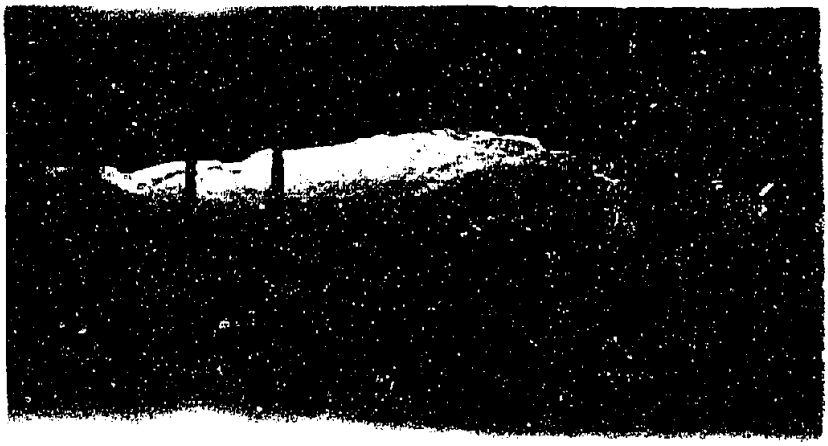


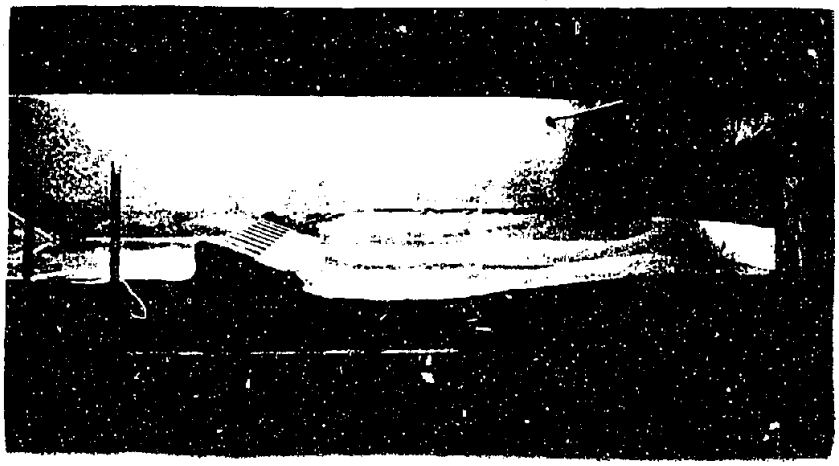
B-AFTER RUN

SCOUR WITH LEVEL POOL BOTTOM AND PLAIN SILL
UPSTREAM FROM END OF FLOOR



A. BEFORE RUN.





C. AFTER RUN.

SCOUR WITH LEVEL SAND BED BELOW DENTATED SILL

Alignment of the Side Walls of the Stilling Pool

A series of tests were made to determine the best alignment of the side walls near the lower end of the trough and in the stilling pool. Runs were made with four different conditions. In the first of these the sides of the flume expanded to a width of 200 ft. at Station 7+84, the top of the steep slope leading into the stilling pool. The sides of the steel slope and the pool were parallel and 200 ft. apart. The second setup had sides expanding to the 200 ft. width at Station 9+00, the bottom of the steep slope leading into the pool, and the sides of the pool were parallel. In the third case the sides expanded gradually with a uniformly increasing curvature until Station 9+96, the upstream side of the dentated sill was reached, below which the sides were parallel. It was found however that since the velocity increased gradually from the throat to the top of the steep slope, at which point it suddenly speeded up, there should be a change in the rate of expansion at the top of the slope. For a given rate of expansion in a given time of travel of the water, the higher the velocity the lower would be the rate of expansion with respect to distance of water travel or length of the flume. In order that the rate of expansion of the flume from the standpoint of time should not increase suddenly at the top of the steep slope, it was necessary to decrease the rate of expansion of the sides, with respect to distance along the flume at that point. This led to a form similar to that shown in Figure 3. This fourth form more closely fitted the natural tendencies of the water to expand than the third form, in which there was a sudden increase in the rate of expansion, with respect to time, at the top of the slope. The results of all four setups with two different types of sill are shown on Figures 22 and 23 and on Plates XXIV and XXV. In each case the average scour with the fourth form was less than with the other three, and the maximum scour at the critical point, the downstream edge of the concrete floor, was no more severe than the others. The fourth form was therefore adopted, later being slightly modified to the form shown in Figure 3, with sides expanding to the downstream edge of the concrete floor.

Effect of Slope of the Trough Floor Leading into the Stilling Pool

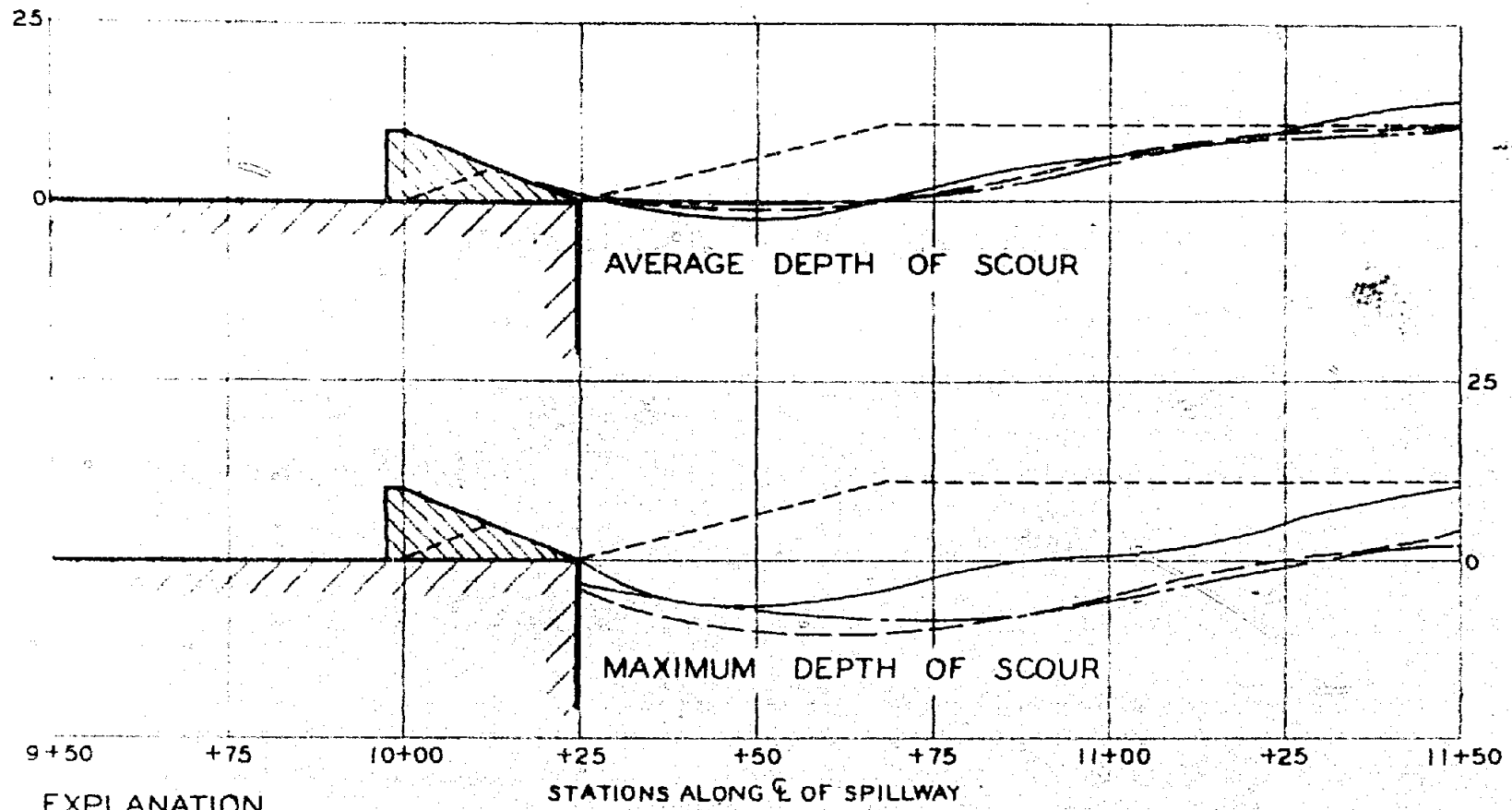
A series of tests were run to determine the slope leading into the stilling pool which would produce the least scour. It has been the practice of the Bureau of Reclamation as a result of their experience, to construct their chutes with a slope of $1\frac{1}{2}$:1 entering the pool. It is the contention of Prof. S. M. Woodward however, that a slope steeper than 3:1 reduces the efficiency of the jump in dissipating energy of the flowing water. Tests were therefore made with slopes of $1\frac{1}{2}$:1, 2:1, 3:1, and 4:1 with the trough with parallel sides.

This type of trough was used as it was believed that the results on it would be more likely to be of general application than if obtained with a flume of other shapes. In all cases the junction of the sloping floor with the level pool floor was made with an angle and not with a transition curve. With no sill the results seem to support Prof. S. M. Woodward's contentions, as they in general show less scour at the flatter slopes (See Figure 24 and Plates XXVI and XXVII). With a plain sill at the end of the floor and also 25 ft. upstream the results are similar (See Figures 25 and 26 and Plates XXVII, XXVIII and XXIX) but with a dentated sill (Figure 27 and Plate XXX) the average depth of scour is about the same for the 2:1, 3:1 and 4:1 slopes, but the maximum scour depth was greater from the flatter slopes. As already stated, the significance of the maximum depth of scour is uncertain.

In determining the best slope entering the pool other considerations than the scour effect downstream must be considered. For a given position of the downstream end of the pool the flatter slopes require that the high retaining walls forming the pool sides be longer, thus increasing their cost. A greater length of trough floor is below the tailwater level when the slopes are flatter, and is subject to uplift, as will be explained later, when a jump is formed in the stilling pool. A thick floor is necessary to resist the uplift especially if the pressure is not relieved by vents and with the flatter slopes this thick floor must be extended over a longer distance, thus increasing the cost. Moreover, a flatter slope would require more excavation than required in the dam, thus increasing the cost. As the results of the tests with the dentated sills indicated that little difference in scour would result from changing the slope of the floor entering the stilling pool, it was believed that the increased cost of the floor and side walls which would result with slopes flatter than the 1½:1 used in the original design would in this case more than offset any advantage in cost which might be gained.

Effect of a Transition Curve at the Bottom of the Slope Entering the Pool

As previously stated, the tests of the effect of the slope of the floor entering the stilling pool were made with an angle at the junction of the slope with the pool floor. As the Cle Elum Dam is constructed in a timber country, where saw logs might pass through in times of flood, and since logs nearly always flow through such a structure end on, it was feared that the end of a large log passing through the spillway might strike the floor of the pool a nearly direct blow of great force. Such a possibility would be greatly decreased if the junction of the sloping floor and pool bottom was made with a curve of sufficient radius so that only a glancing blow would be struck. The effect of such a curve, with a radius of approximately



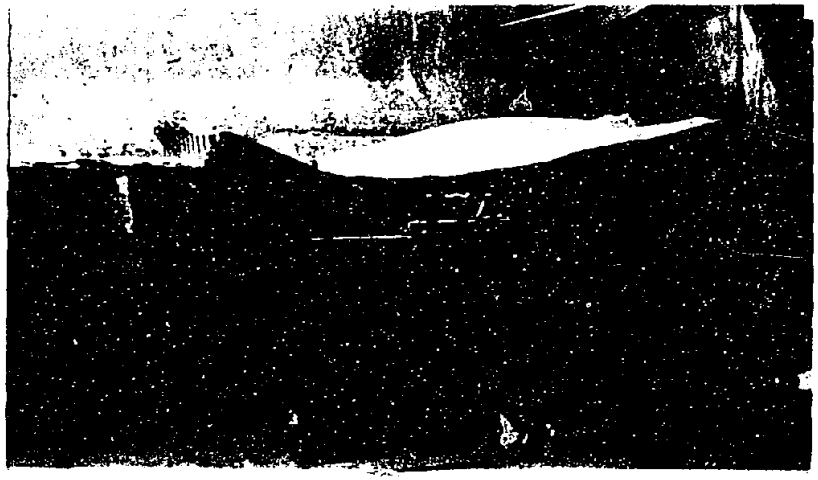
EXPLANATION

- - - - - - - SAND BEFORE RUN
——— SCOUR 2:1 SLOPE
—— · —— SCOUR 3:1 SLOPE
— — — SCOUR 4:1 SLOPE

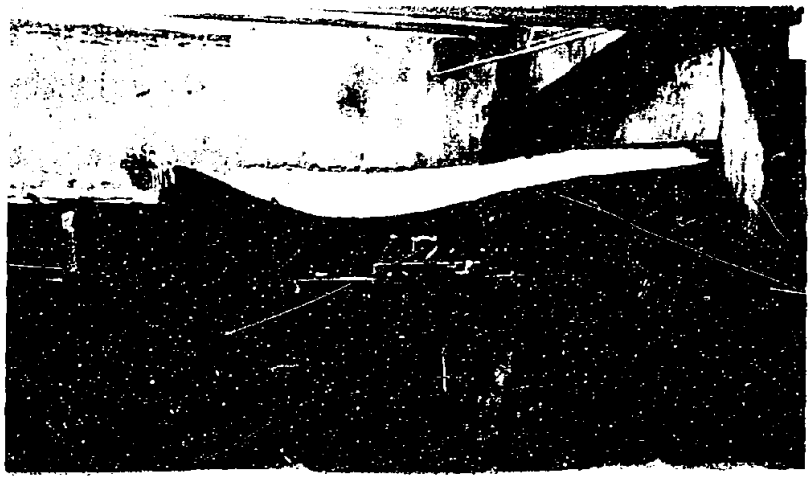
0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF TROUGH SLOPE ON SCOUR
DENTATED SILL

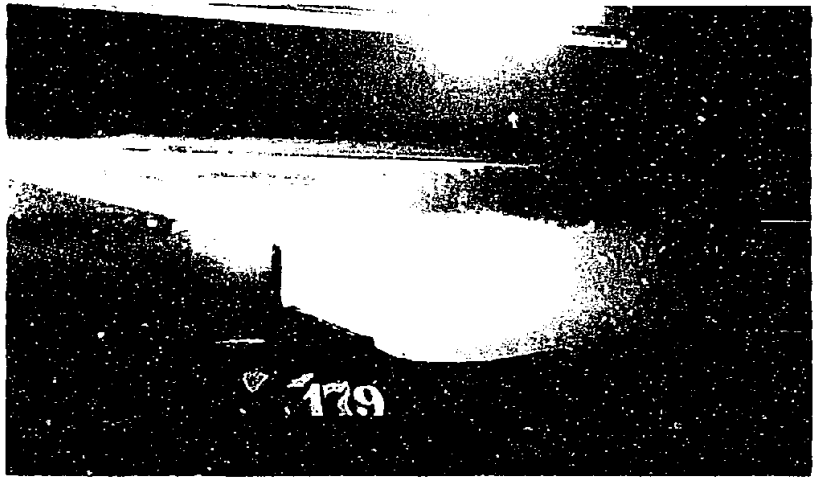
FIGURE 27



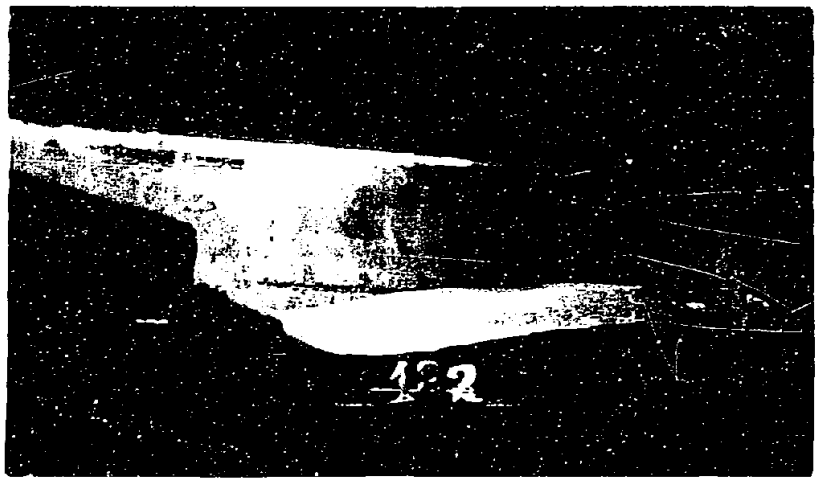
a. EXPANDING SIDES ENDING AT TOP OF SLOPE



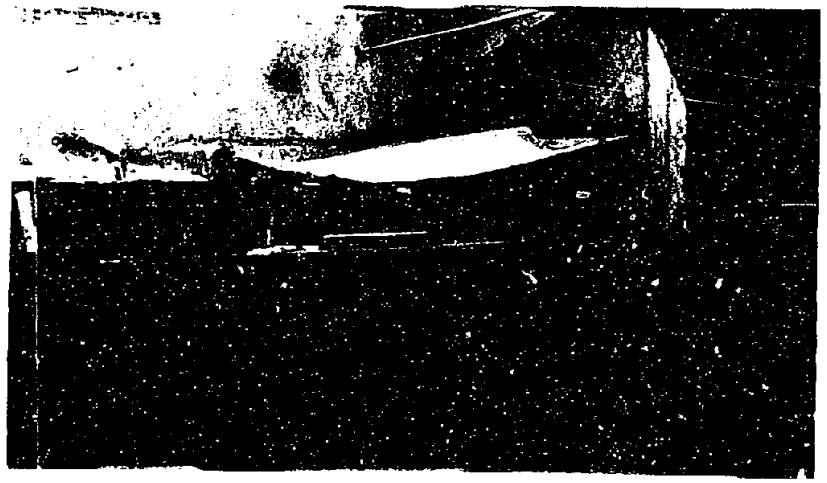
a. EXPANDING SIDES ENDING AT BOTTOM OF SLOPE

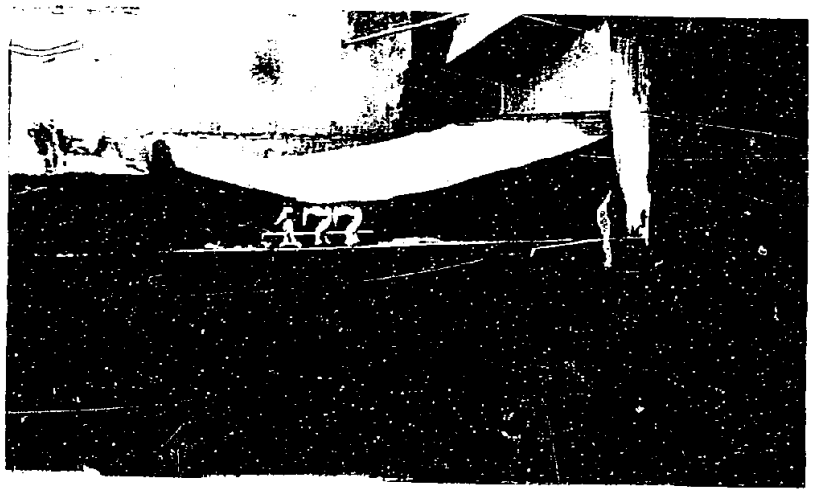


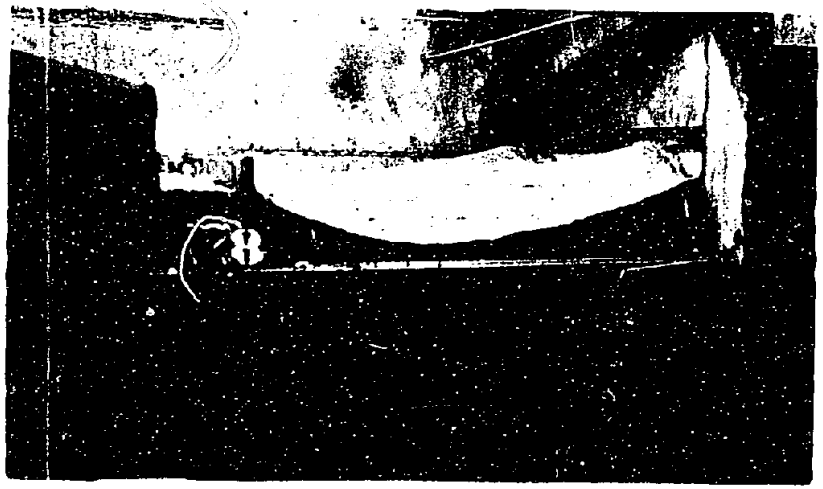
c-SIDES EXPANDING UNIFORMLY TO END OF POOL

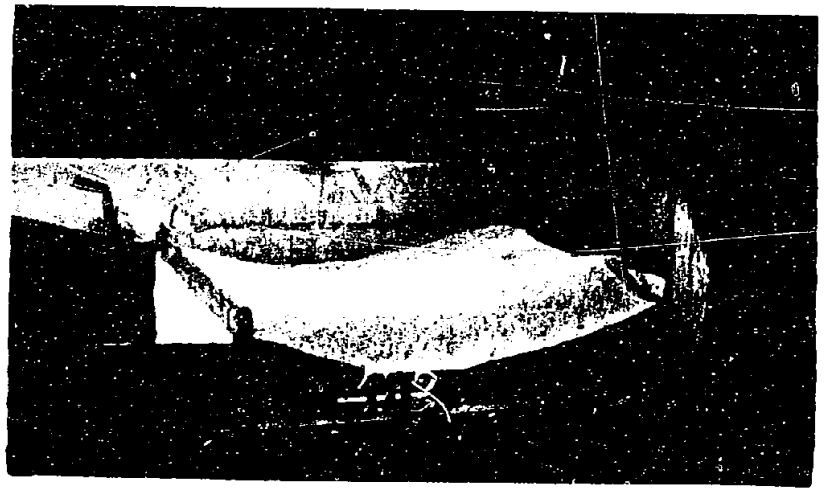


-SIDES EXPANDING NON-UNIFORMLY TO END OF POOL

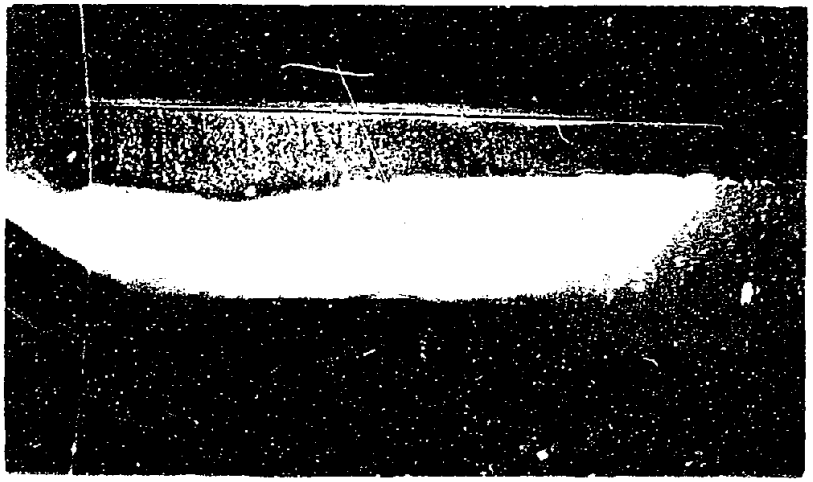




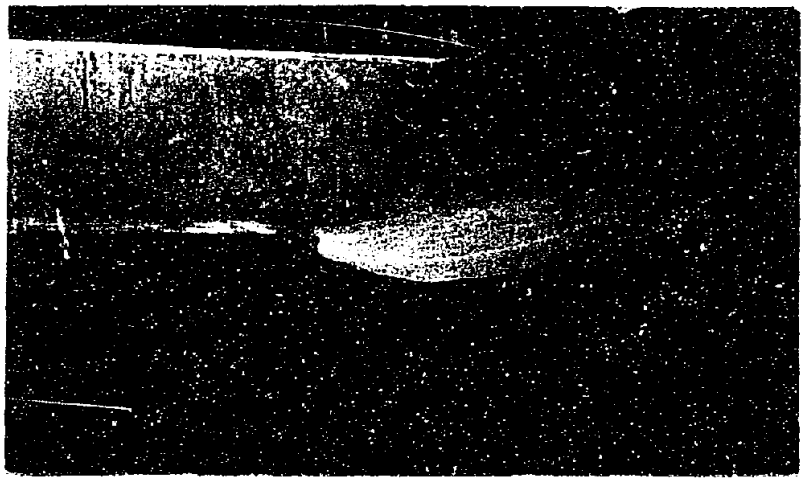


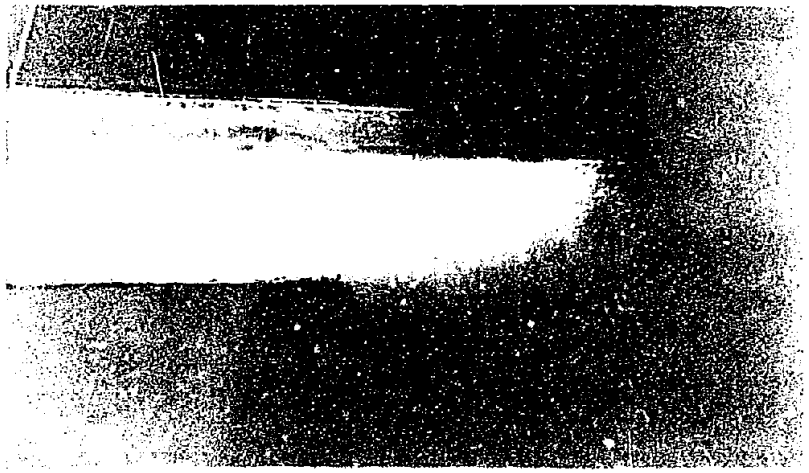


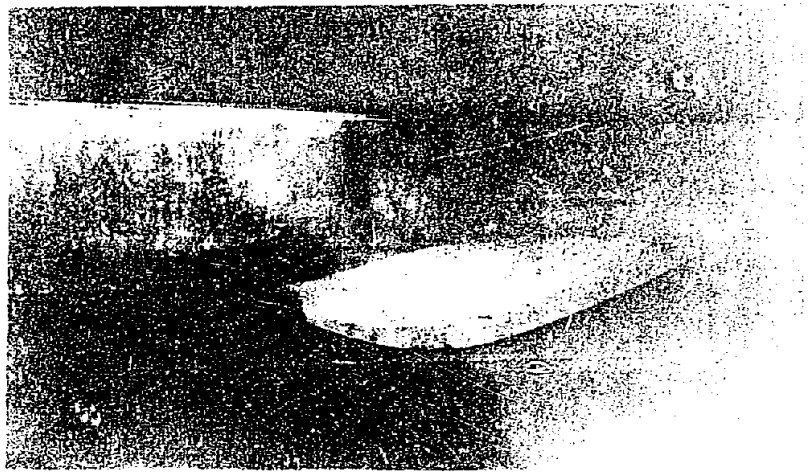
SIDES EXPANDING NON-UNIFORMLY
TO END OF POOL



A ½ SLICE (NURING RUN)

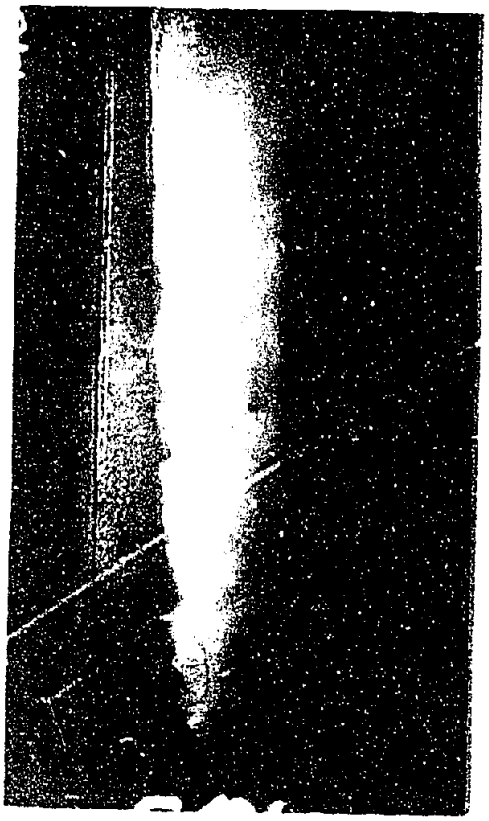


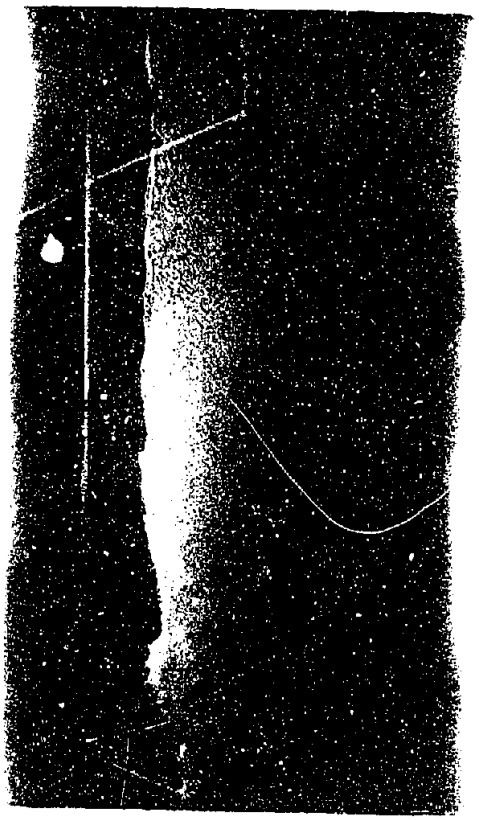




B-3:1 SLOPE AFTER RUN

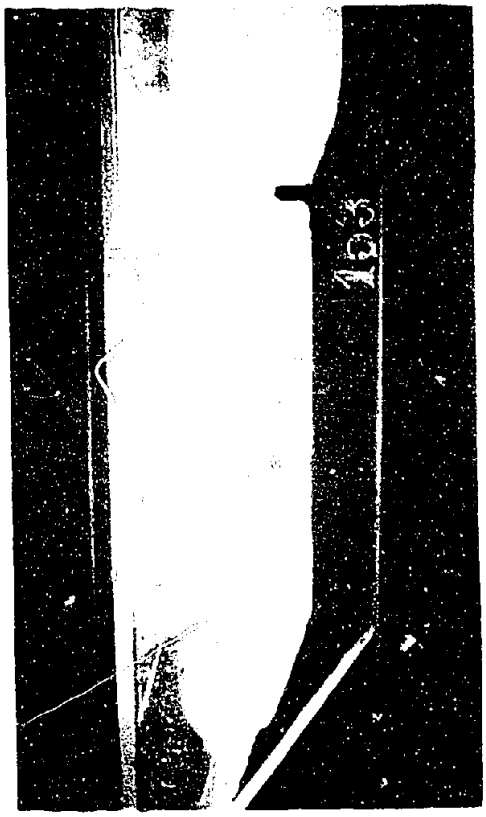
B-4:1 SLOPE-AFTER RUN



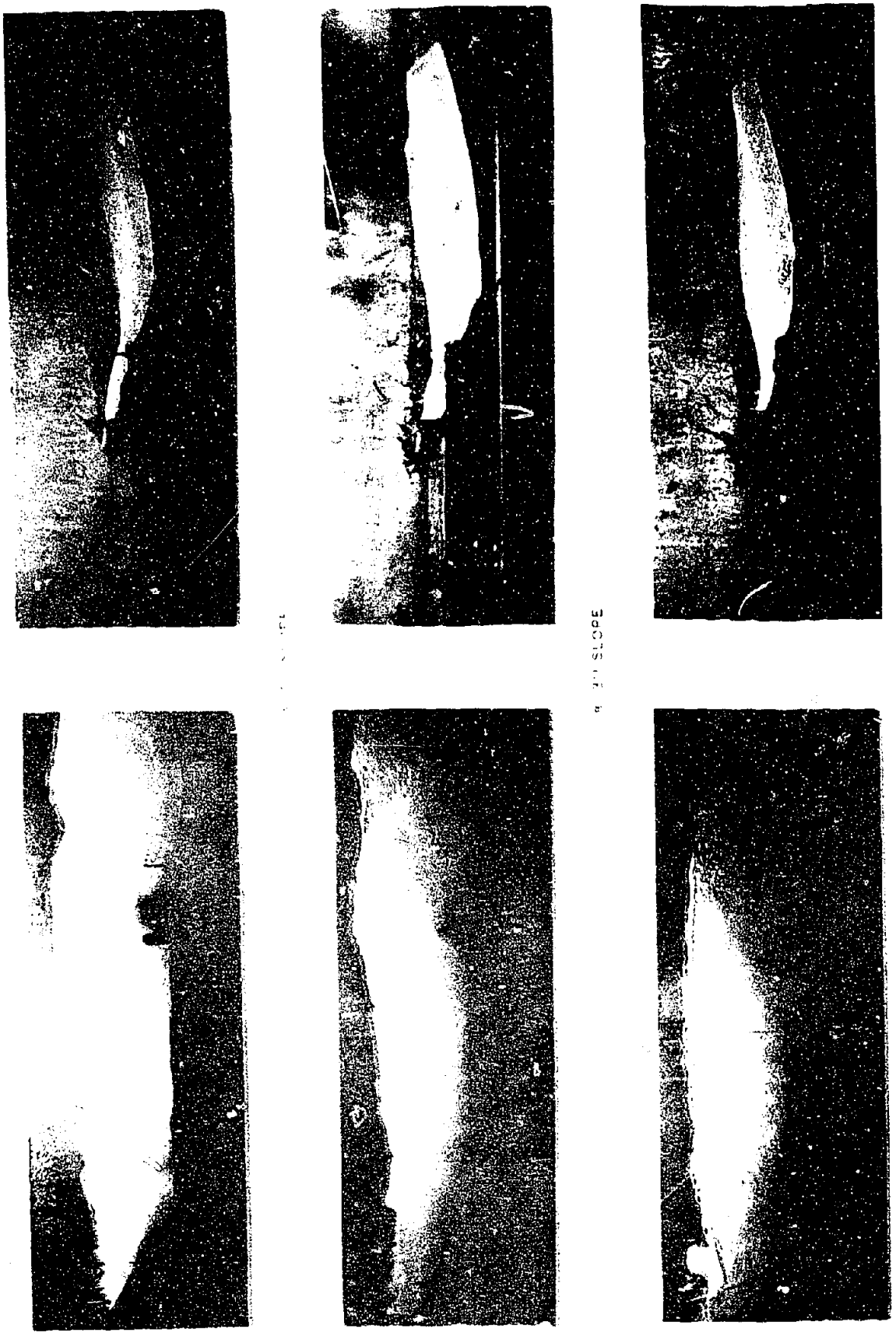


EFFECT OF SLOPE OF TROUGH ENTERING
POOL WITH NO SILL

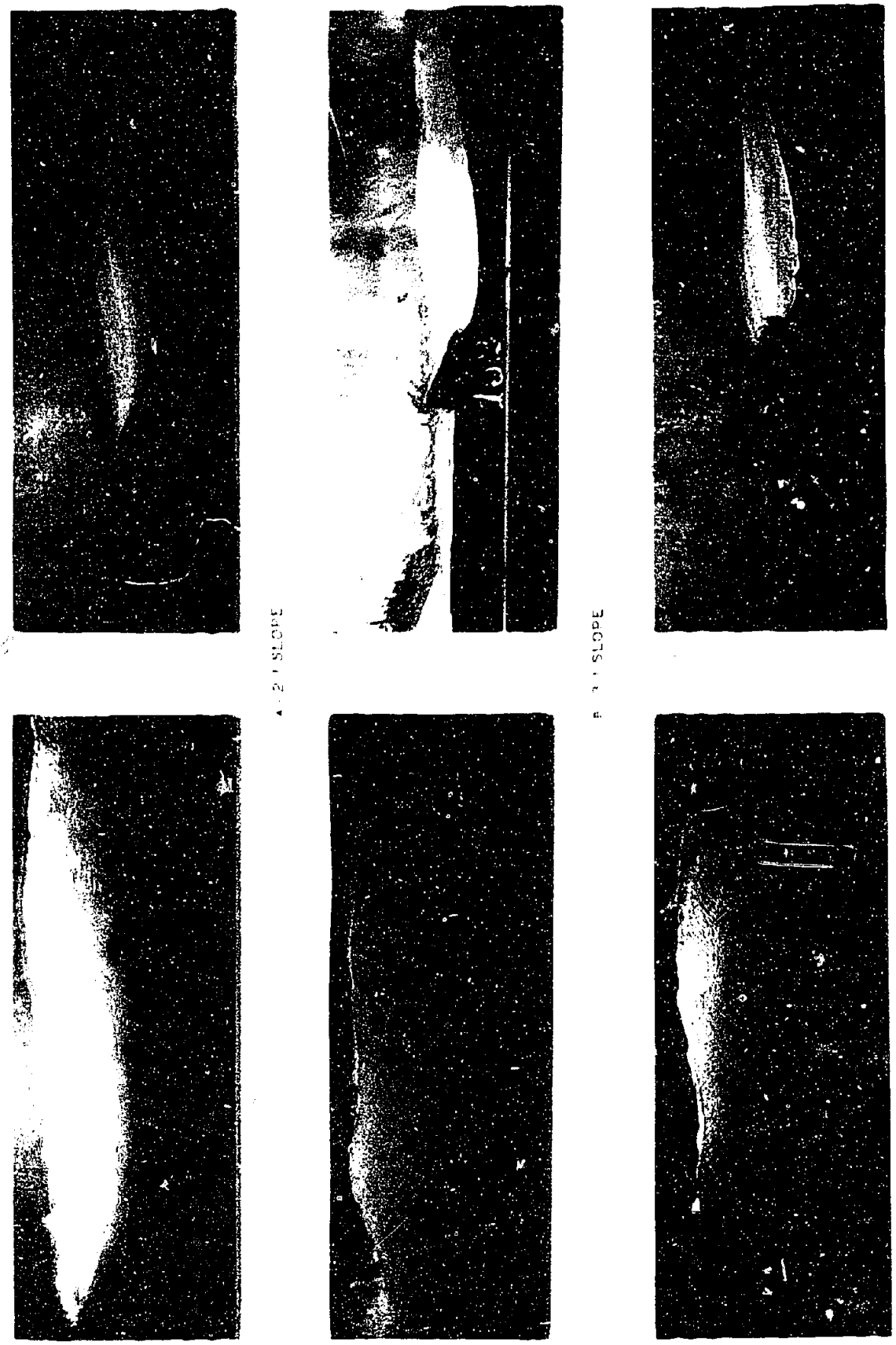
b-4:1 SLOPE - AFTER RUN



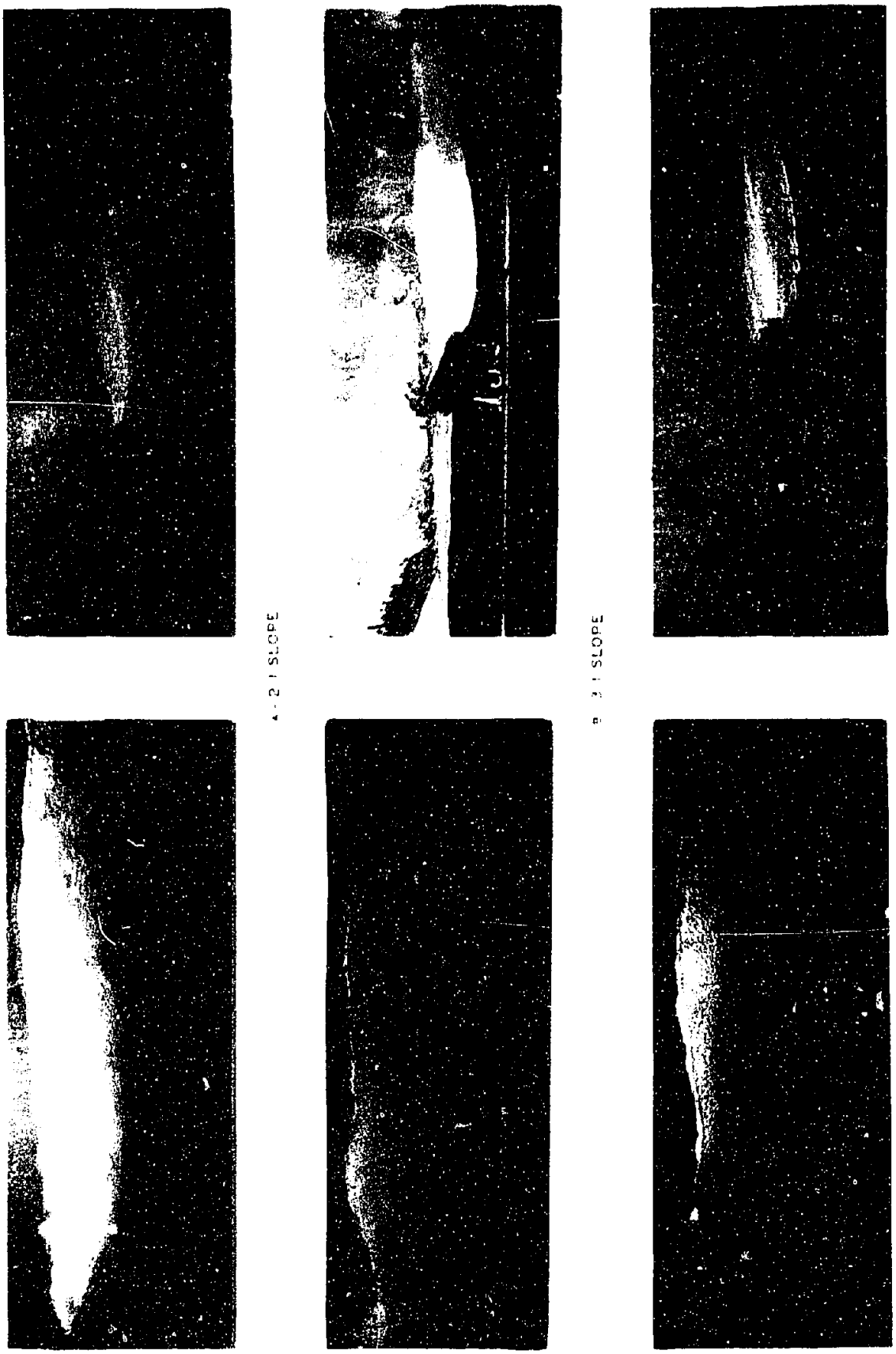
EFFECT OF SLOPE OF TROUGH ENTERING POOL WITH
PLAIN SILL AT END OF FLOOR



EFFECT OF SLOPE OF TROUGH ENTERING POOL WITH PLAIN SILL 25 FEET UPSTREAM FROM END OF FLOOR



EFFECT OF SLOPE OF TROUGH ENTERING
POOL WITH DENTATED SILL



EFFECT OF SLOPE OF TROUGH ENTERING
POOL WITH DENTATED SILL

40 ft., on the scour below the stilling pool was determined by experiment. For both sill types tried the scour was less with the transition curve than with the angle and a curve was therefore adopted for the final design. Figure 28 and Plate XXXI shows differences in scour with and without the curve using the dentated sill and Figure 29 and Plate XXXIII shows similar results with a plain sill.

Study of the Dentated Sill

Runs previously discussed showed that the bottom of the stilling pool should be placed at a level below that of the river bed and that it should be level, with some form of sill near the downstream end. In order to determine what form and location of sill was best adapted for the conditions existing at Cle Elum, extensive studies of various types of sills were made under a variety of conditions and comparisons of the results were made. The form most extensively tested was the dentated sill perfected by Dr. Rehbock.

The dentated sill was tested under a great variety of conditions. In all cases its use considerably reduced the scour at the downstream edge of the floor, as compared with that occurring with no sill. Figures 30 to 33 inclusive show the results of some of these comparisons. The first was made with the flume with the 60 ft. throat and 1½:1 slope entering the stilling pool. The remainder were made with the trough 200 ft. wide throughout. In all cases the sill was 10 ft. high. Except for the scour along the edges, as shown by the maximum scour lines, the dentated sill prevented cutting at the downstream edge of the floor. Later experiments, on the bin where the sand sides replaced the vertical walls, showed that this severe cutting along the edges would not take place in the prototype, and it is therefore safe to conclude that the dentated sill in all cases would have practically prevented severe scour at the downstream edge of the apron.

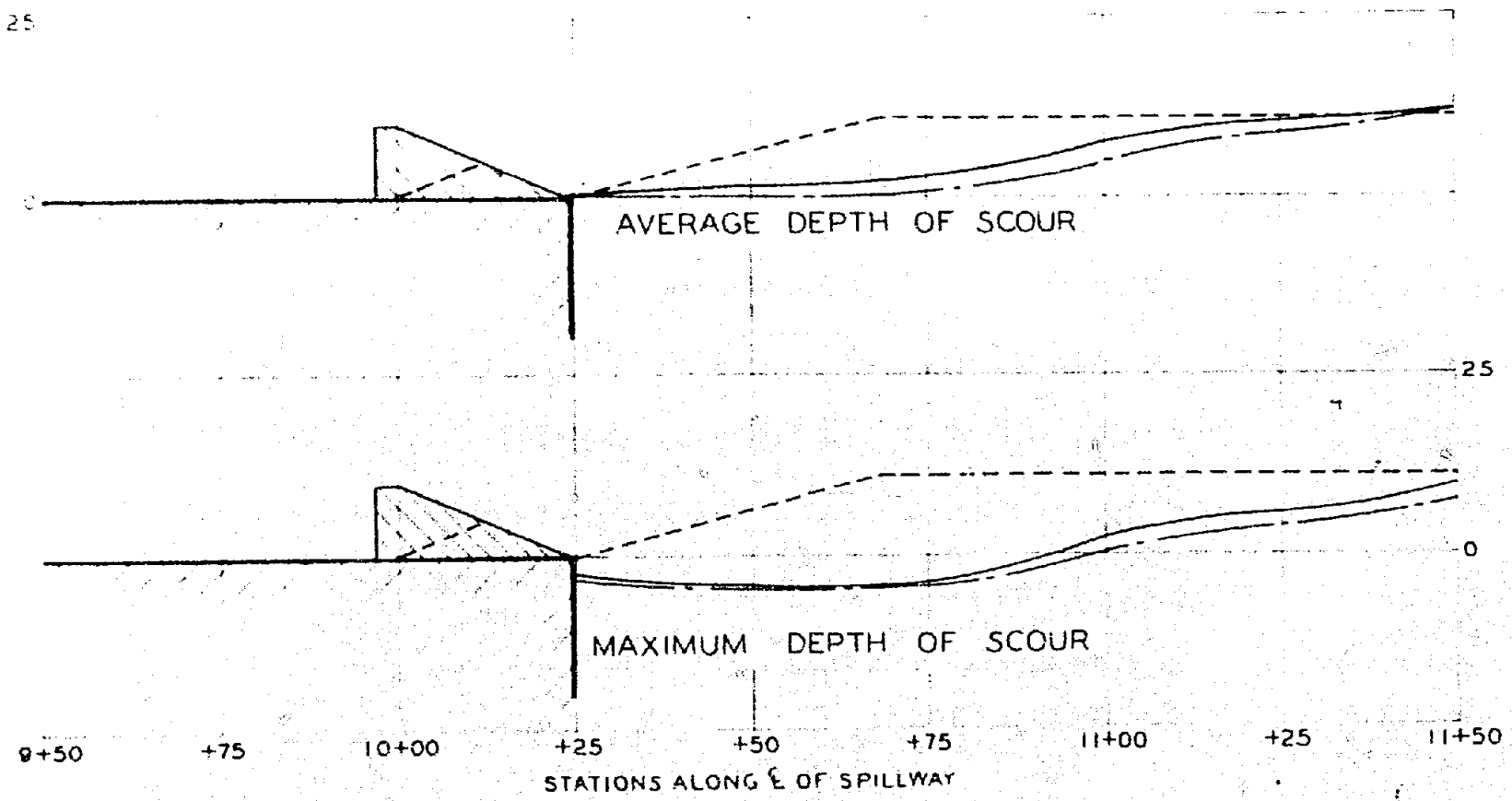
One of the first studies was for the purpose of determining the size of sill to be used. Dr. Rehbock's recommendations were for a height of about one-tenth the fall. In these tests runs were made with sills 7.5, 10.0 and 12.5 ft. high, the velocity entering the pool corresponding approximately to a 100 ft. drop. A comparison of these results is shown on Figure 34. The scour for the 7.5 and 12.5 ft. heights is shown on Plate XXXIII. Unfortunately the experiments on the 10 ft. height were not made under conditions otherwise identical with those of the other heights, but were made with the wall expansion ending at Sta. 7+84 (Figure 22) while the others were made with the fourth form. The scour may be corrected with reasonable accuracy according to the comparative results obtained with these two set-ups, as shown on Figure 28. These results show that a sill 10 ft.

high gives practically as good results as one 12.5 ft. high, and considerably better than one 7.5 ft. high. As large quantities of concrete are involved in these structures and the volume varies with the square of the height, the 10 ft. height would be much cheaper than the 12.5 ft. height, and the former was therefore adopted. The 7.5 ft. height was not used, not only because it permitted somewhat undesirable scour but also because it was desirable to have a considerable mass of concrete in the dentates in order that they might withstand possible blows from logs passing over the spillway.

Another study was made to determine the shape of this sill adjacent to the side walls. Available literature from Dr. Rehbock shows more than one form of sill end and these with other possible forms were tested. Figures 35 and 36 show the dimensions of these ends. The first of these (A in Figure 35) has a tooth at the end half as wide as the others. The second, B, has a tooth 1½ times the normal width with a secondary block on top, to cut down the eddies which seem to be set up in contact with the side wall, which effect has been mentioned in connection with the excessive scour along the glass wall of the model sand pit. The third from (C of Figure 36) is the same as B with the secondary block removed. The fourth, D, has a space adjacent to the wall instead of a tooth. As the effect of these ends would be evident in the scour on the side slopes of the earth channel below the stilling pool, as well as on the bottom, these tests were made with a sand bin where the model of the channel sides were of sand rather than being represented by the glass wall of the flume.

The result of these tests is shown on Figure 37. The first two sets of cross sections shows the shape of the sand bottom before and after the run for sills with ends of types A, B and C at the downstream end of the floor and 100 ft. below that point. The third set of cross sections was made for types C and D at the downstream edge of the floor. Conditions further downstream were not strictly comparable. The result of these comparisons does not show any marked superiority of one type over the other, indicating that any of these forms would be satisfactory. The choice of type C for most of the remaining experiments and for use in the prototype was therefore somewhat arbitrary.

To determine the magnitude of the pressures exerted by the swiftly moving water on the dentated sill, in order to determine the feasibility of using it, piezometers were installed at various points in it as shown on Figure 38 and observations of the pressures at these points were made. The sill used was 10 ft. high and the flow conditions were as in the final design except that other sill positions 12½ ft. further upstream and downstream were also tested, the floor length being varied to accommodate these changes. The results



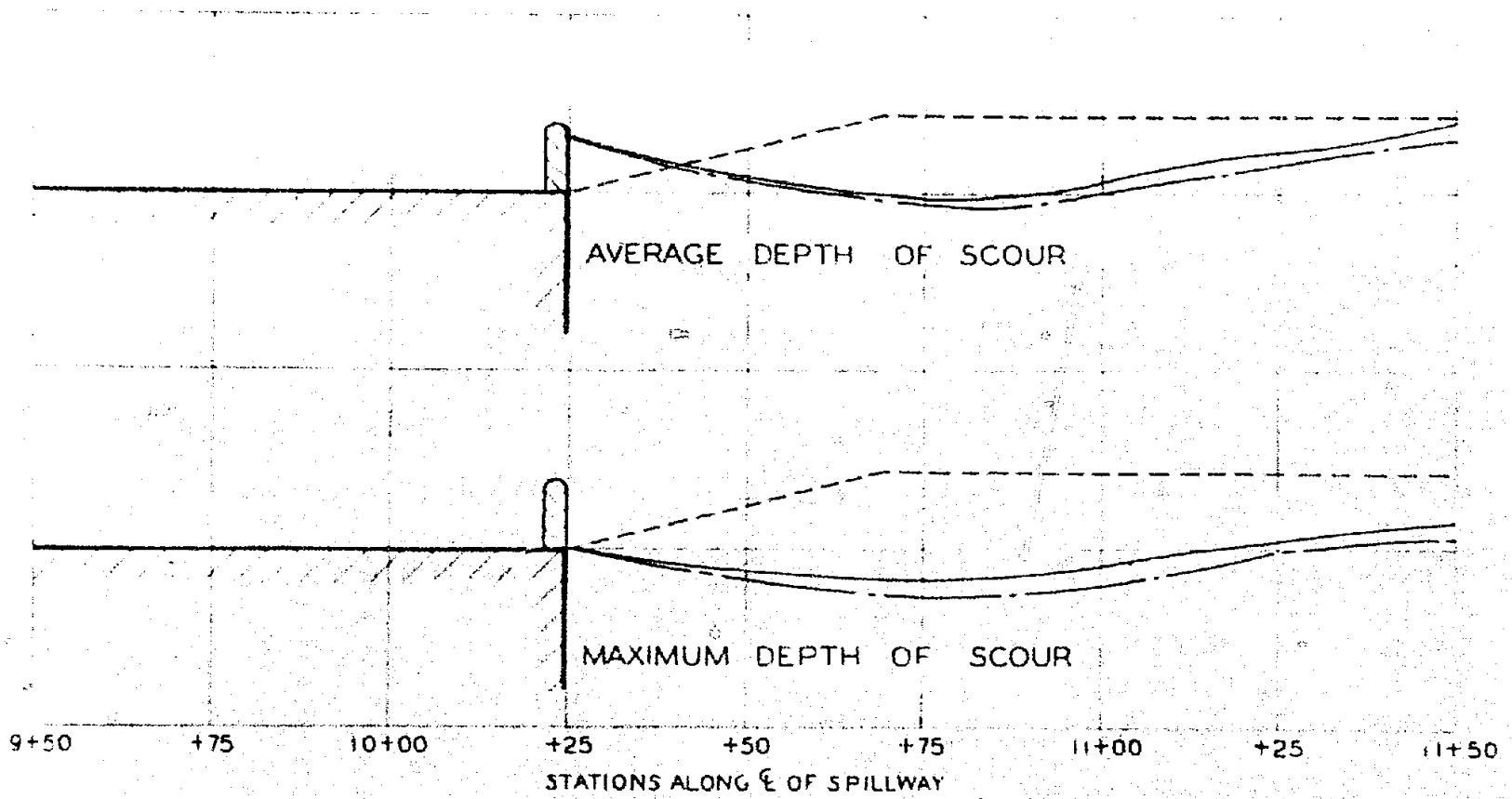
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF CURVE AT BOTTOM OF SLOPE
FLUME SLOPE 1½:1 THROAT 100 FT.

EXPLANATION

- - - - - - - - SAND BEFORE RUN
———— CURVE AT BOTTOM OF SLOPE
——·—— NO CURVE AT BOTTOM

0 25 50
PROTOTYPE SCALE OF FEET

FIGURE 28



EXPLANATION

- - - - - - - - SAND BEFORE RUN

———— CURVE AT BOTTOM OF SLOPE

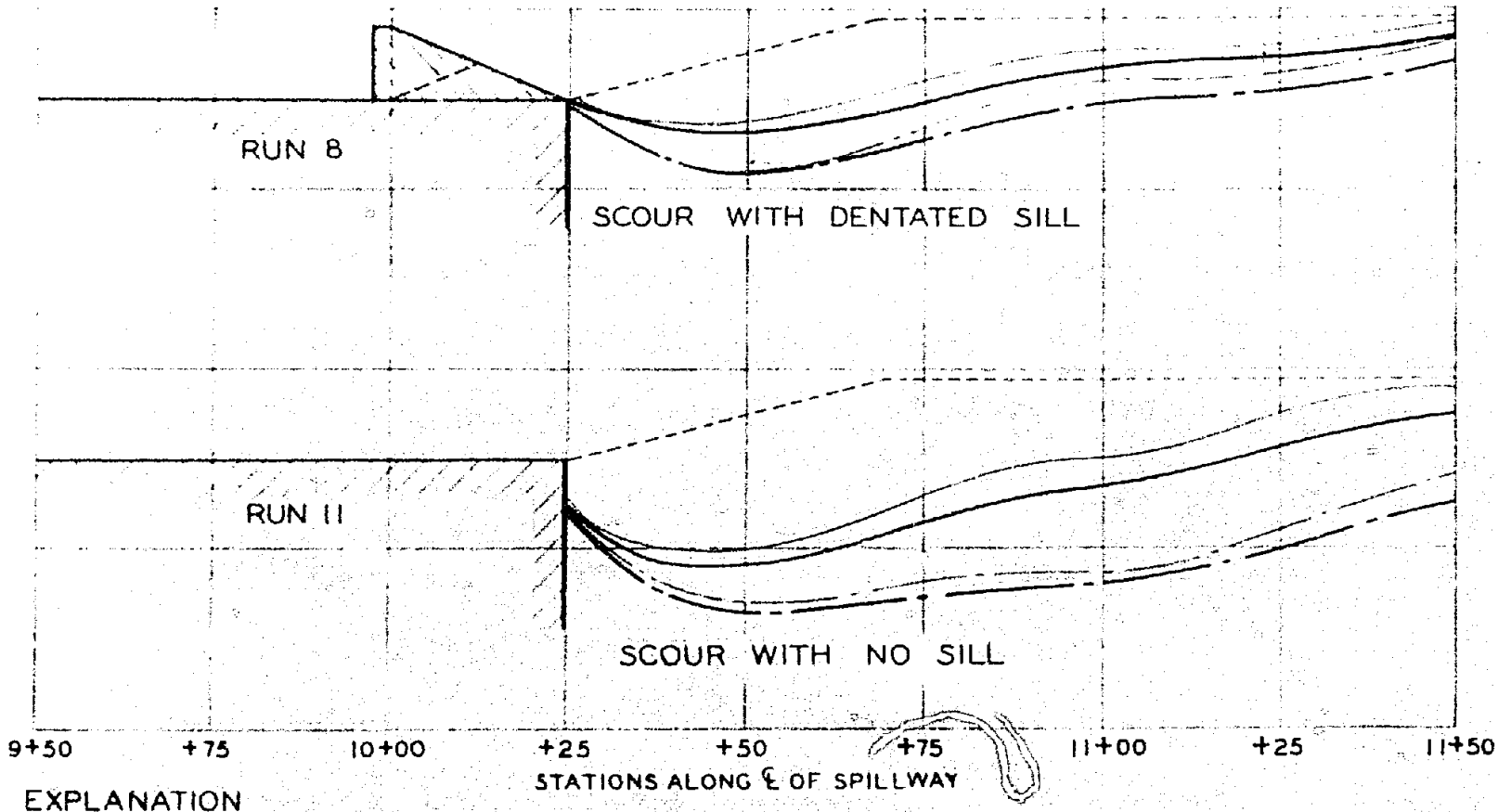
—— · —— NO CURVE AT BOTTOM

0 25 50

PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF CURVE AT BOTTOM OF SLOPE
FLUME SLOPE 1½:1 THROAT 100 FT
EXPANSION ENDS AT STA. 7+84

FIGURE 29



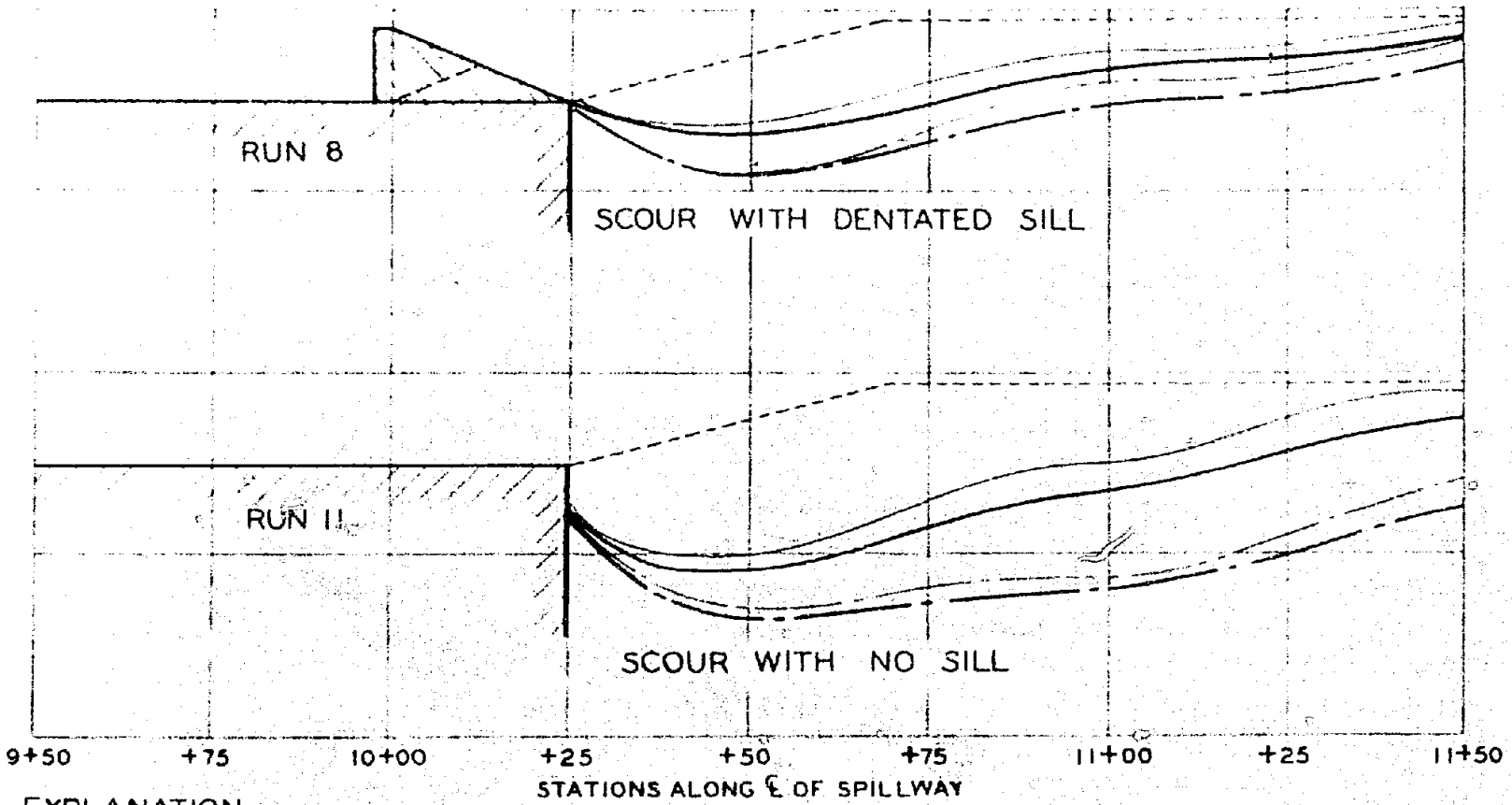
EXPLANATION

- - - - - - SAND BEFORE RUN
AV DEPTH 30 MIN. RUN
——— AV DEPTH 60 MIN. RUN
MAX. DEPTH 30 MIN. RUN
—·— MAX. DEPTH 60 MIN. RUN

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF DENTATED SILL
TROUGH SLOPE $1\frac{1}{2}:1$
60 FT. THROAT

FIGURE 30



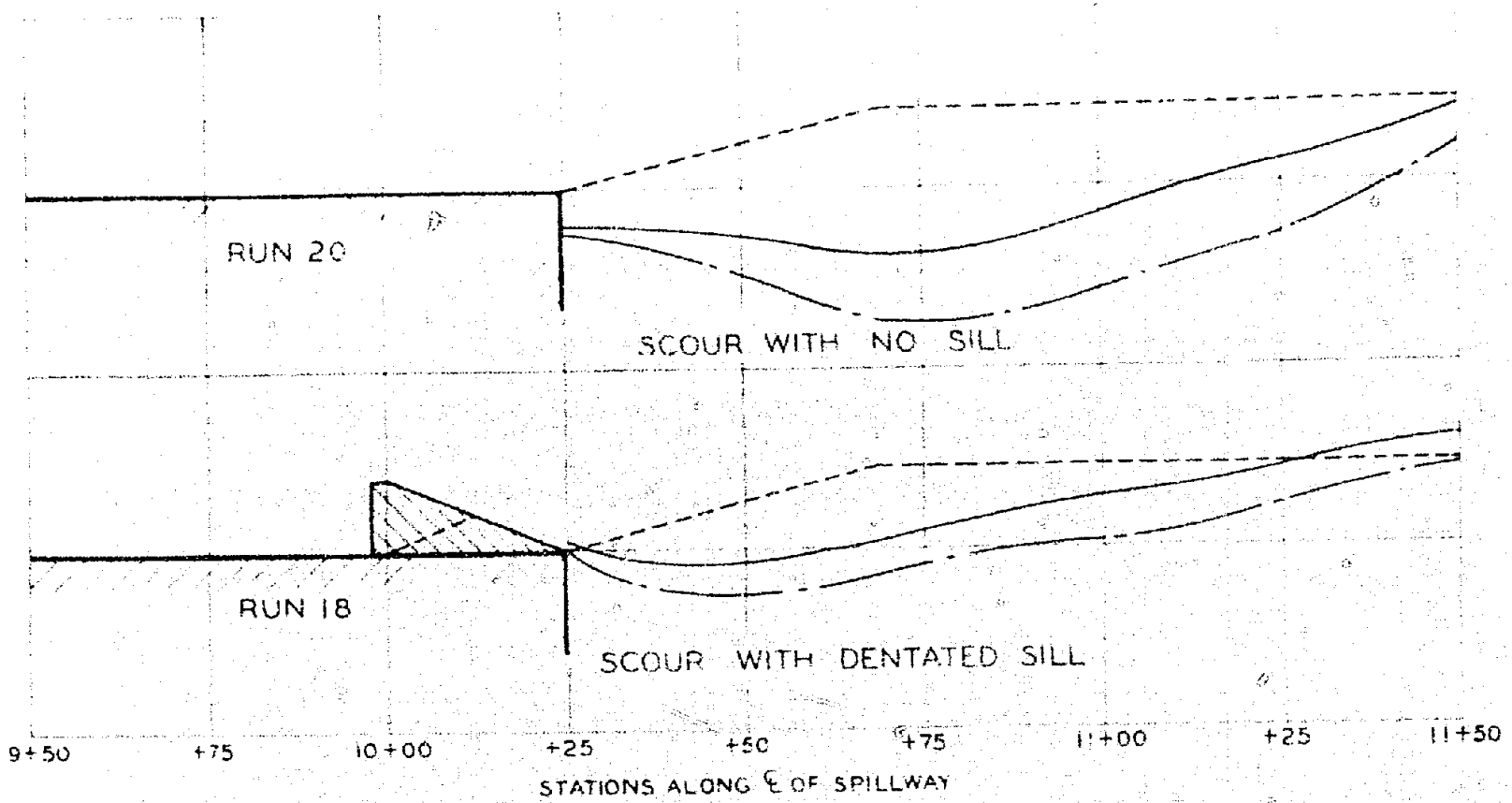
EXPLANATION

- - - - - - SAND BEFORE RUN
AV. DEPTH 30 MIN. RUN
——— AV DEPTH 60 MIN. RUN
MAX DEPTH 30 MIN. RUN
—·— MAX. DEPTH 60 MIN. RUN

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF DENTATED SILL
TROUGH SLOPE $1\frac{1}{2}:1$
60 FT. THROAT

FIGURE 30



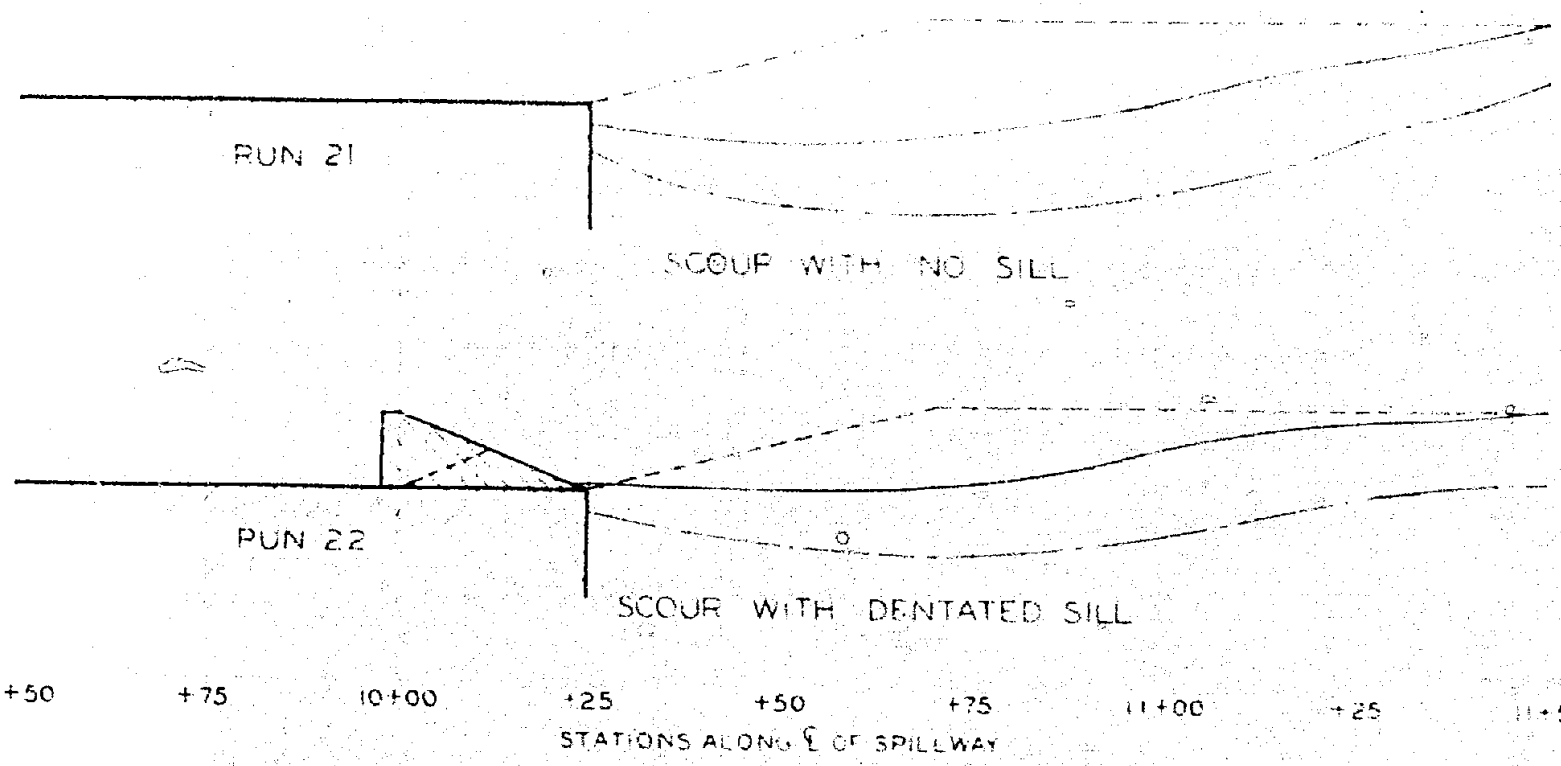
EXPLANATION

- - - - - - SAND BEFORE RUN
——— AV DEPTH 60 MIN. RUN
——·—— MAX. DEPTH 60 MIN. RUN

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF DENTATED SILL ON SCOUR
TROUGH SLOPE 2:1

FIGURE 31



EXPLANATION

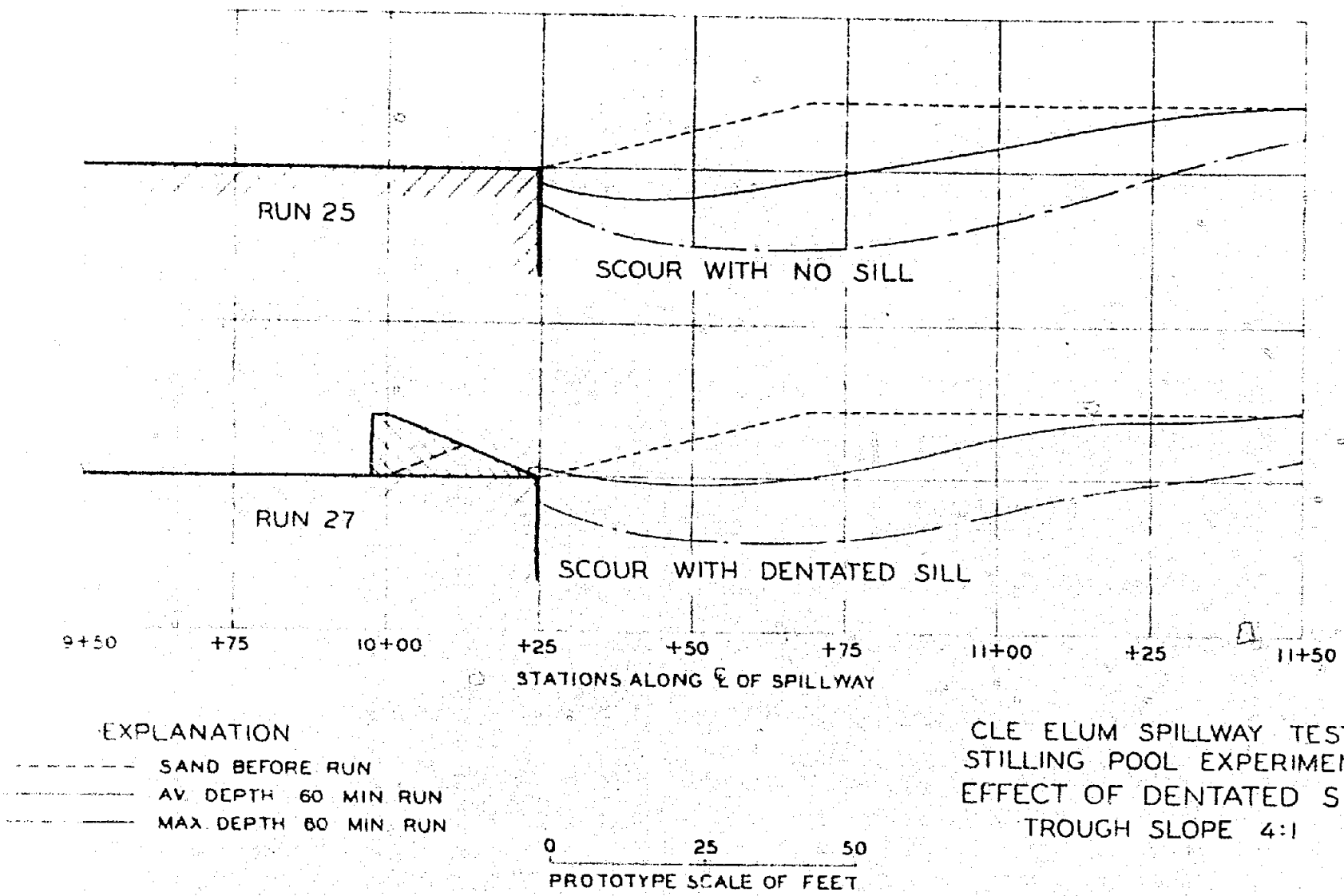
- SAND BEFORE RUN
- AV. DEPTH 60 MIN. RUN
- MAX. DEPTH 60 MIN. RUN

0 25 50

PROTOTYPE SCALE OF FEET

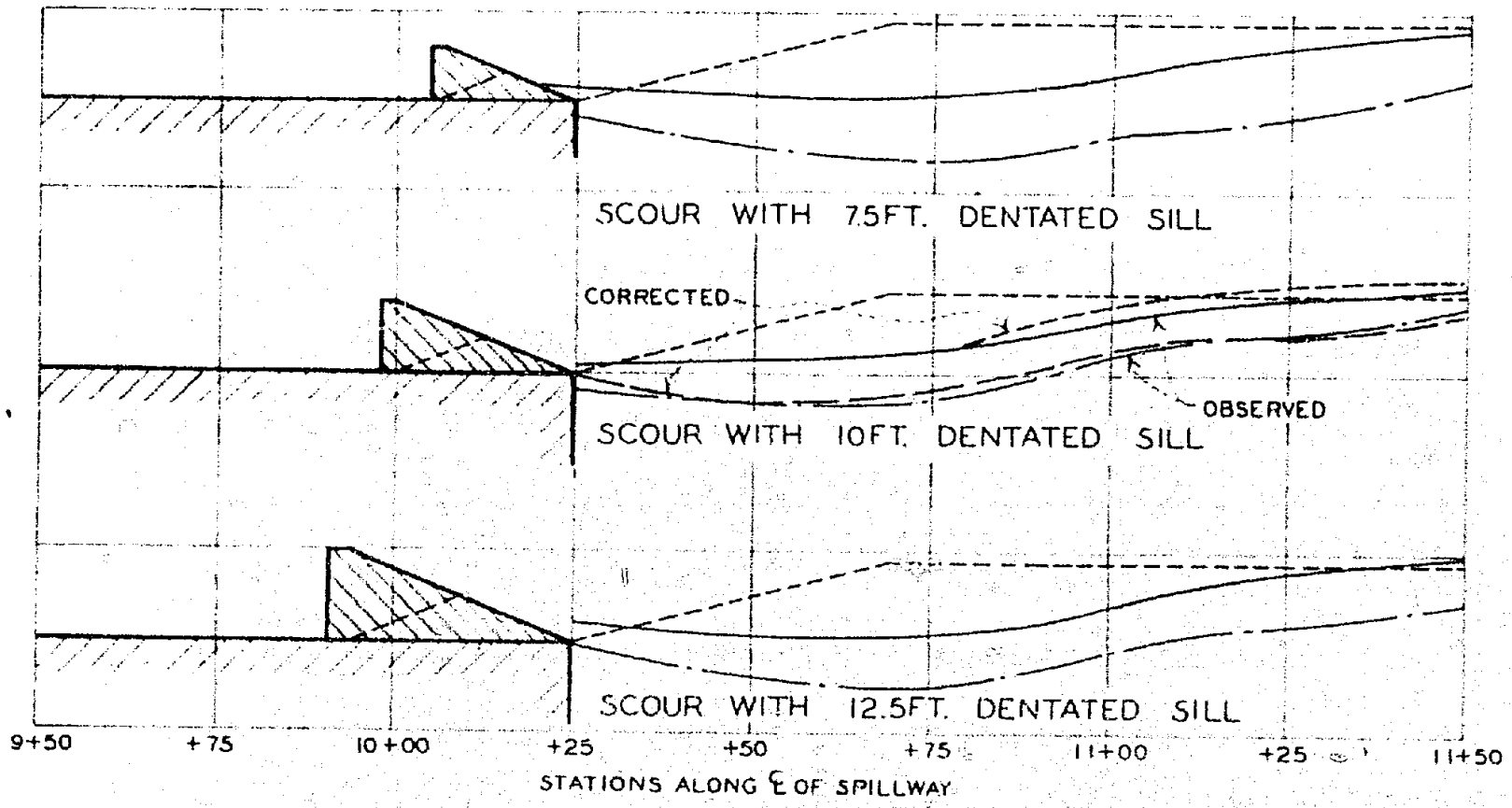
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF DENTATED SILL
TROUGH SLOPE 3:1

FIGURE 32



CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF DENTATED SILL
TROUGH SLOPE 4:1

FIGURE 33



EXPLANATION

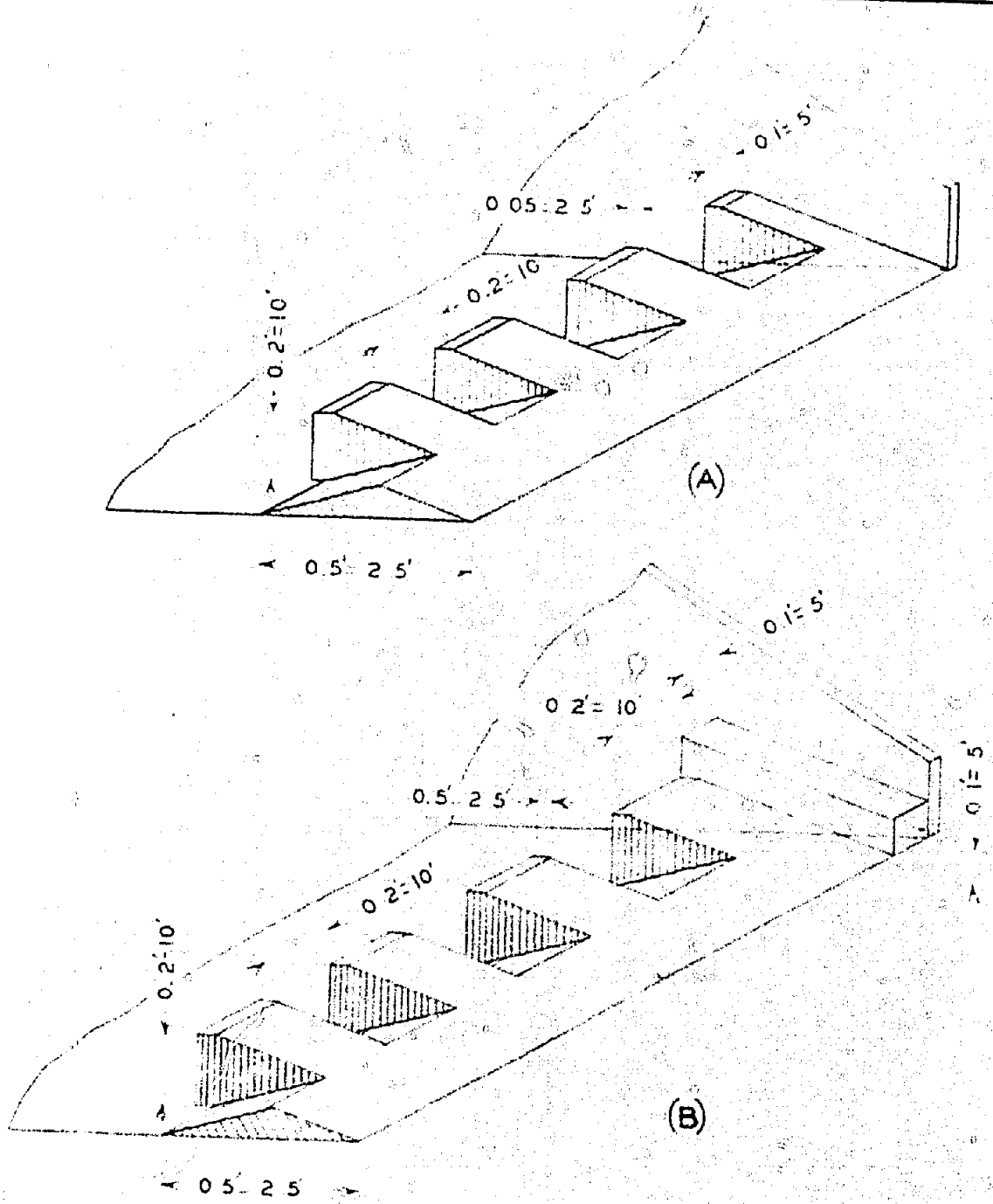
- - - - - - SAND BEFORE RUN
——— AV. DEPTH 60 MIN. RUN
—·—·— MAX. DEPTH 60 MIN. RUN

0 25 50
PROTOTYPE SCALE OF FEET

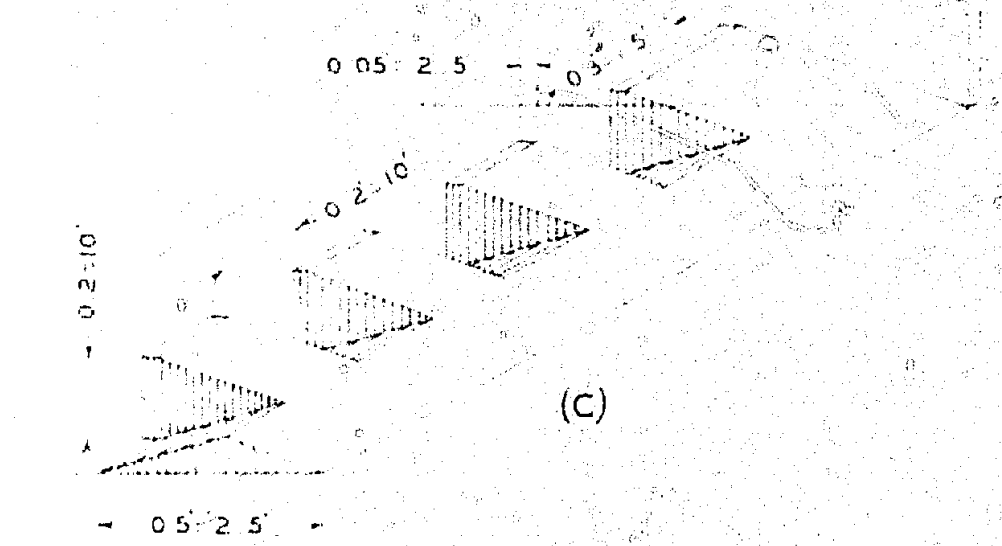
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF SIZE OF DENTATED SILL
TROUGH SLOPE $1\frac{1}{2}$:1, SET UP B

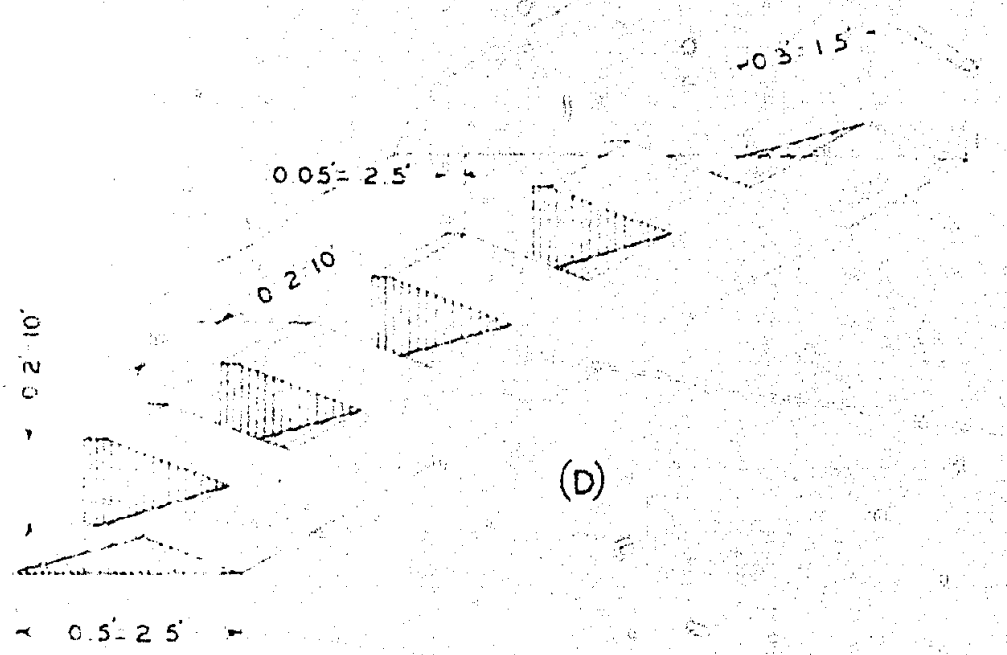
FIGURE 34

FIGURE 35



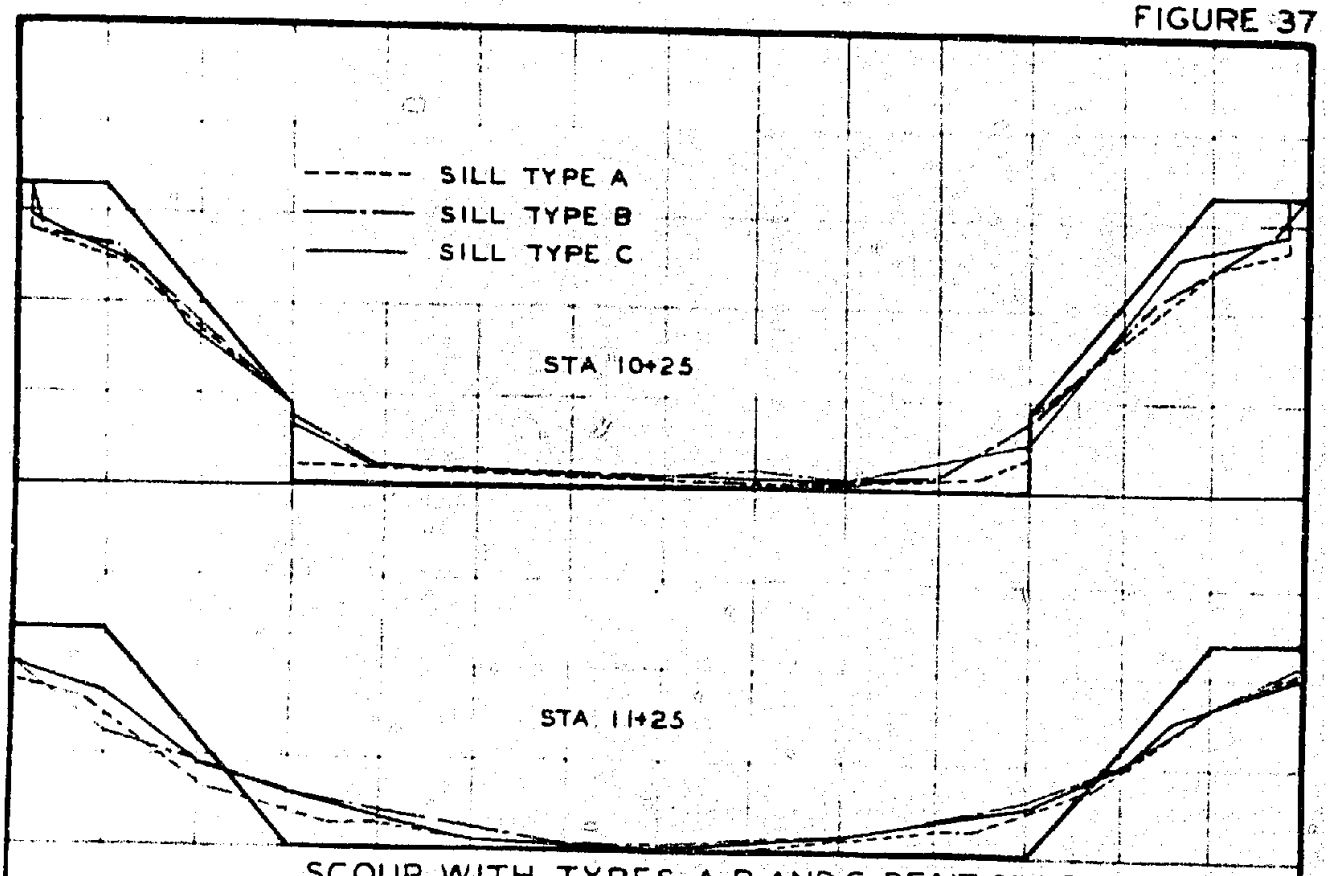
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
TYPES OF DENTATED SILLS INVESTIGATED



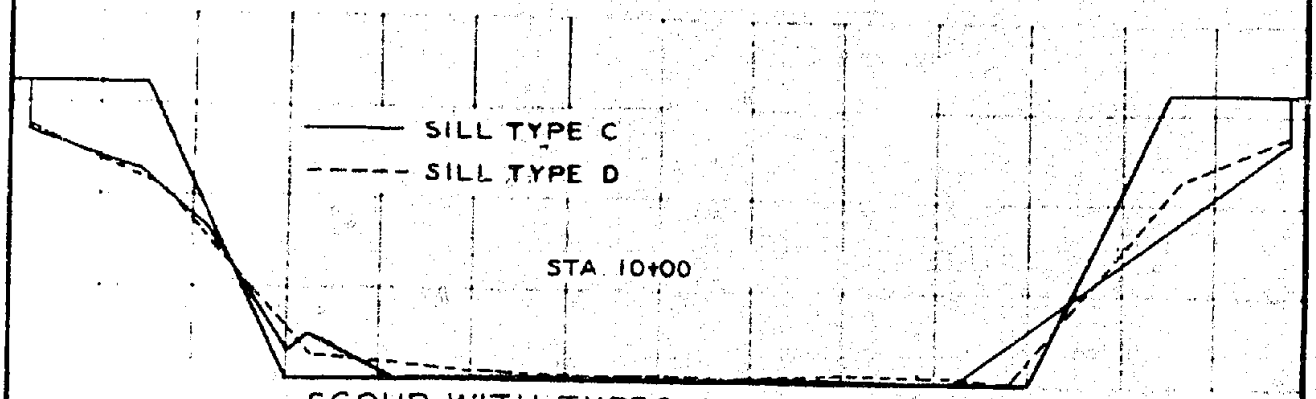


CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
TYPES OF DENTATED SILLS INVESTIGATED

FIGURE 37



SCOUR WITH TYPES A B AND C DENT. SILLS
FLOOR ENDS 10+25 TYPE I END WALLS

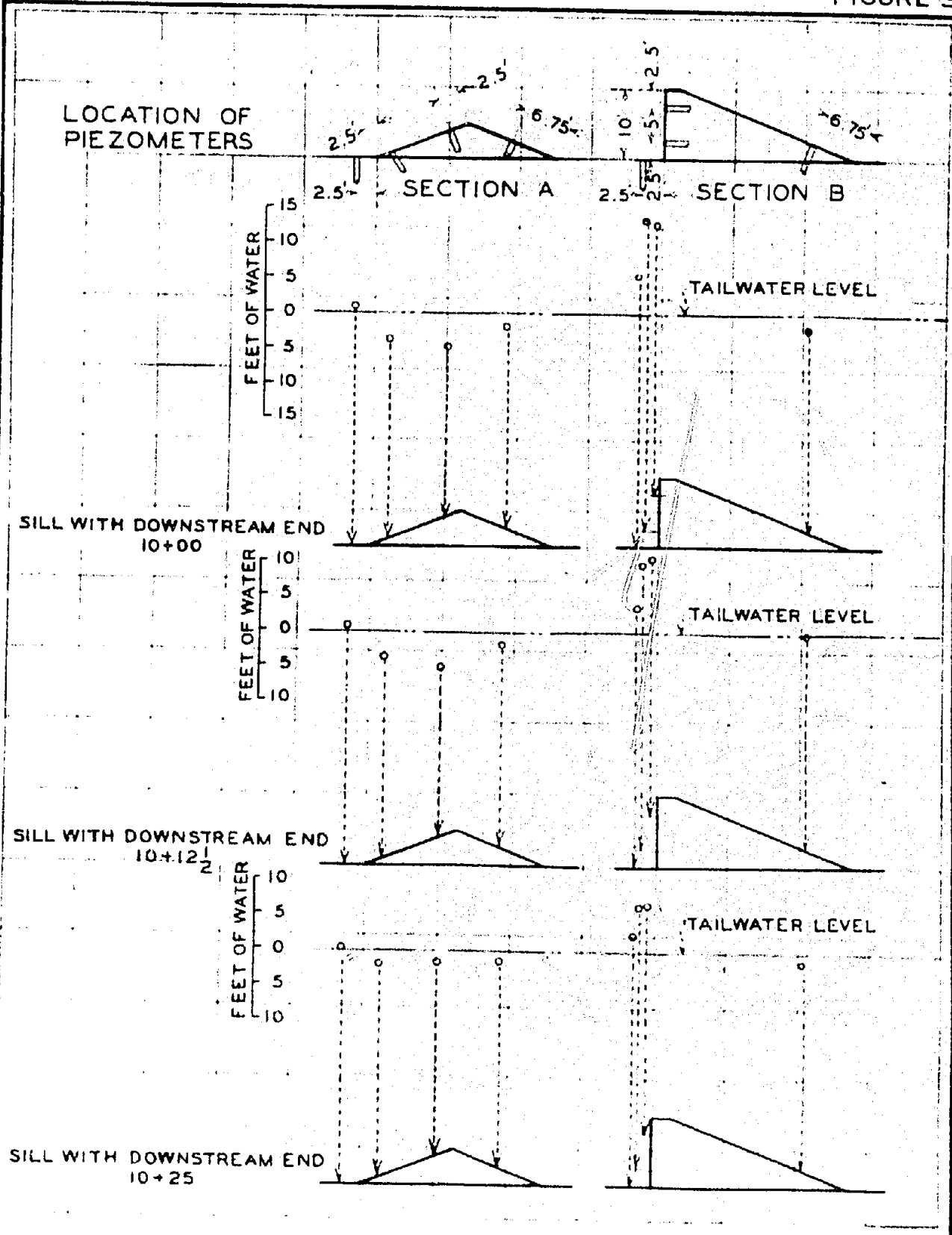


SCOUR WITH TYPES C AND D DENT. SILLS
FLOOR ENDS 10+00 TYPE II END WALLS

0 25 50
VERTICAL SCALE OF FEET
0 25 50 75 100
HORIZONTAL SCALE OF FEET

CLE ELUM SPILLWAY TESTS
COMPARATIVE SCOUR
DENTATED SILL - VARIOUS TYPES

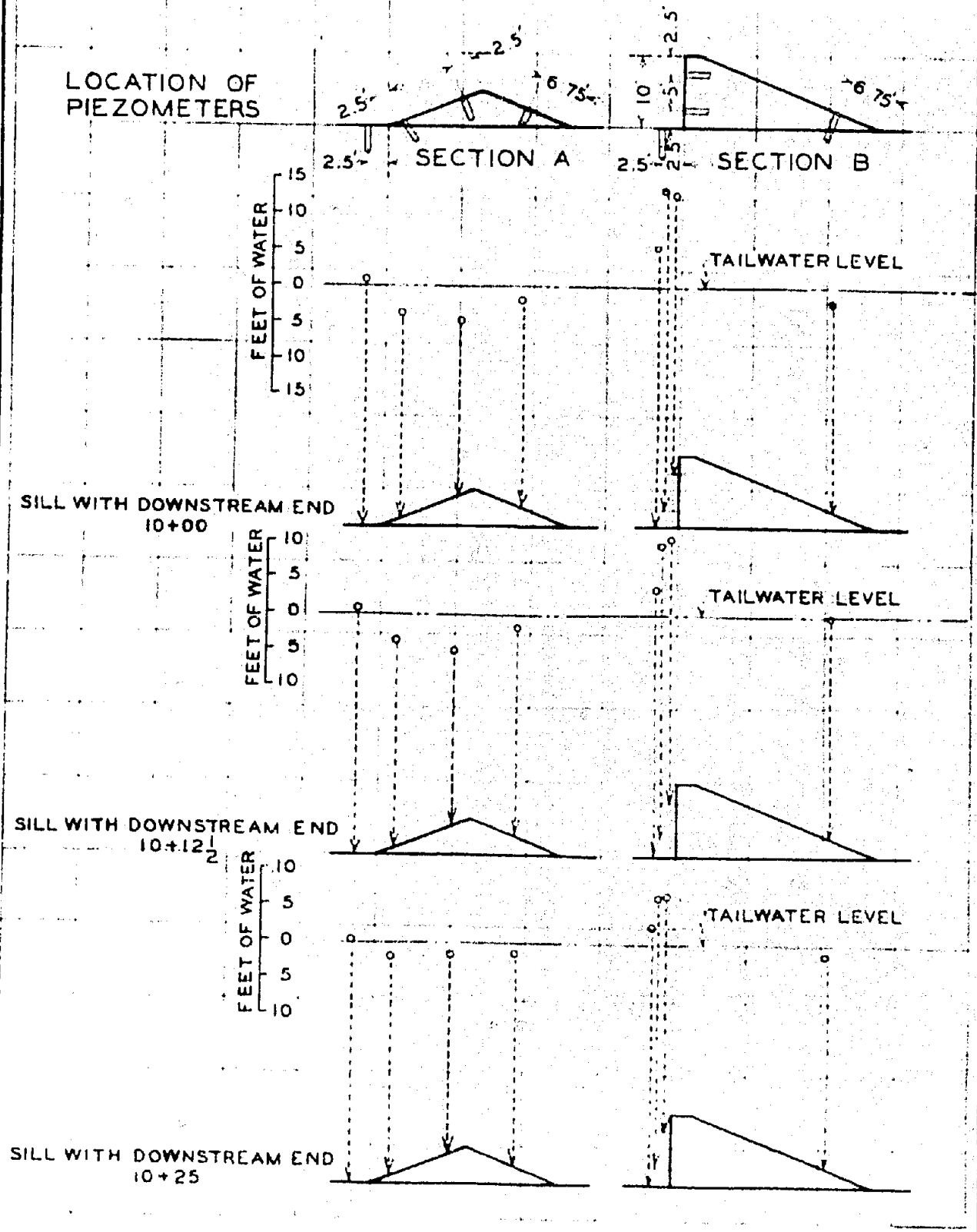
FIGURE 38



CLE ELUM SPILLING TESTS

WATER PRESSURES ON THE DENTATED SILL

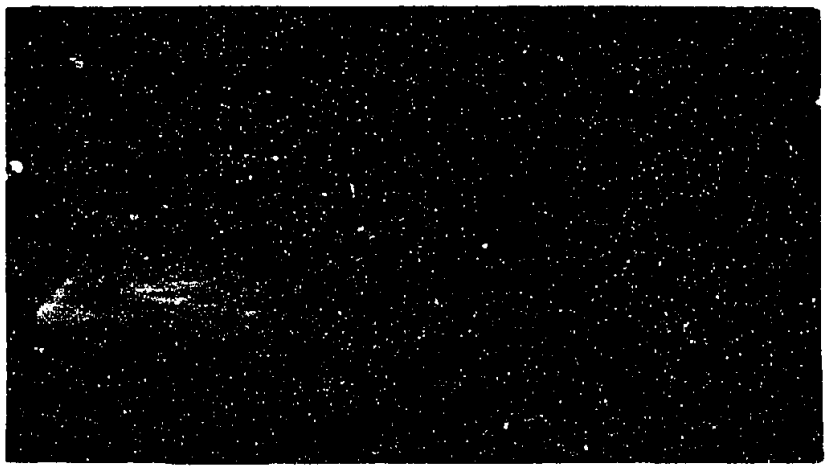
FIGURE 38



CLE ELUM SPILLING TESTS

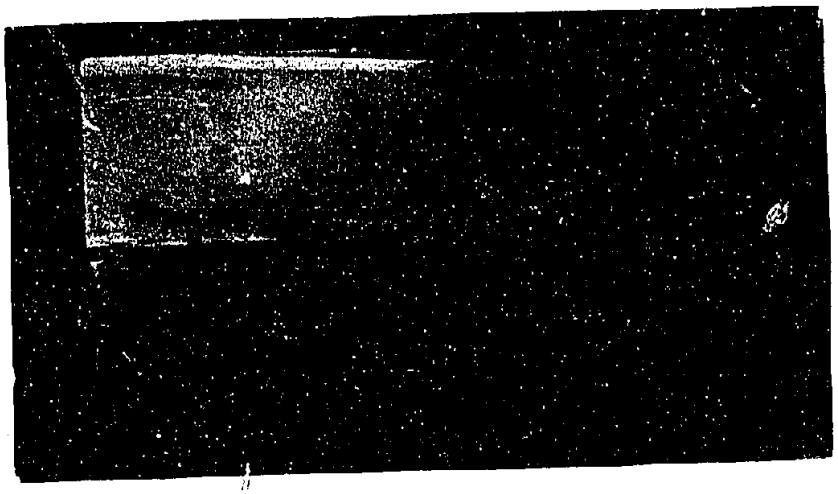
WATER PRESSURES ON THE DENTATED SILL

A-WITHOUT CURVE

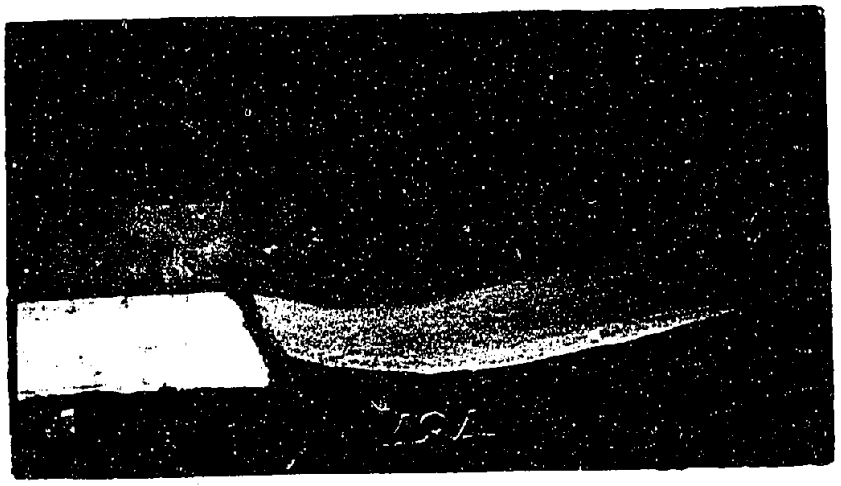


B-WITH CURVE

EFFECT OF CURVE AT BOTTOM OF SLOPE INTO
STILLING POOL WITH DENTATED SILL

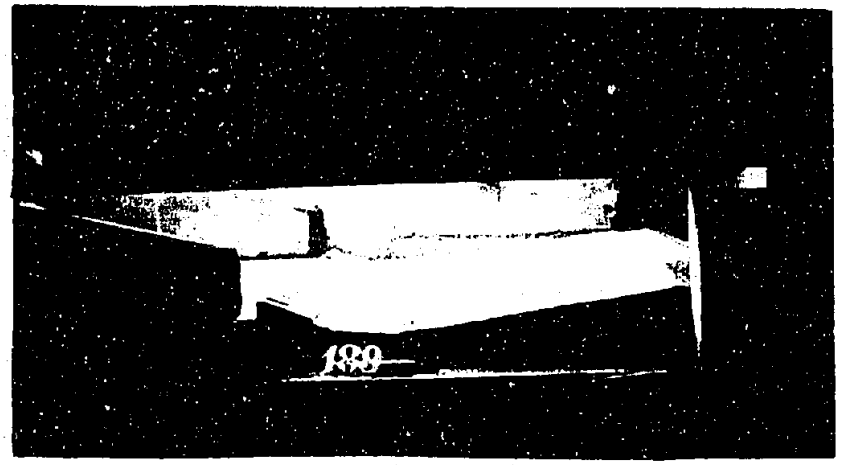


A - WITHOUT CURVE

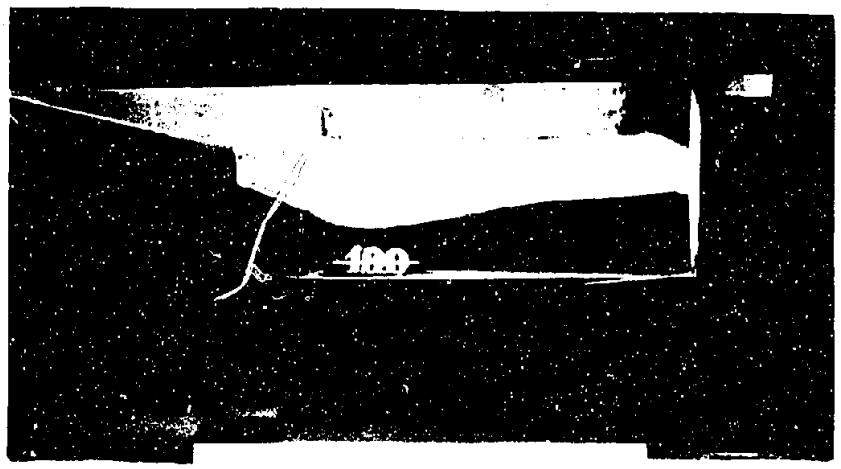


B - WITH CURVE

EFFECT OF CURVE AT BOTTOM OF SLOPE INTO STILLING POOL WITH PLAIN SILL AT END OF FLOOR



A - SILL 7.5 FT. HIGH



B - SILL 12.5 FT. HIGH

SCOUR WITH VARIOUS SIZES OF DENTATED SILLS

locations for various setups is shown on Figures 43 to 45 inclusive. The first of these comparisons was for a 60 ft. throat flume with 1½:1 slopes leading into the pool, and the remainder for a 200 ft. trough with slopes into the pool as indicated. In all cases the sill was 10 ft. high. The results show that there was less scour in each case with the sill at the downstream edge of the floor than 25 ft. above it.

A comparison of the effect of sills of various heights was also made with the results given on Figure 46 and Plate XXXIV. It showed that the scour decreased as the height of the sill was increased. It is probable that the 7.5 ft. height sill would give satisfactory results in the prototype, but to keep the results comparable with those on the dentated sill, the 10 ft. height was used in most of the runs.

A series of runs was also made to determine whether decreased scour would result if the height of the sills at the ends was increased as shown on Plate XXXV. The results showed slight improvement in some cases but the height of the sills would be so great, (approximately 20 ft.) that they were impractical from a structural standpoint, and therefore would not be desirable.

Although the location of the plain sill at the downstream edge of the floor was well adapted to the conditions at the Cle Elum Dam, it might not be advantageous in some other locations. Where the tailwater is sufficiently low so that water falls after passing over the sill, scour would be set up at the downstream edge of the floor which might undermine the floor. With the floor and sill at a very low elevation with respect to the tailwater level as at Cle Elum, no such drop existed, and therefore the plain sill in such a location could be used to advantage. In order to establish definitely that no undesirable scour would take place below the sill, runs covering discharges less than the maximum were made, with their corresponding tailwater levels. The results of the scour obtained under these conditions are shown on Figure 47.

Study of the Beveled Sill

A few tests were made on a sill 10 ft. high, the upstream face of which was beveled at a 45° angle with the floor as shown on Plate XXXVI. It was believed that the reaction of the force necessary to lift the high velocity water up over the sill would increase the stability of this type of sill. Figure 48 shows a comparison of the scour caused by this sill as compared with that under otherwise identical conditions using the dentated sill. At the downstream edge of the apron the scour on the bottom of the channel with the beveled sill was less than with the dentated sill, due to the eddy which tends

to bring the sand upstream and pile it against the downstream side of the sill as was the case with the plain sill. With the beveled sill there was considerably more scour on the sides of the channel and the scour about 100 ft. downstream from the end of the floor was greater on both bottom and sides of the channel.

In some respects this sill resembled that developed by the model tests for the lower end of the Madden Dam apron. From those tests it was decided that the sill should be small, having only sufficient height to deflect the high velocity current off the river bed. This conclusion is not applicable to the conditions at Cle Elum however as the pool floor at Cle Elum was below the river bed, and considerably greater deflection of the currents would have been necessary to make them clear the river bed. For this purpose a higher sill would have been required than the Madden experiments indicated to be desirable.

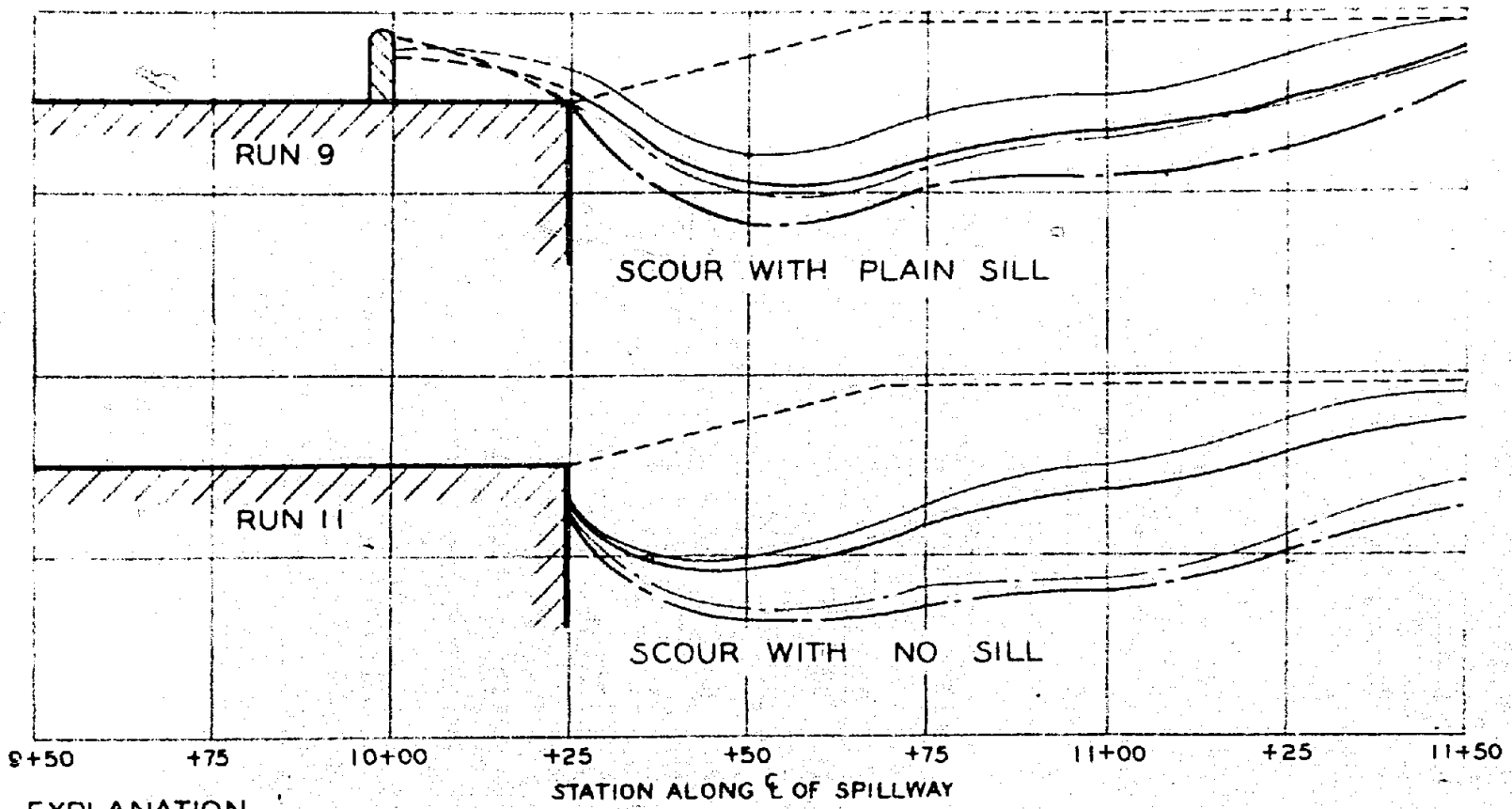
Basis of the Selection of the Dentated Sill

A comparison of runs made on the dentated sill and on the plain sill at the end of the floor, using the pool with glass sides, is shown on Figures 49 to 55 incl. In general the average depth of scour is less with the dentated sill than the plain sill, although due to the backroll, the scour at the downstream edge of the floor is less with the plain sill. The maximum depth of scour was generally less with the dentated sill. The results of comparative tests with the larger sand pit are shown on Figure 56. These results are quite similar to those in the pool with glass sides. The dentated sill usually gave somewhat less scour on the sides and bottom, except that due to the backroll, a fill was built by the plain sill at the downstream edge of the flow, which gave this critical point better protection than with the dentated sill.

From the hydraulic standpoint therefore the dentated sill showed up somewhat better than the other types experimented upon, but it is believed that reasonably satisfactory results would have been obtained by the use of several of the other types of sill. Since this bureau has permission to use the dentated sill on any of its structures, the question of royalties was not a factor in the decision. A structural advantage of the dentated sill is that the pressure on it would be distributed over a wide base, which would be an advantage on earth foundation.

Selection of Best Combination of Pool Width, Length and Depth

The width of the stilling pool of the preliminary design of the spillway was somewhat arbitrarily set at 200 ft. and the pool depth was fixed at that required to insure the formation of a hydraulic



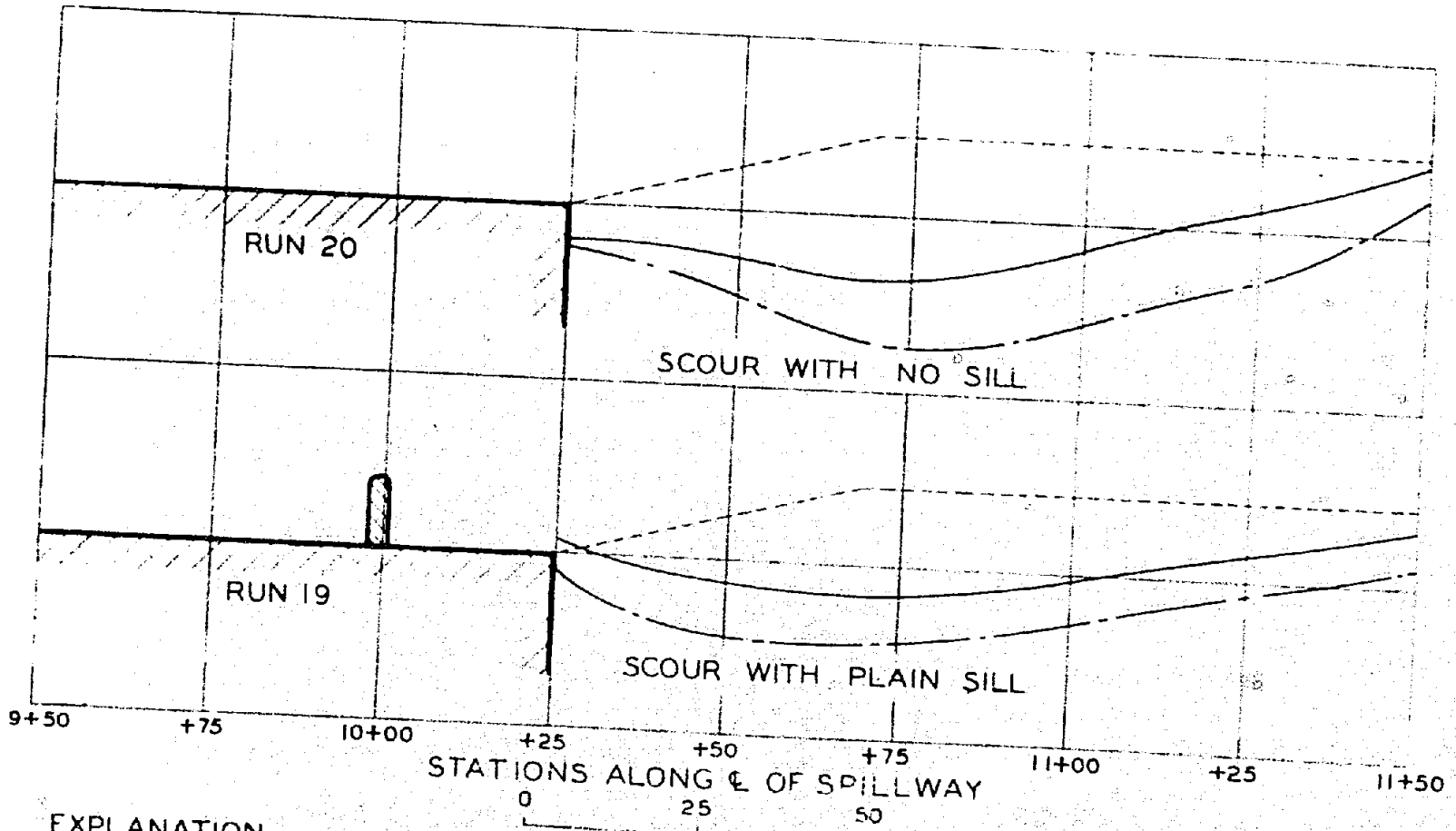
EXPLANATION

- SAND BEFORE RUN
- AV. DEPTH 30 MIN. RUN
- AV. DEPTH 60 MIN. RUN
- MAX. DEPTH 30 MIN. RUN
- MAX. DEPTH 60 MIN. RUN

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF PLAIN SILL
TROUGH SLOPE $1\frac{1}{2}:1$
60 FT. THROAT

FIGURE 39

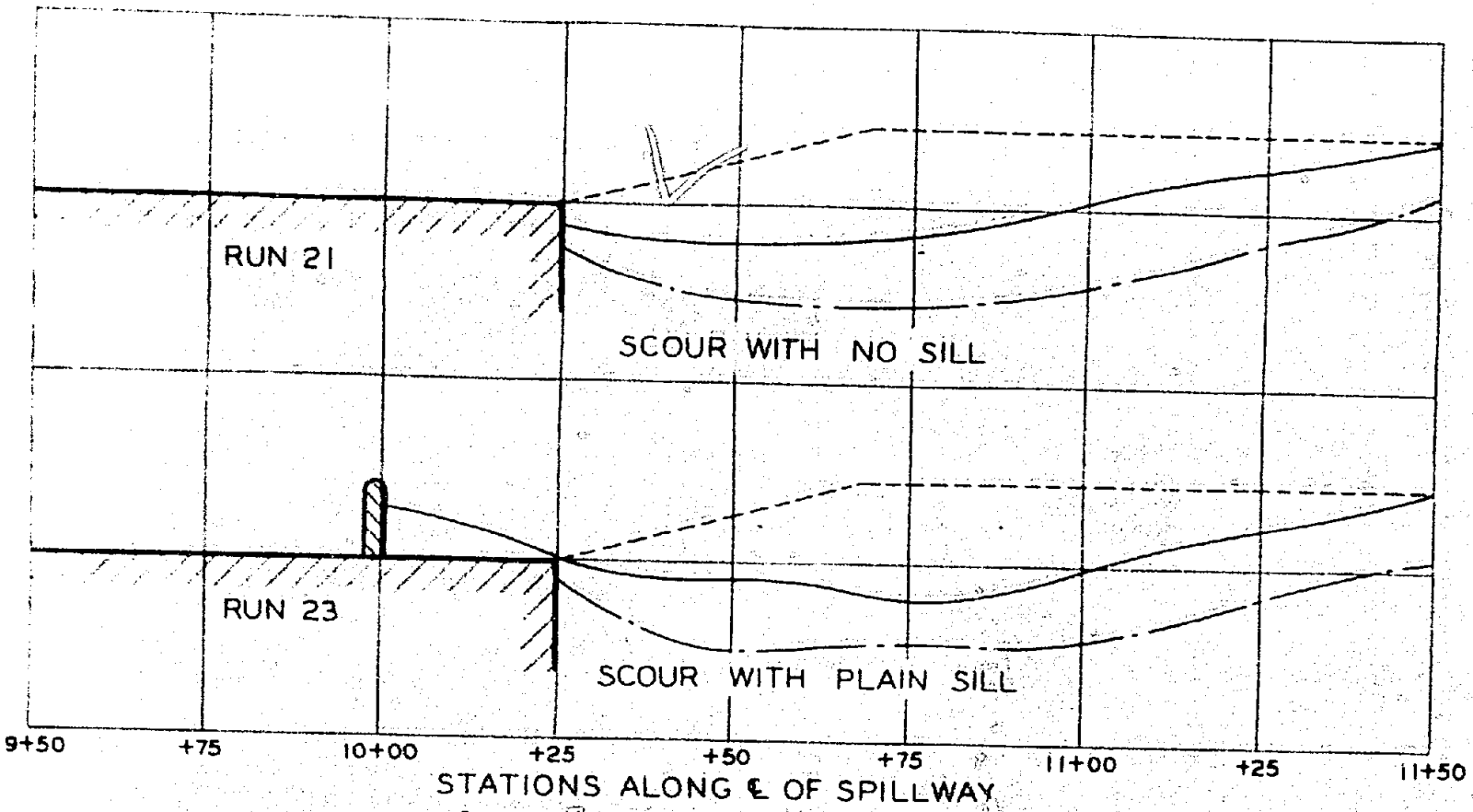


EXPLANATION

- SAND BEFORE RUN
- AV. DEPTH 60 MIN. RUN
- MAX. DEPTH 60 MIN. RUN

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF PLAIN SILL
TROUGH SLOPE 2:1

FIGURE 40



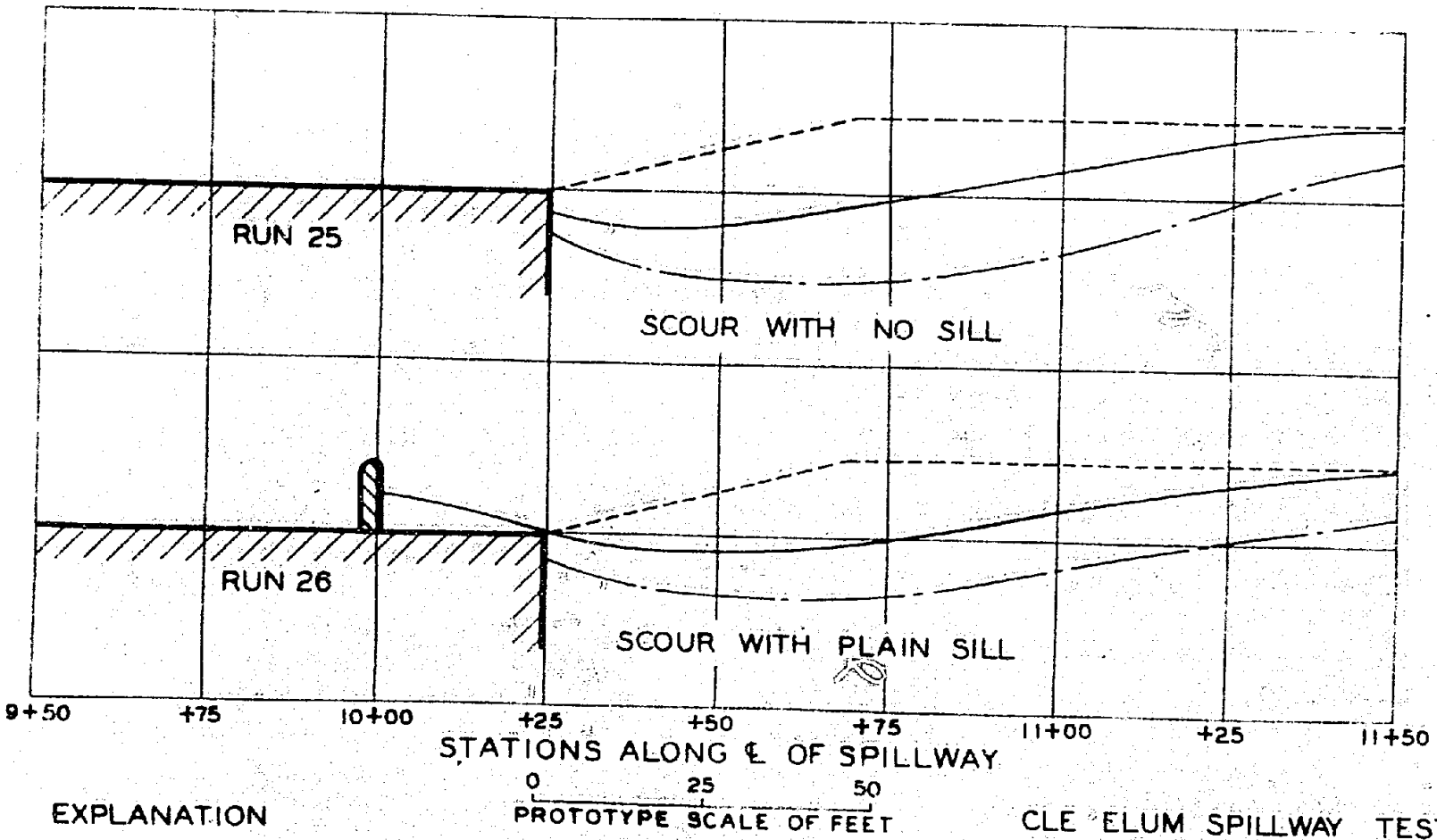
0 25 50
PROTOTYPE SCALE OF FEET

EXPLANATION

- - - - - - SAND BEFORE RUN
——— AV. DEPTH 60 MIN. RUN
—·— MAX. DEPTH 60 MIN. RUN

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF PLAIN SILL ON SCOUR
TROUGH SLOPE 3:1
PLAIN SILL 25FT. FROM END OF FLOOR

FIGURE 41

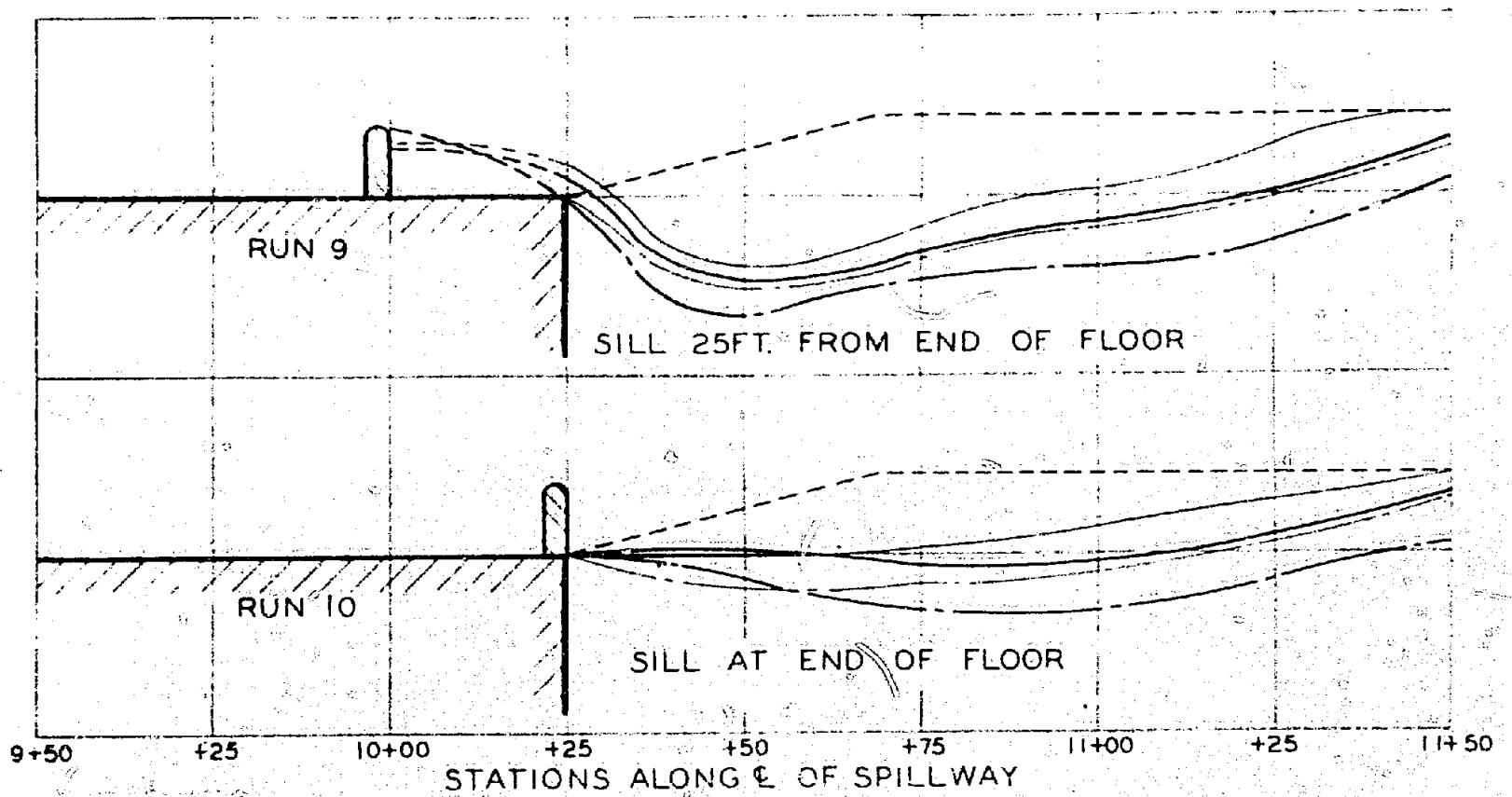


EXPLANATION

- - - - - - - - SAND BEFORE RUN
———— AV. DEPTH 60 MIN. RUN
—·—·— MAX. DEPTH 60 MIN. RUN

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF PLAIN SILL
TROUGH SLOPE 4:1

FIGURE 42



EXPLANATION

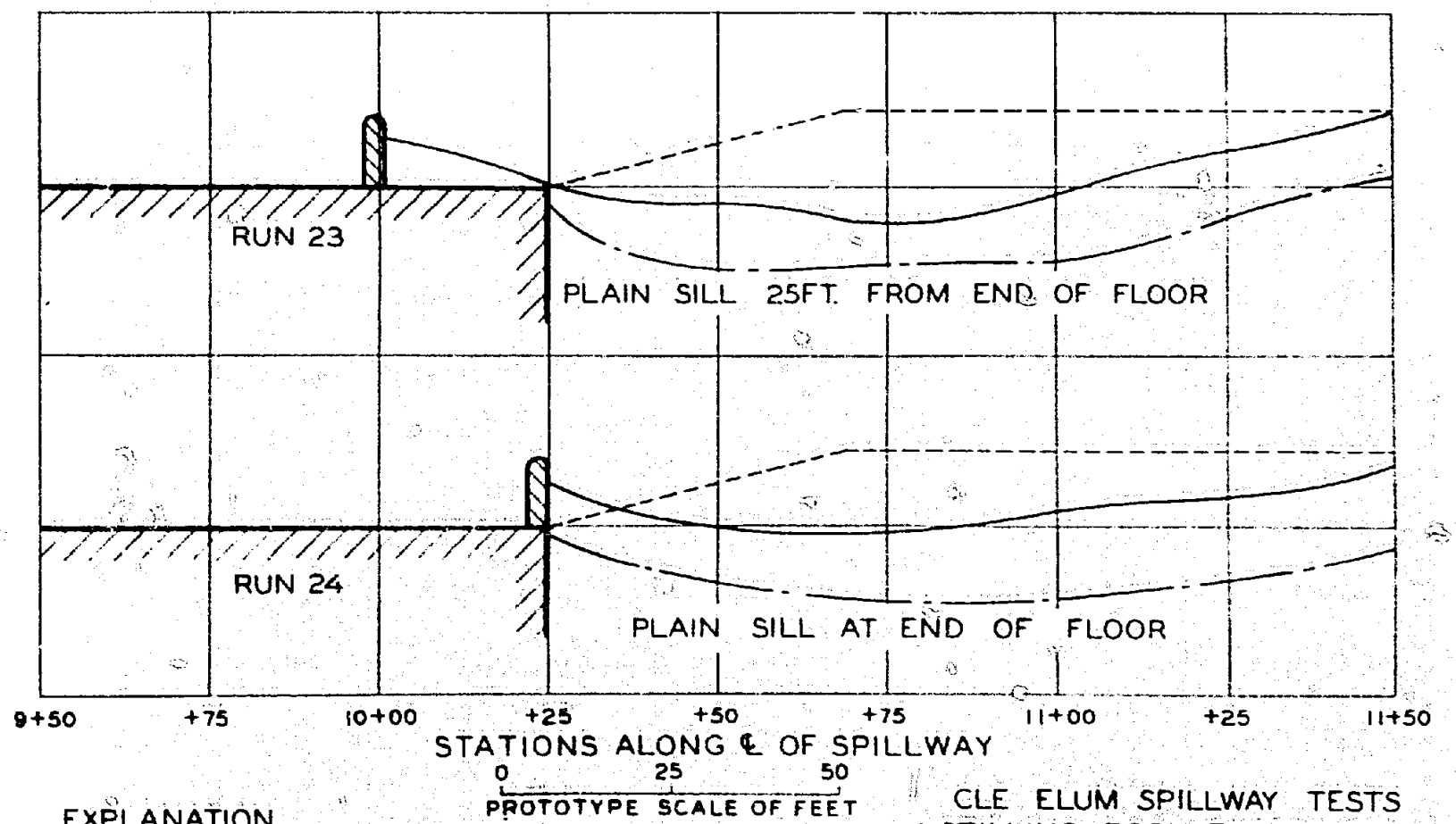
- SAND BEFORE RUN
- AV. DEPTH 30 MIN. RUN
- AV. DEPTH 60 MIN. RUN
- MAX. DEPTH 30 MIN. RUN
- MAX. DEPTH 60 MIN. RUN

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS

EFFECT OF LOCATION OF PLAIN SILL

FIGURE 43

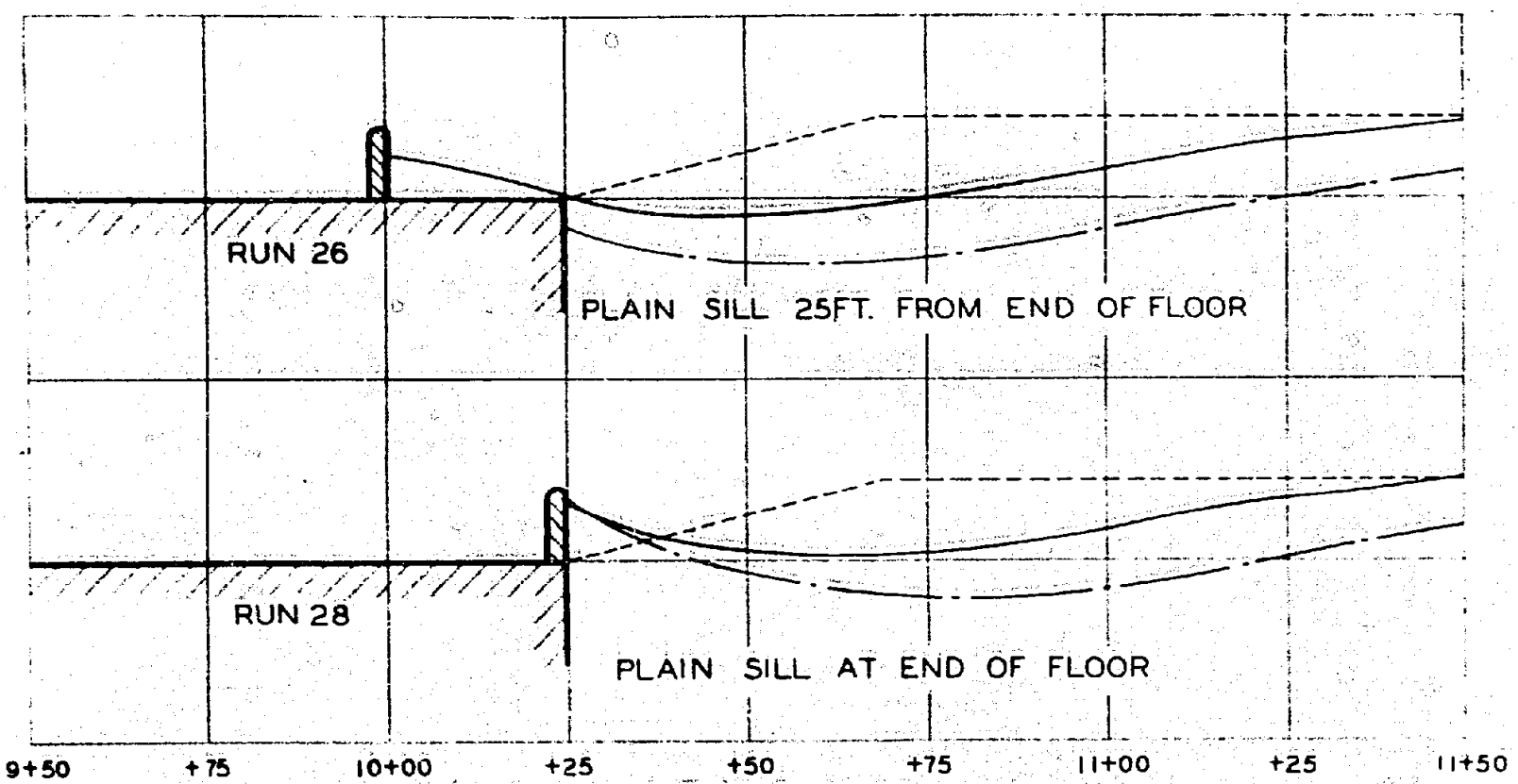


EXPLANATION

- - - - - - - SAND BEFORE RUN
——— AV. DEPTH 60 MIN. RUN
—·— MAX. DEPTH 60 MIN. RUN

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF LOCATION OF PLAIN SILL
TROUGH SLOPE 3:1

FIGURE 44

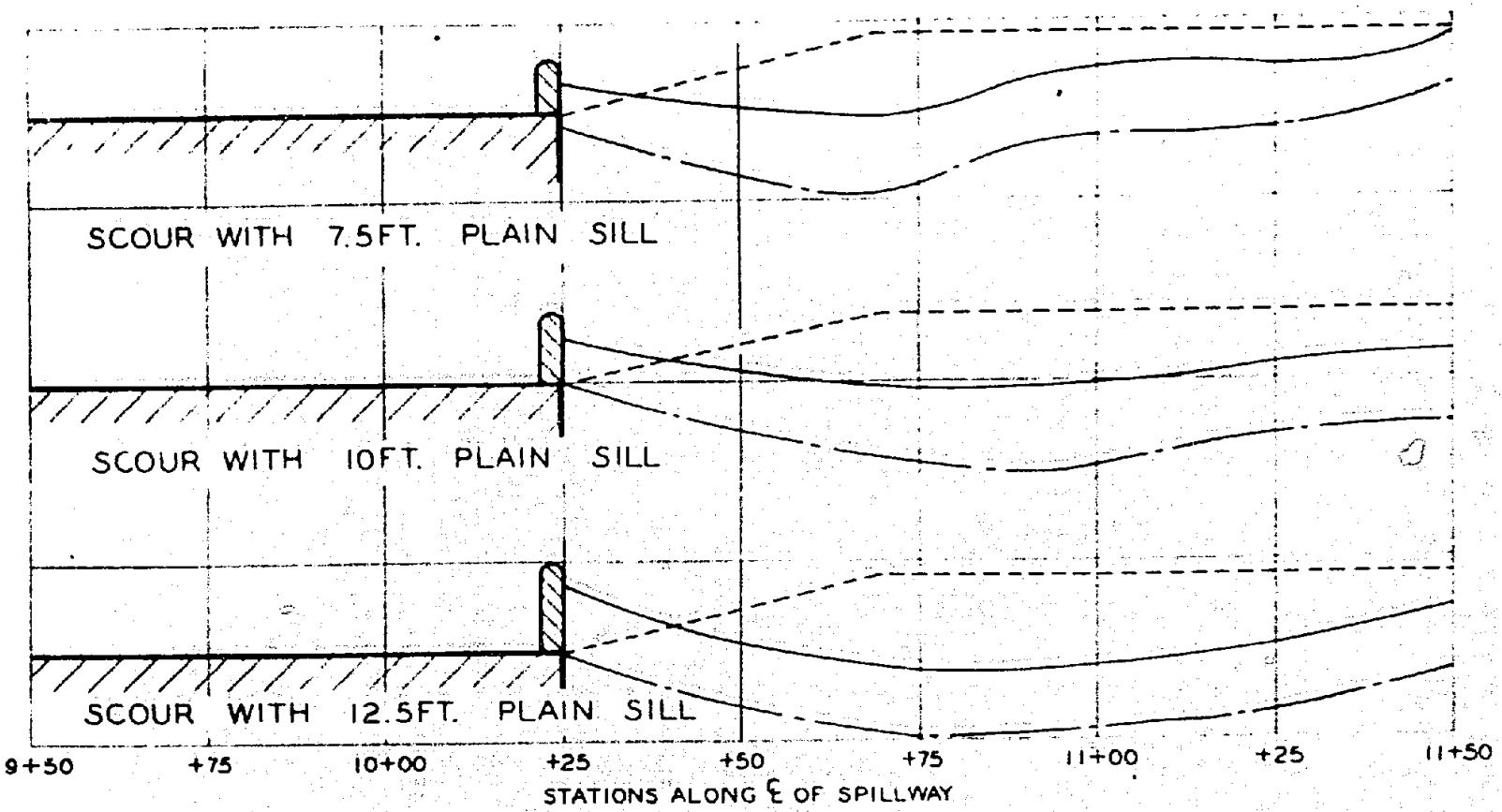


EXPLANATION

- - - - - - - SAND BEFORE RUN
——— AV. DEPTH 60 MIN. RUN
—— · —— MAX. DEPTH 60 MIN. RUN

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF LOCATION OF PLAIN SILL
TROUGH SLOPE 4:1

FIGURE 45



CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF SIZE OF PLAIN SILL
TROUGH SLOPE $1\frac{1}{2}$ 1

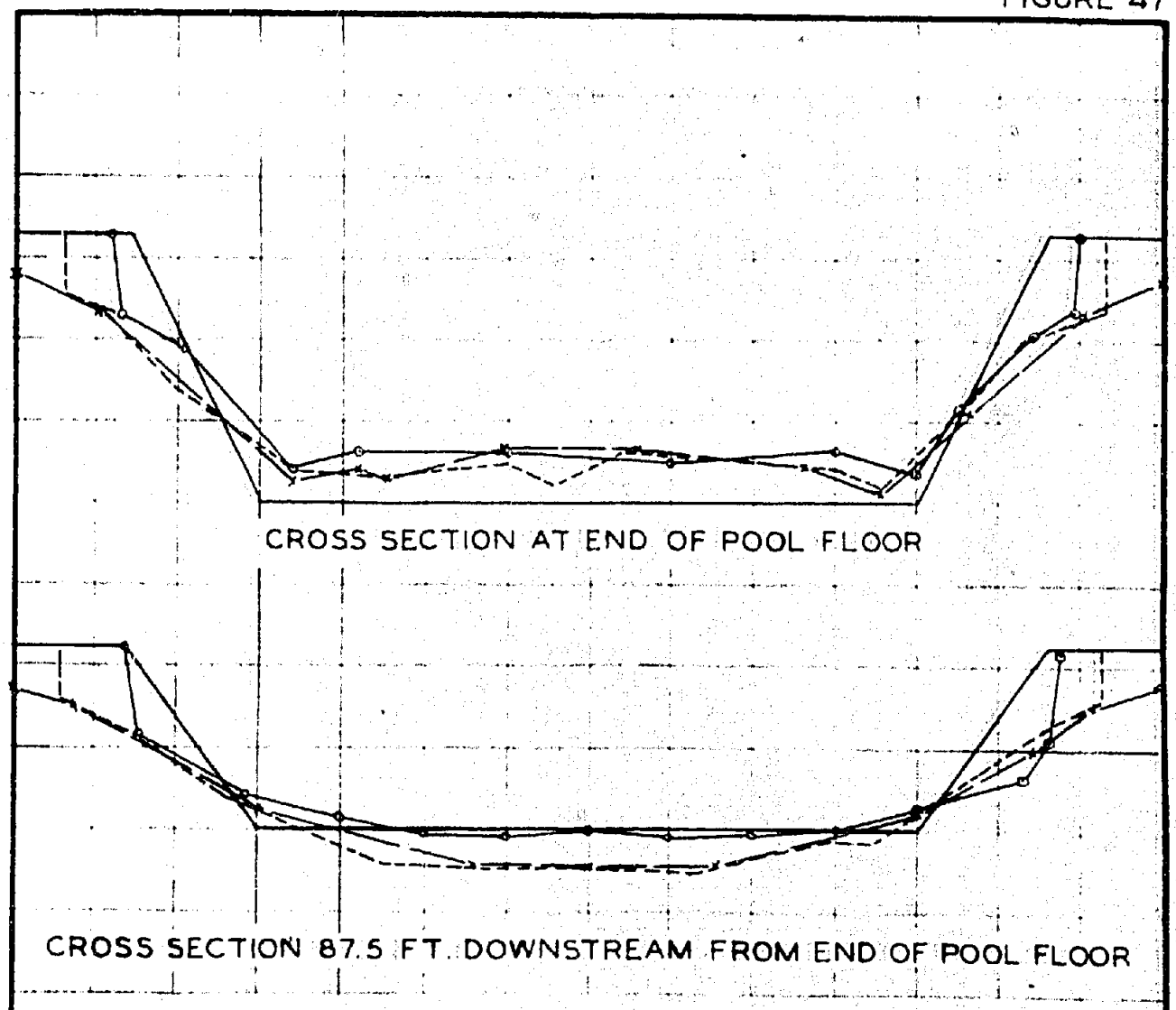
EXPLANATION

- SAND BEFORE RUN
- AV. DEPTH 60 MIN RUN
- MAX. DEPTH 60 MIN. RUN

0 25 50
PROTOTYPE SCALE OF FEET

FIGURE 46

FIGURE 47



SPILLWAY BOTTOM

BEFORE RUN

AFTER 20,000 SEC. FT. RUN

AFTER 30,000 SEC. FT. RUN

AFTER 40,000 SEC. FT. RUN

0 25 50

VERTICAL SCALE OF FEET

0 25 50 75 100

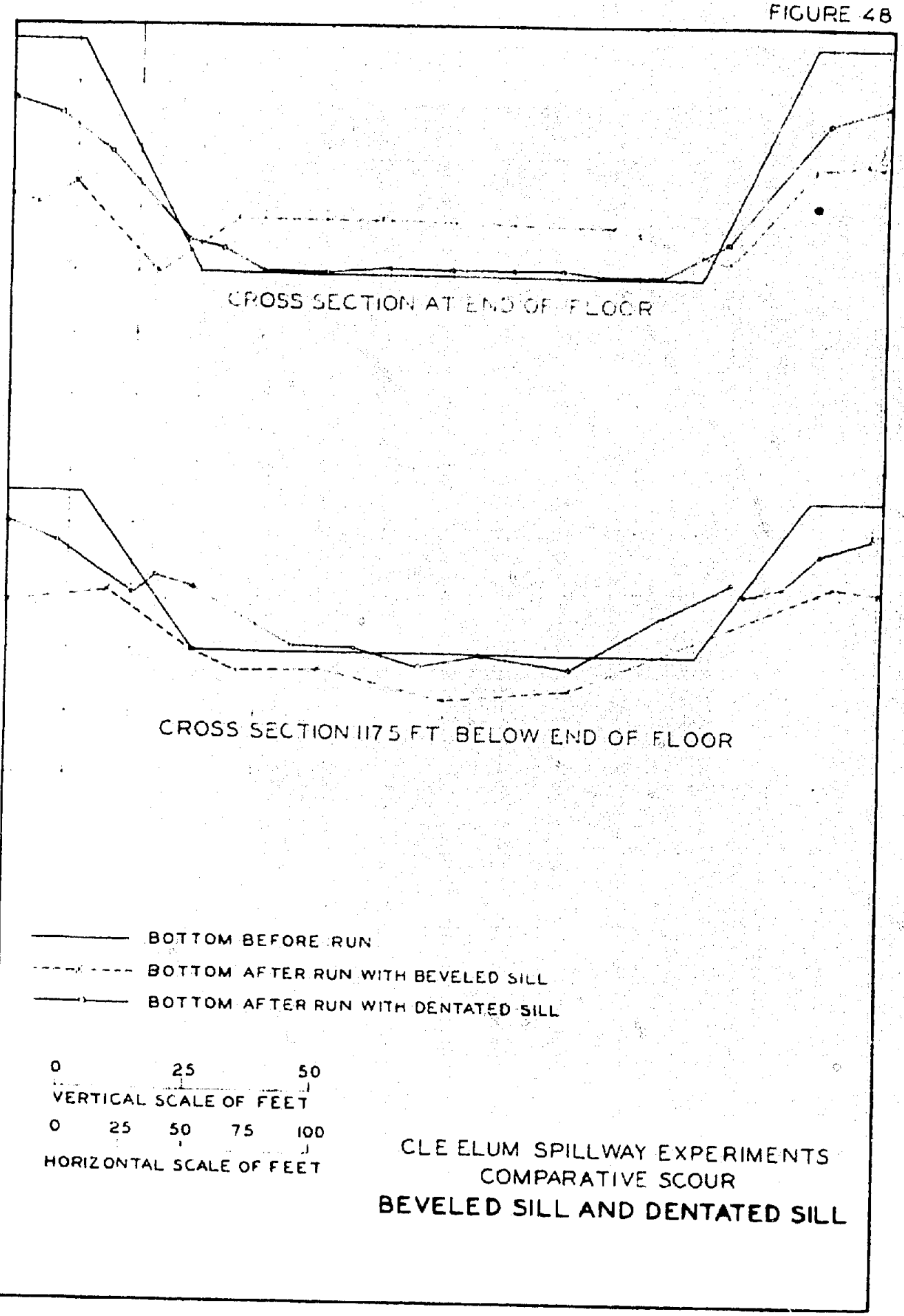
HORIZONTAL SCALE OF FEET

CLE ELUM SPILLWAY EXPERIMENTS

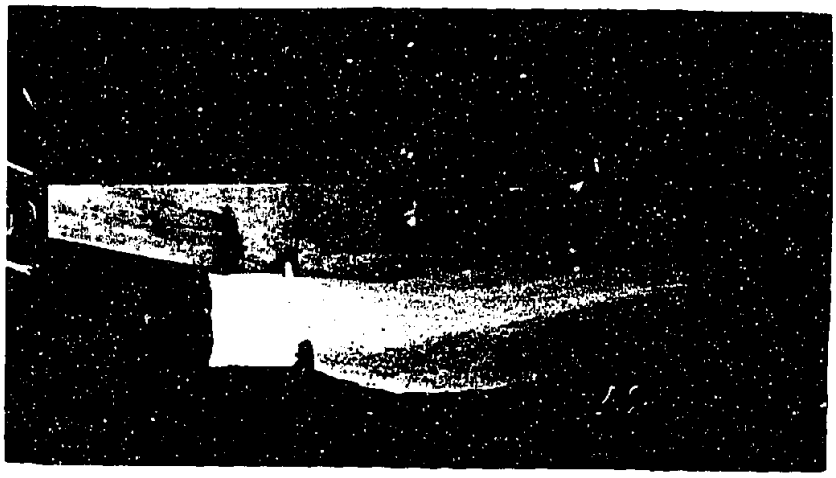
EFFECT OF LESS THAN DESIGN FLOW

PLAN SILL AT END OF FLOOR

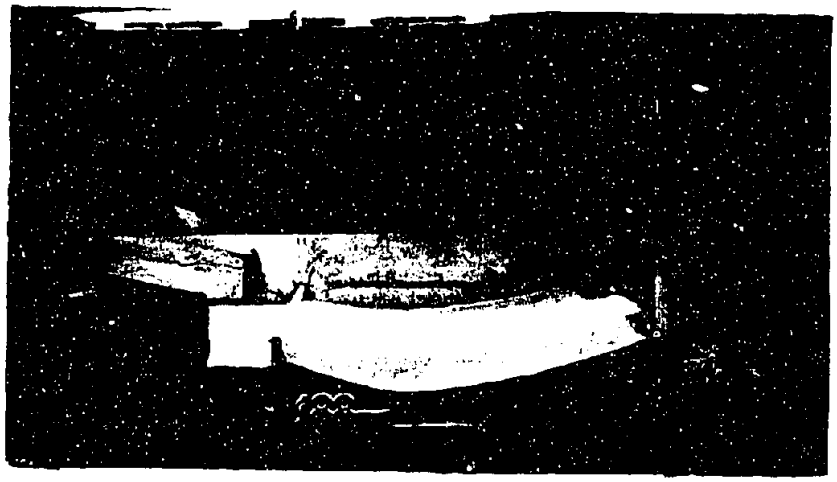
FIGURE 48



CLE ELUM SPILLWAY EXPERIMENTS
COMPARATIVE SCOUR
BEVELED SILL AND DENTATED SILL

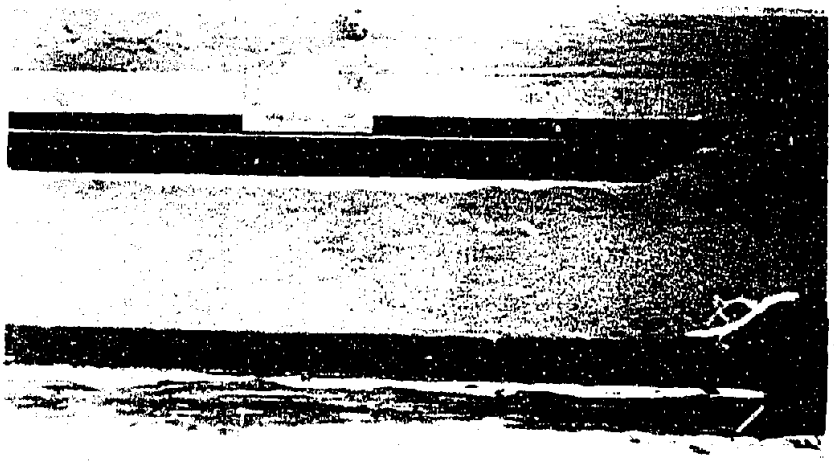


A - SILL 7.5 FT HIGH

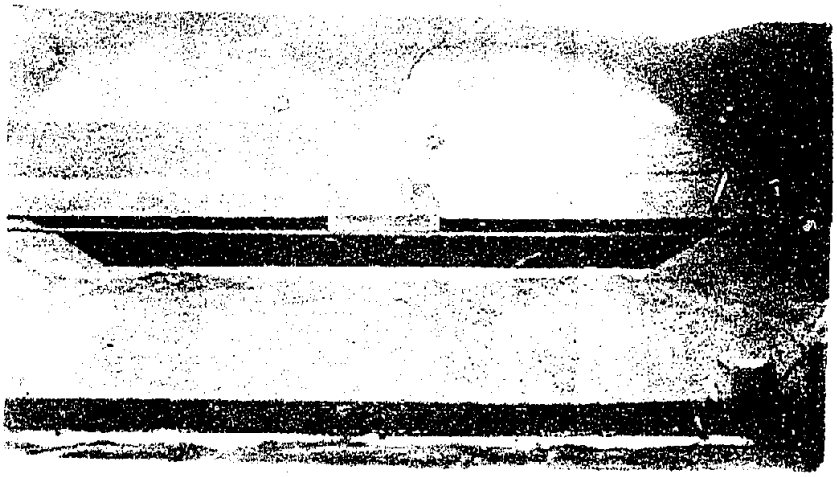


B - SILL 12.5 FT HIGH

SCOUR WITH VARIOUS SIZES OF PLAIN SILL

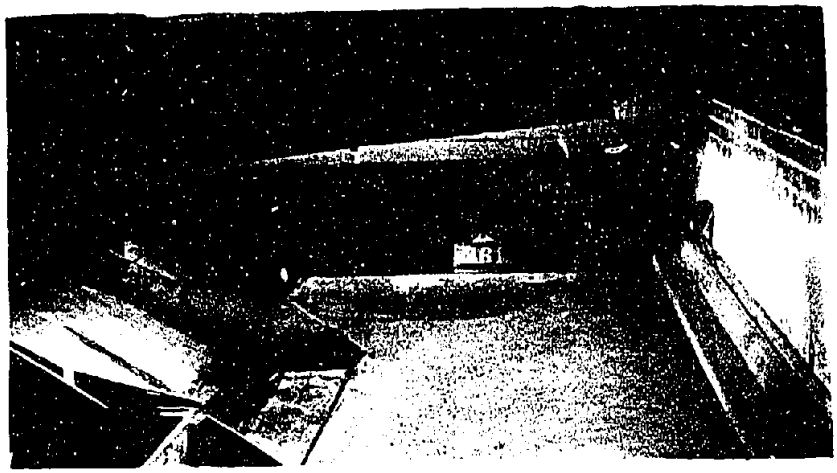


A-ENDS OF OGEE SHAPE

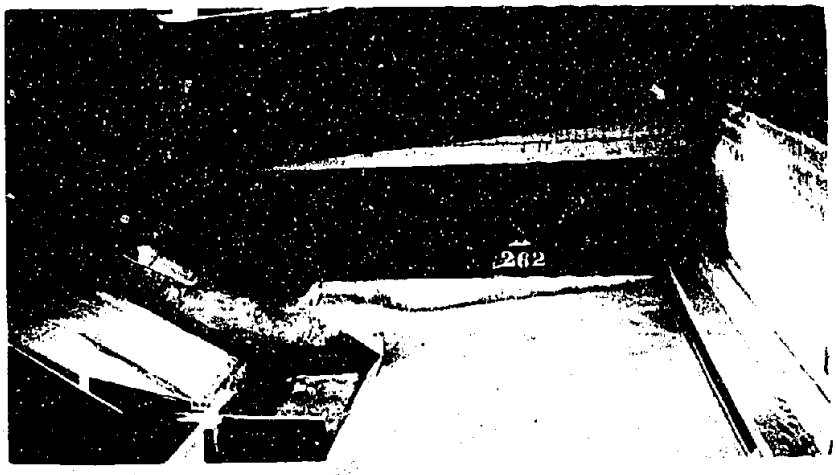


B-ENDS OF SQUARE SHAPE

PLAIN SILL WITH RAISED ENDS



A - TYPE II END WALL



B - TYPE V END WALL

SCOUR WITH BEVELED SILL

jump in the pool for all flows including the maximum discharge. It was recognized however that a wide range of widths and depths were possible, and that model tests should be used to determine the best combination of these dimensions, as well as the length of pool necessary to prevent excessive scour downstream.

To work out the best solution of these questions a large number of tests were made with various widths, depths and lengths of pool. In all cases a 10 ft. dentated sill with ends of type C (Figure 36) was used, with sloping end walls (Type II, Figure 58). Tests were made with pool widths of 200, 150 and 120 ft., each width with two to four different depths and each depth with two to four different lengths, making a total of 38 conditions. The width of the flumes for the various pool widths is given on Table I.

An optical comparison of the results of these runs is given on Plates XXXVII to XL inclusive. They show that for all conditions tested, even for those in which the pool was swept out, there was no serious scour on the channel bottom at the downstream edge of the floor. The scour on the channel sides and behind the pool walls was more variable than the scour on the bottom and was an important factor in the decision of the best combination of length, width and depth of pool. The consistency of the scour results with the various changes of length and depth was much better than might be expected, which gives considerable confidence in the comparative results and in this form of model test in general. The results of these runs are shown graphically on Figure 57. On this figure the elevation and length of floor is shown for the various runs. For each pool width a line is shown passing through the points representing lengths and depths which gave slight scour as shown on Plates XXXVIII to XL and other lines indicated those giving severe scour and very severe scour. As nearly as it was possible to determine, the conditions of scour along any line was the same. These lines indicate that the scour decreases as the floor length is increased. For the two runs on the highest floor level of the 150 ft. pool width the pool was swept out and severe scour occurred. With the exception of these runs the results cause the lines of equal scour to have a slope nearly parallel to the floor slope entering the pool, which makes the floor lengths practically the same for a given severity of scour. There is a tendency for the severe scour lines to slope more than the entrance slope, indicating that for equal floor lengths the severity of scour increases as the depth is increased. The reduction of scour with increasing length of pool was anticipated but it was also expected that deepening the pool would have the same effect, since it would give a greater mass of water in the pool in which the energy of the high velocity stream could be dissipated.

The explanation of this result probably lies in the steepness of the entrance slope to the pool and the curve at the bottom of it. With a tailwater level near the elevation required to sustain a jump for the flow conditions existing, the jump begins near the bottom of the curve, and the water enters the jump in a nearly horizontal direction. Under these circumstances the jump is quite efficient in dissipating the kinetic energy of the water. With greater pool depths the beginning of the jump moves upstream to a point where the water enters the jump in an inclined direction and the swift moving water tends to dive under the other water without forming an efficient jump. It is believed that with flatter entrance slopes or a greater radius curve at the bottom of the entrance slope, the advantage of a deeper pool would be greater.

A rigid determination of the best combination of pool width, length and depth would involve the estimation of cost of a series of pool designs for the various conditions, as well as a consideration of the hydraulic aspects of the problem. It is believed however that quite accurate estimate of the situation can be made by means of a concise statement of the factors involved in the comparative costs without quantitatively evaluating them.

For a given width and depth of pool, there is a minimum safe length, which can only be shown by model tests. The results shown on Plates XXXVIII to XL indicate that to lengthen the floor beyond the line on Figure 57 labeled "Slight Scour" would decrease the scour but little. Moreover, the scour at this point is such as could easily be taken care of by riprap or paving on the banks, or by raising the wing walls. It is believed therefore that this line marked "Slight Scour" represents the length of pool which should be built for the various widths and depths.

To narrow the stilling pool and compensate for the increased discharge per foot of width entering the pool by increasing the depth has the following effects for the conditions existing at Cle Elum.

(1) It considerably reduces the width of the floor in the lower part of the flume section of the spillway. This results in a considerably saving of cost of concrete, which is to a certain extent offset by the increased height required in the side walls. It is probable however that the net result is a decrease in cost.

(2) It reduces the width of the pool floor but increases the height of the jump, which necessitates an increase in the thickness of the floor to resist the greater unbalanced pressure of the jump. It is probable that the decrease in width is considerably more than offset by the increase in thickness and the net result is increased in cost. If the apron and pressure on the floor is relieved

by underdrainage, the balance in favor of a wider pool is decreased and possibly eliminated, but it is not believed that it would make the balance appreciably in favor of the narrower pool.

(3) It probably reduces the volume of excavation but increases the unit price, due to the greater cost of pumping as the depth below ground water increases. The net result is probably an increase in cost.

(4) To obtain the same severity of scour with a deeper pool is shown by experiment to require an equal or greater length of pool floor and greater length of side walls which would result in an increase in cost.

(5) It increases the height and strength required in the walls. The increased strength is due not only to the greater earth pressure resulting from the higher walls but also the greater unbalanced water pressure resulting from the lower level in the upper end of the stilling pool with the higher jump, the tailwater level and the water pressure on the back of the wall being the same. Since the height of the walls at Cle Elum is such that the bearing pressure limit is about reached, an increase of height and unbalanced water pressure may therefore necessitate an increase in the normal base width of the wall to secure a low enough bearing pressure. In any event these effects would cause an increase in cost. Venting the side walls will reduce the balance in favor of the wider pool but not eliminate it.

It is evident therefore that the only saving in narrowing and deepening the pool is a decrease in the cost of the flume paving. Hence it is desirable to have the pool relatively wide and shallow. There are factors however which limit the width to which the pool can economically be built. They are:

(1) The increase in cost of the flume paving previously discussed.

(2) An increase in the excavation which might exceed the quantity required in the dam.

(3) The extent to which the stream flowing through the spillway can spread before entering the pool.

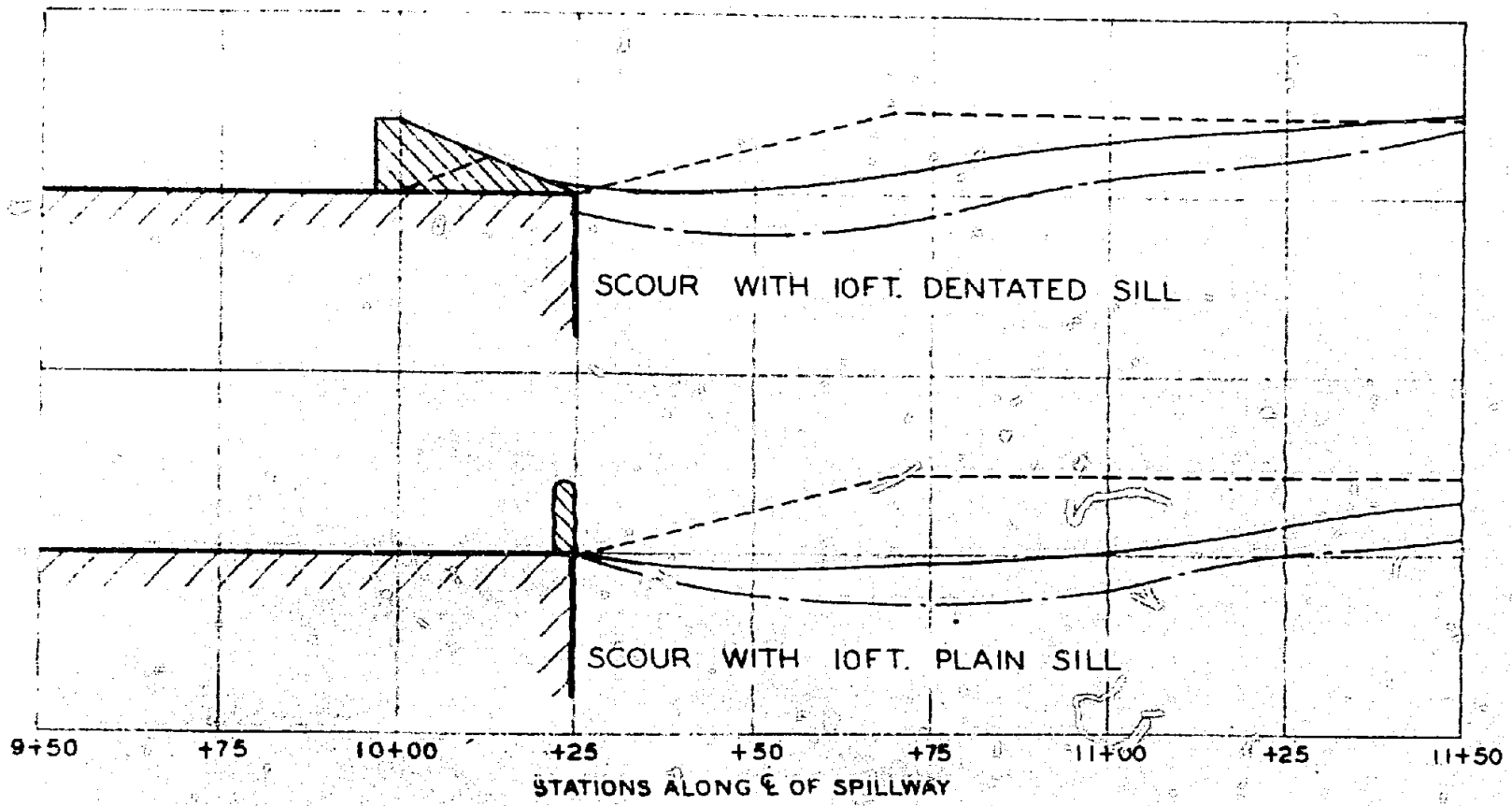
(4) The undesirability of increasing indefinitely the area of the spillway floor and thus the risk of a failure of it at some point.

(5) The fact that there is a certain minimum depth of pool floor below which it is undesirable to go.

Without evaulating quantitatively those of these factors subject to such analysis, it seems probable that the limit of width from the economic and hydraulic standpoints is about reached at the 200 ft. width, and it was therefore considered to be the best width for the final design. The scour tests show that as long as the pool is not swept out, the scour for any pool width and reasonable depth is roughly the same for the same horizontal length of pool floor. Placing the floor at a lower level than is necessary to form the jump will decrease somewhat the thickness of floor necessary, on account of a lesser reduction of downward pressure caused by the jump, but it would increase both the volume and unit price of the excavation and also increase the length and height of the pool walls. The greatest economy is therefore secured by placing the floor as high as safety permits. The experiments show that for the 200 ft. pool width, the bottom can be placed at least 5 ft. higher than the computations based on the theory of the jump indicates, without sweeping out. This is probably due to the obstructing effect of the sill. It is believed that the model test results would be duplicated in the prototype. If they were not duplicated however and sweeping out occurred, destruction of the pool might follow from the lifting of the pool floor from upward pressure or from impact on the dentated sill, which could hardly be anchored firmly enough with an earth foundation. A failure of the pool would almost certainly result in the failure of the dam, and a great disaster. To secure the factor of safety which should exist under these circumstances, it is believed that depths should not be used which are less than those which the jump theory indicate to be necessary. This would require the pool floor of the 200 ft. width to be at El. 2093. As previously stated, it is believed that the length of floor required for safety is that shown by the "Slight Scour" line on Figure 57 which, for a pool width of 200 ft. and floor level 2093 gives a pool floor extending approximately to Sta. 10+00. Summarizing the foregoing reasoning; it was considered that the best pool width was 200 ft., the best pool floor level El. 2093 and best pool length to end at Sta. 10+00, giving a pool length of approximately 100 ft. including the dentated sill.

Best Type of Side Walls for the Pool

Extensive experimentation was carried out to determine the best form of the ends of the walls of the stilling pool. As these walls were about forty feet high, considerable saving would be accomplished if the length or height could be reduced. The types of wall tested are shown on Figure 58. Photographs of these are shown on Plates XLI and XLII. Type I was used in some preliminary tests. With the raised ends on the plain sill however the sill extended above the wall and it was necessary to raise the wall to form types III and IV. Types VI and VII were tested and gave good protection



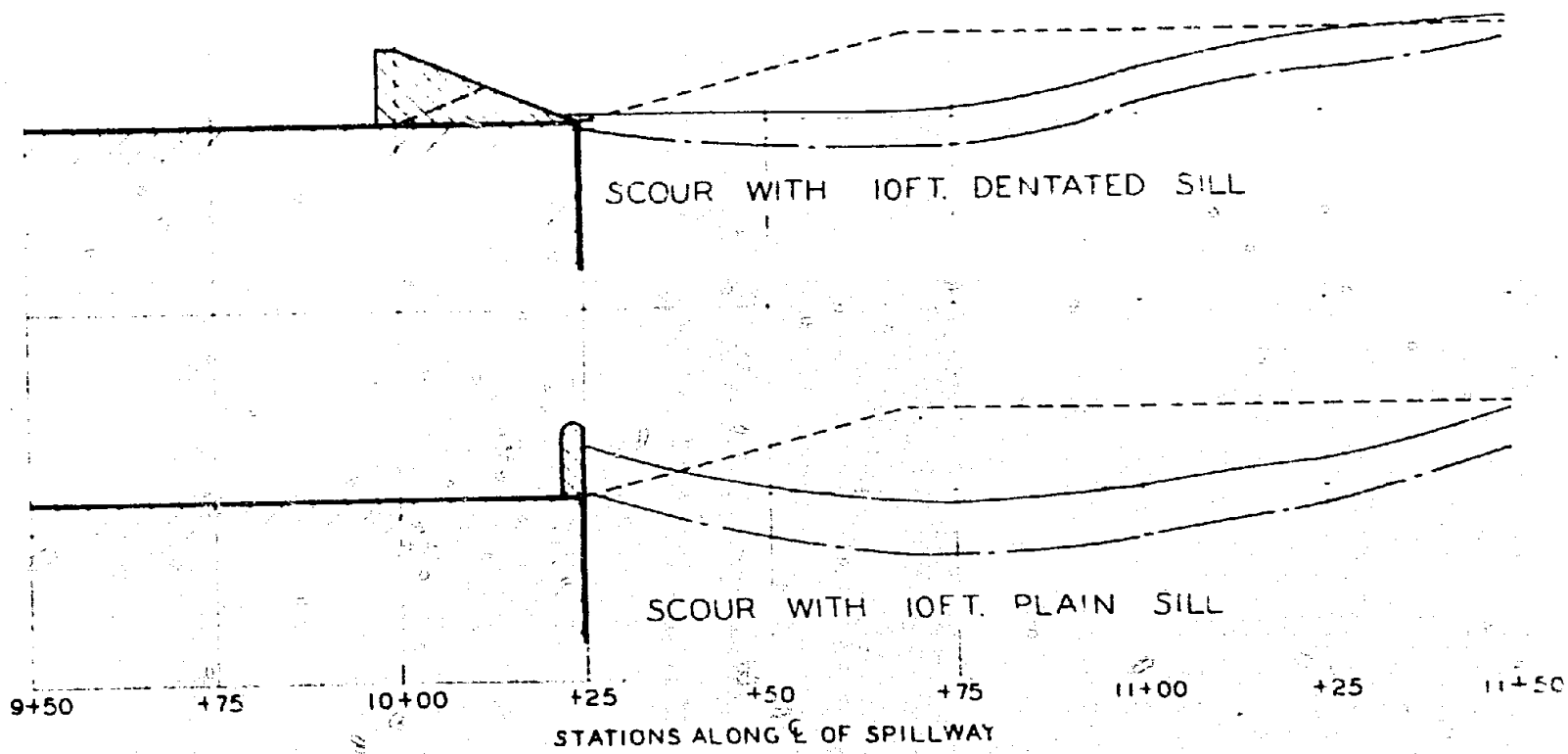
EXPLANATION

- - - - - - SAND BEFORE RUN
——— AV. DEPTH 60 MIN. RUN
—·—·— MAX. DEPTH 60 MIN. RUN

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
10FT. DENTATED VS. 10FT. PLAIN SILL
EXPANSION ENDS AT STA. 7+84
FLUME SLOPE 1½:1 THROAT 100FT.

FIGURE 49



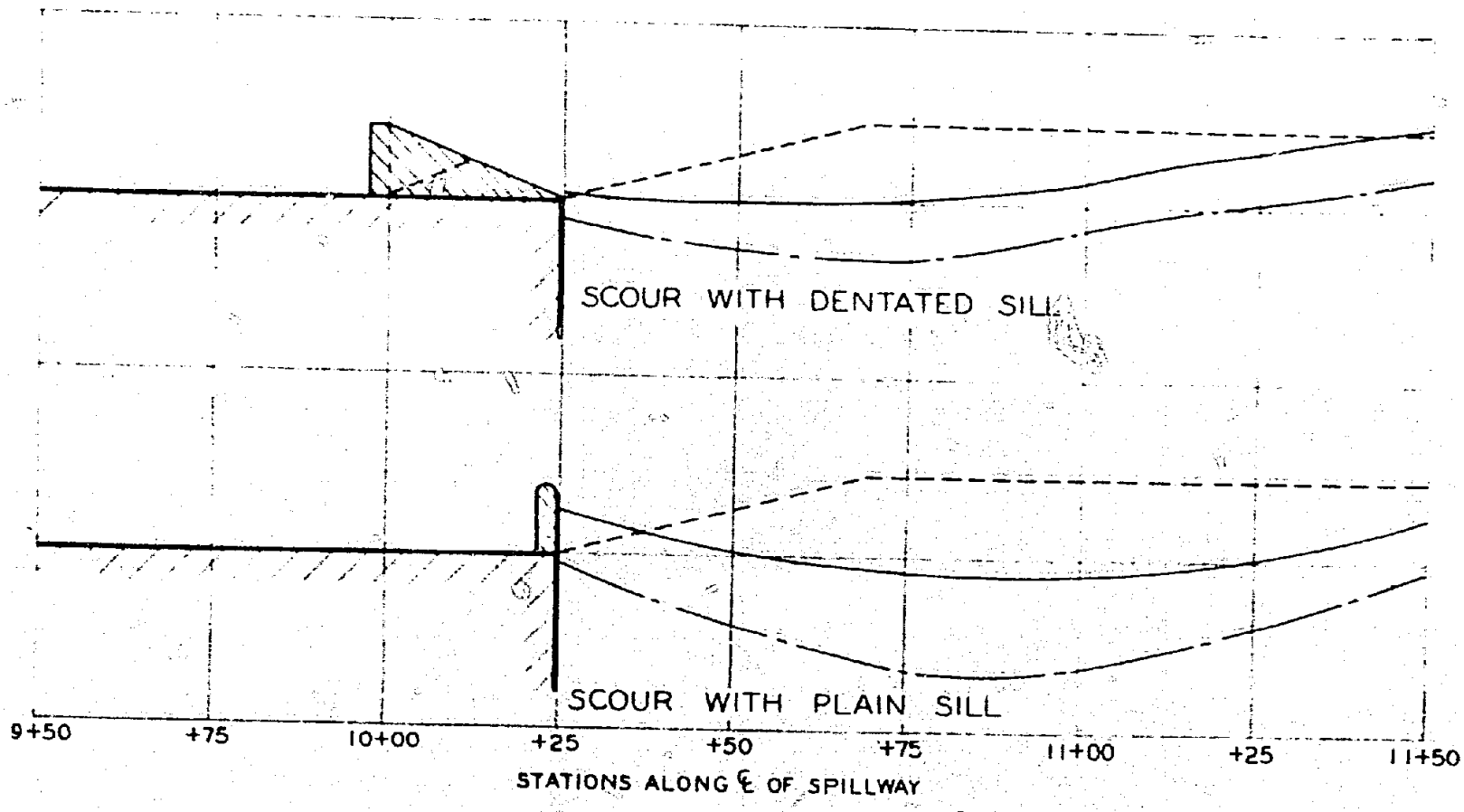
EXPLANATION

- - - - - - - SAND BEFORE RUN
——— AV. DEPTH 60 MIN. RUN
—— · —— MAX. DEPTH 60 MIN. RUN

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
10FT. DENTATED VS. 10FT. PLAIN SILL
EXPANSION ENDS AT STA. 9+00
FLUME SLOPE $1\frac{1}{2}$:1 THROAT 100FT.

FIGURE 50



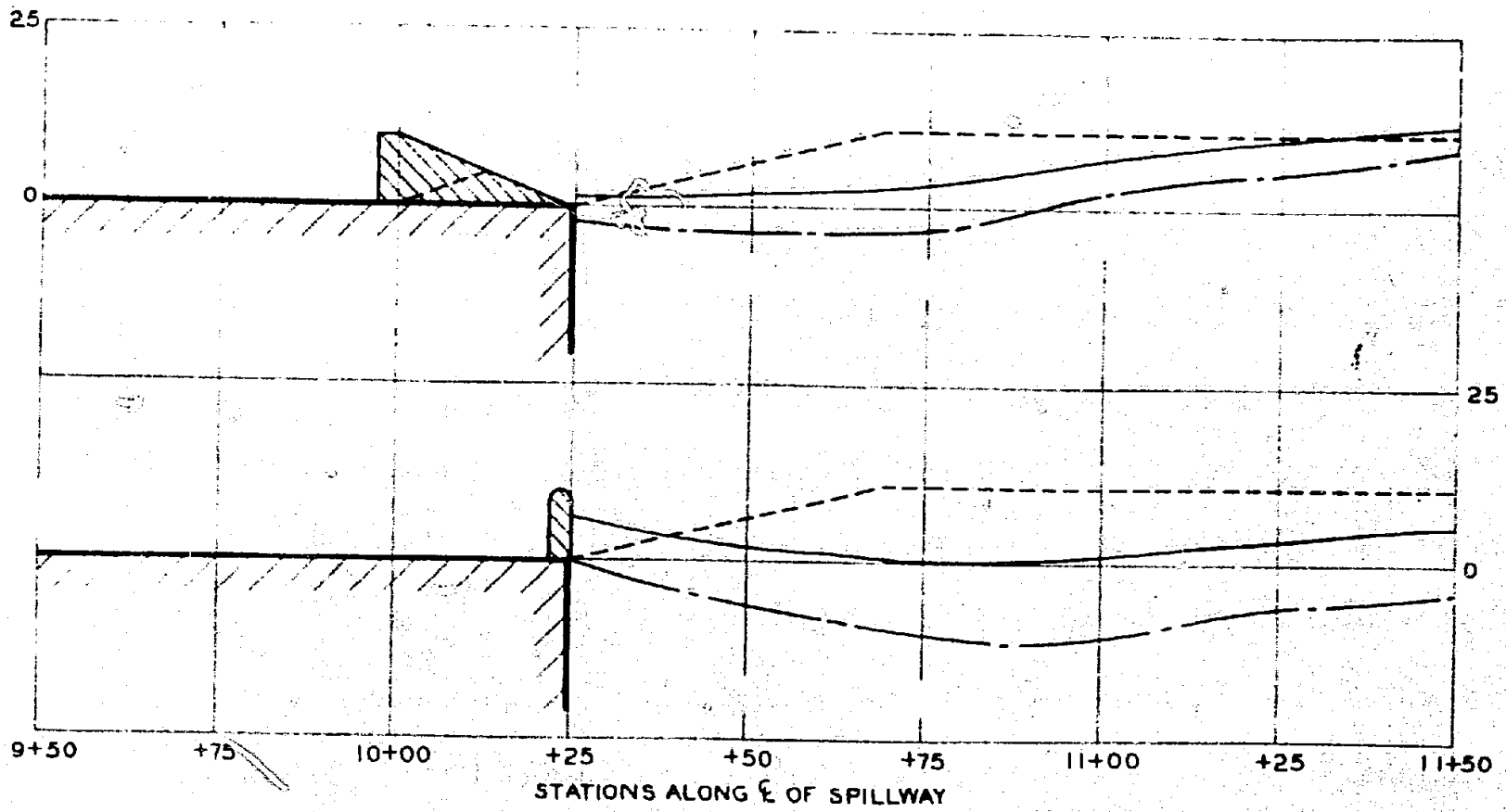
EXPLANATION

- - - - - - SAND BEFORE RUN
——— AV. DEPTH 60 MIN. RUN
—·— MAX. DEPTH 60 MIN. RUN

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
10FT. DENTATED VS. 10FT. PLAIN SILL
EXPANSION ENDS AT STA 9+96
FLUME SLOPE $1\frac{1}{2}$:1 SET UP "A"

FIGURE 51



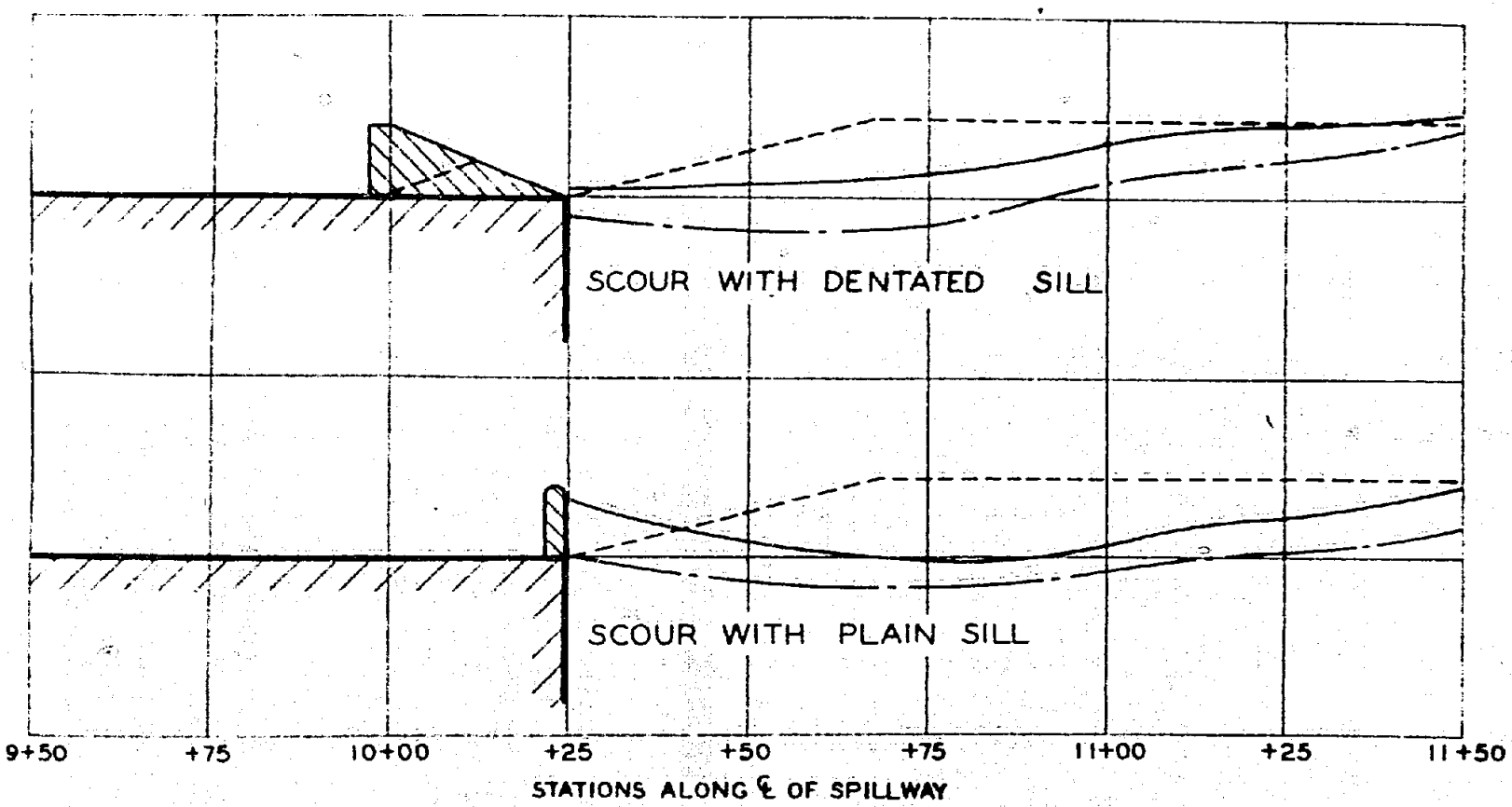
EXPLANATION

- - - - - - - SAND BEFORE RUN
——— AV. DEPTH 60 MIN. RUN
—·— MAX. DEPTH 60 MIN. RUN

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
DENTATED VS. PLAIN SILL
EXPANSION ENDS AT STA. 9+96
FLUME SLOPE 1½:1 SET UP "B"

FIGURE 52



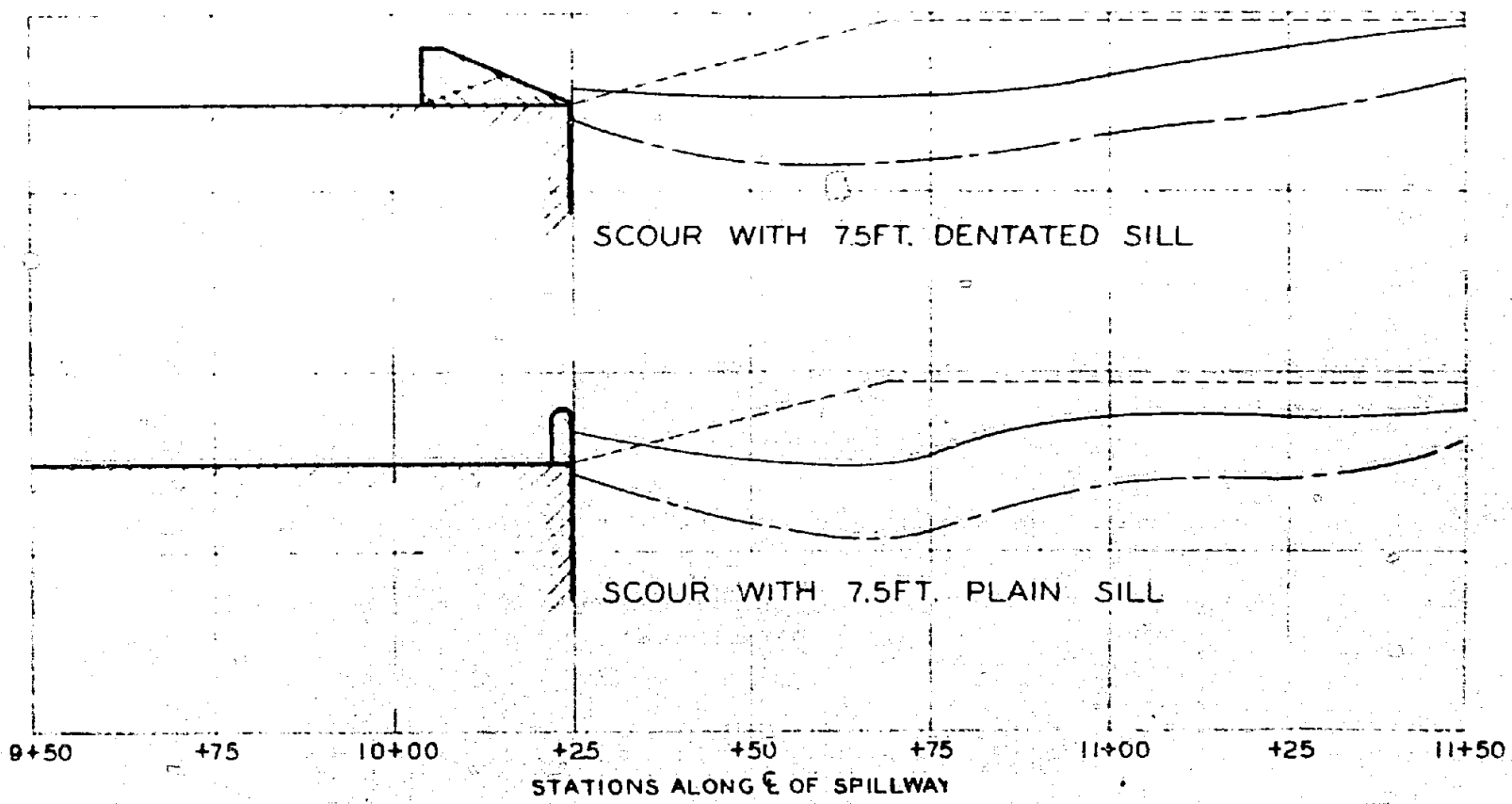
EXPLANATION

- - - - - - - - SAND BEFORE RUN
———— AV. DEPTH OF SCOUR
——— · ——— MAX. DEPTH OF SCOUR

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
10FT. DENTATED VS. 10FT. PLAIN SILL
CURVE AT BOTTOM OF SLOPE
FLUME SLOPE 1½:1 THROAT 100FT.

FIGURE 53



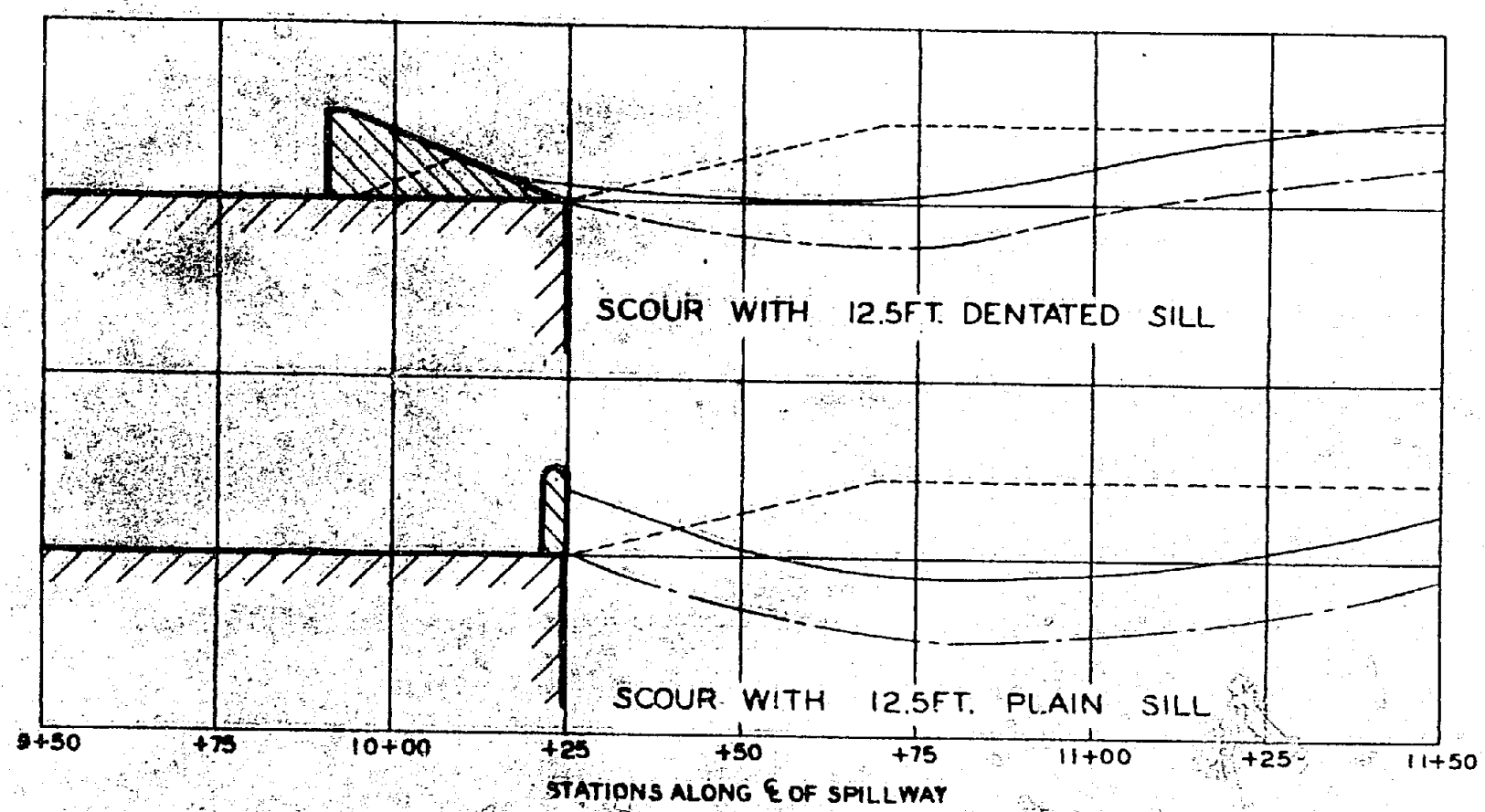
EXPLANATION

- - - - - - - SAND BEFORE RUN
——— AV. DEPTH 60 MIN. RUN
—·— MAX. DEPTH 60 MIN. RUN

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
7.5FT. DENTATED VS. 7.5FT. PLAIN SILL
EXPANSION ENDS AT STA. 9+96
FLUME SLOPE $1\frac{1}{2}$:1 SET UP "B"

FIGURE 54



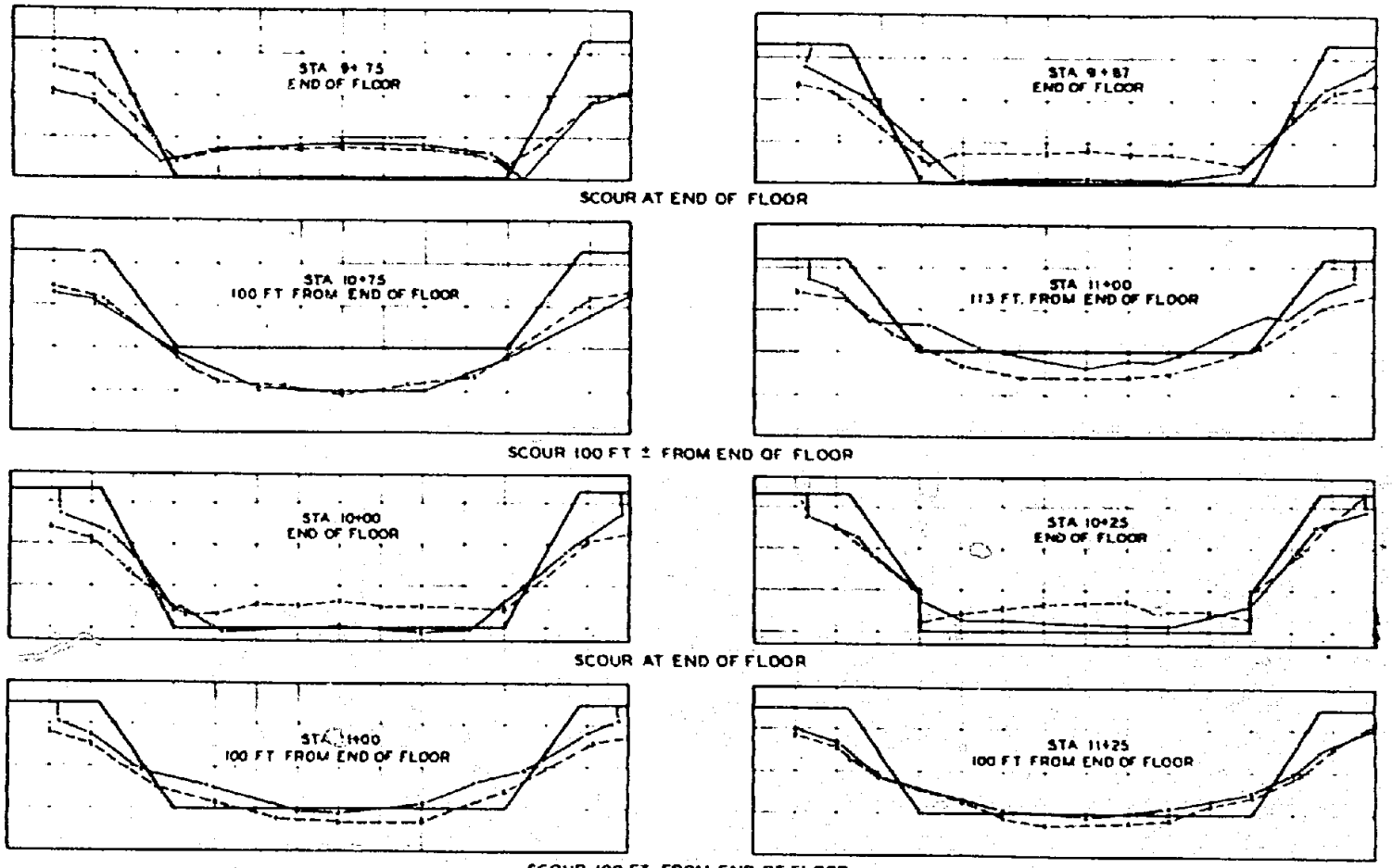
EXPLANATION

- - - - - - SAND BEFORE RUN
——— AV. DEPTH 60 MIN. RUN
—·—·— MAX. DEPTH 60 MIN. RUN

0 25 50
PROTOTYPE SCALE OF FEET

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
12.5FT. DENTATED VS. 12.5FT. PLAIN SILL
EXPANSION ENDS AT STA. 9+96
FLUME SLOPE $1\frac{1}{2}$:1 SET UP "B"

FIGURE 55



EXPLANATION

BOTTOM BEFORE RUN

BOTTOM AFTER RUN 10 FT PLAN SILL

BOTTOM AFTER RUN 10 FT DENTATED SILL

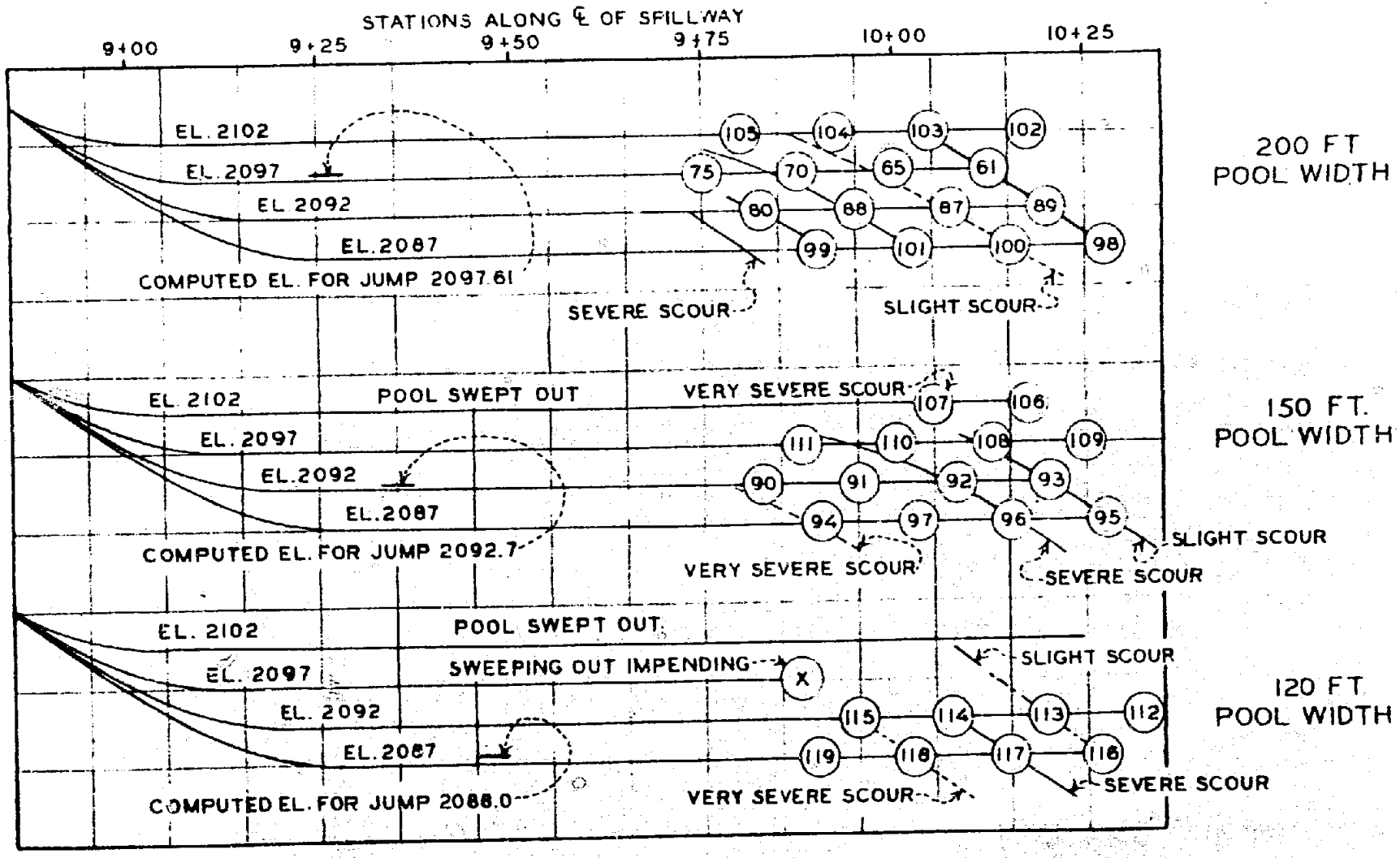
0 25 50
VERTICAL SCALE OF FEET

0 25 50 75 100
HORIZONTAL SCALE OF FEET

CLE ELUM SPILLWAY EXPERIMENTS
SCOUR, DENTATED SILL VS.
PLAN SILL AT END OF FLOOR

32-D-294

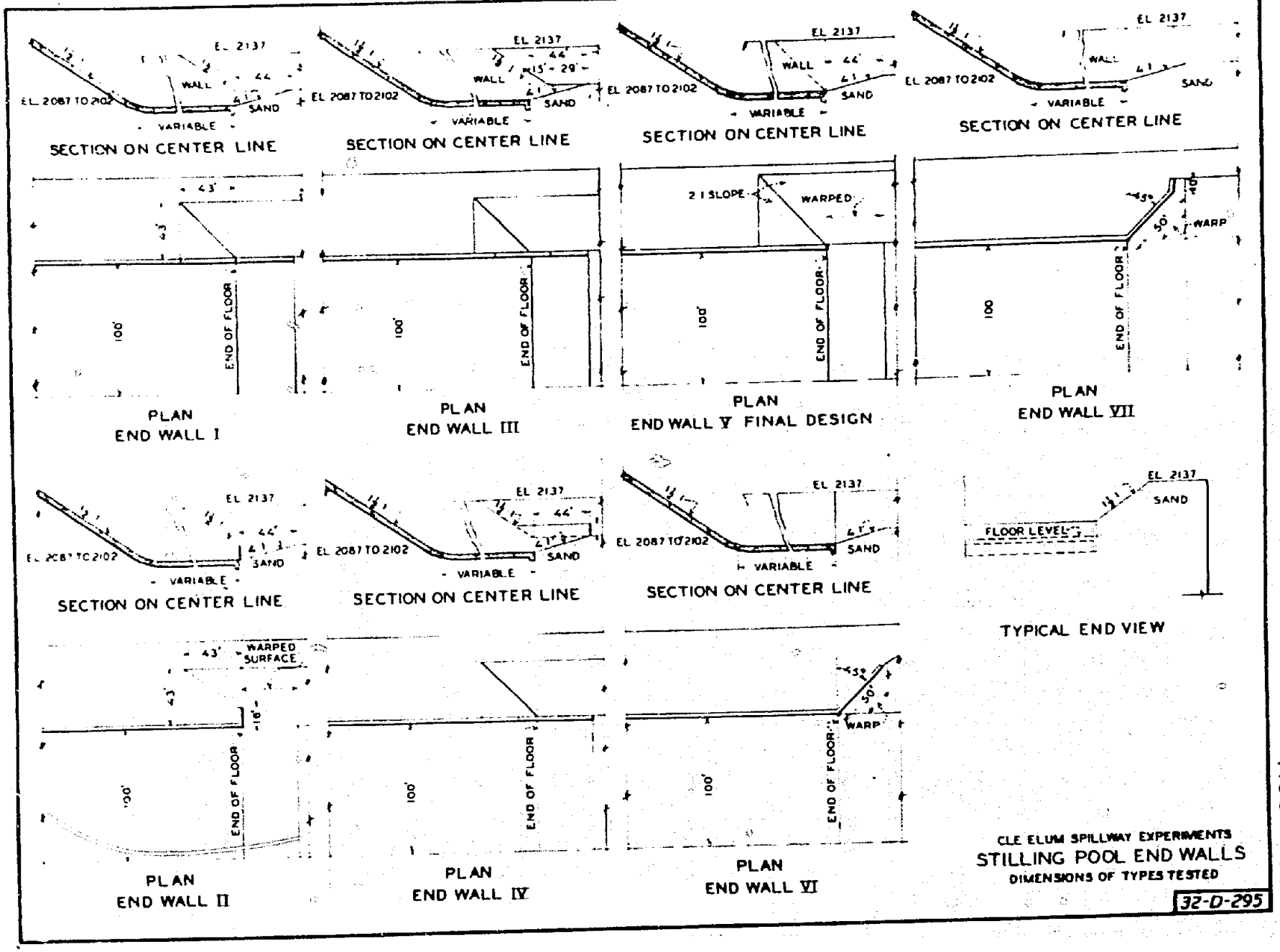
FIGURE 56



TAILWATER LEVEL EL. 2130
10 FT. DENTATED SILL
TYPE II END WALLS
40,000 SEC. FT. DISCHARGE

CLE ELUM SPILLWAY EXPERIMENTS
COMPARATIVE SCOUR WITH POOLS OF VARIOUS LENGTHS, DEPTHS AND WIDTHS

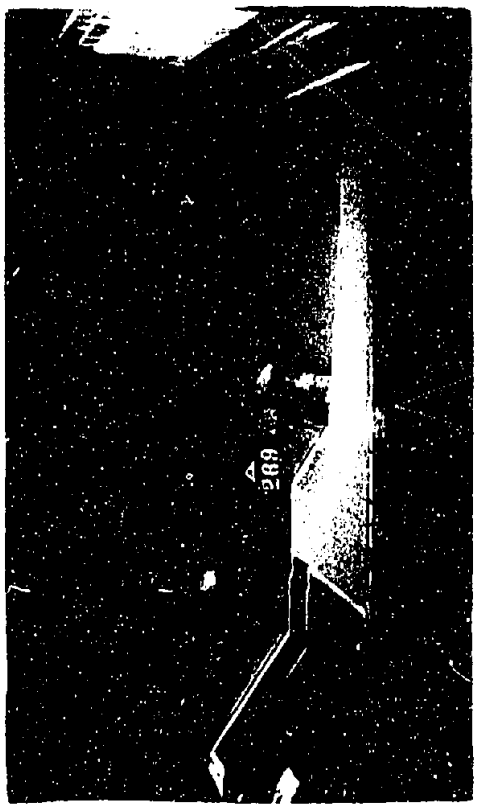
FIGURE 57

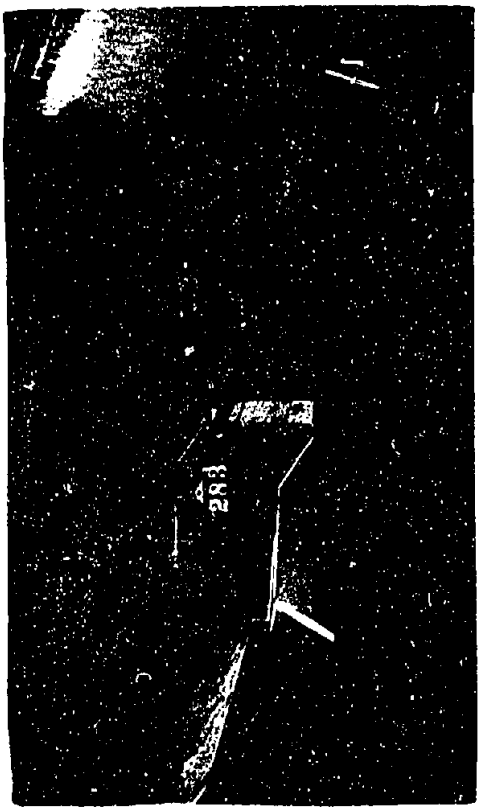


CLE ELUM SPILLWAY EXPERIMENTS
STILLING POOL END WALLS
DIMENSIONS OF TYPES TESTED

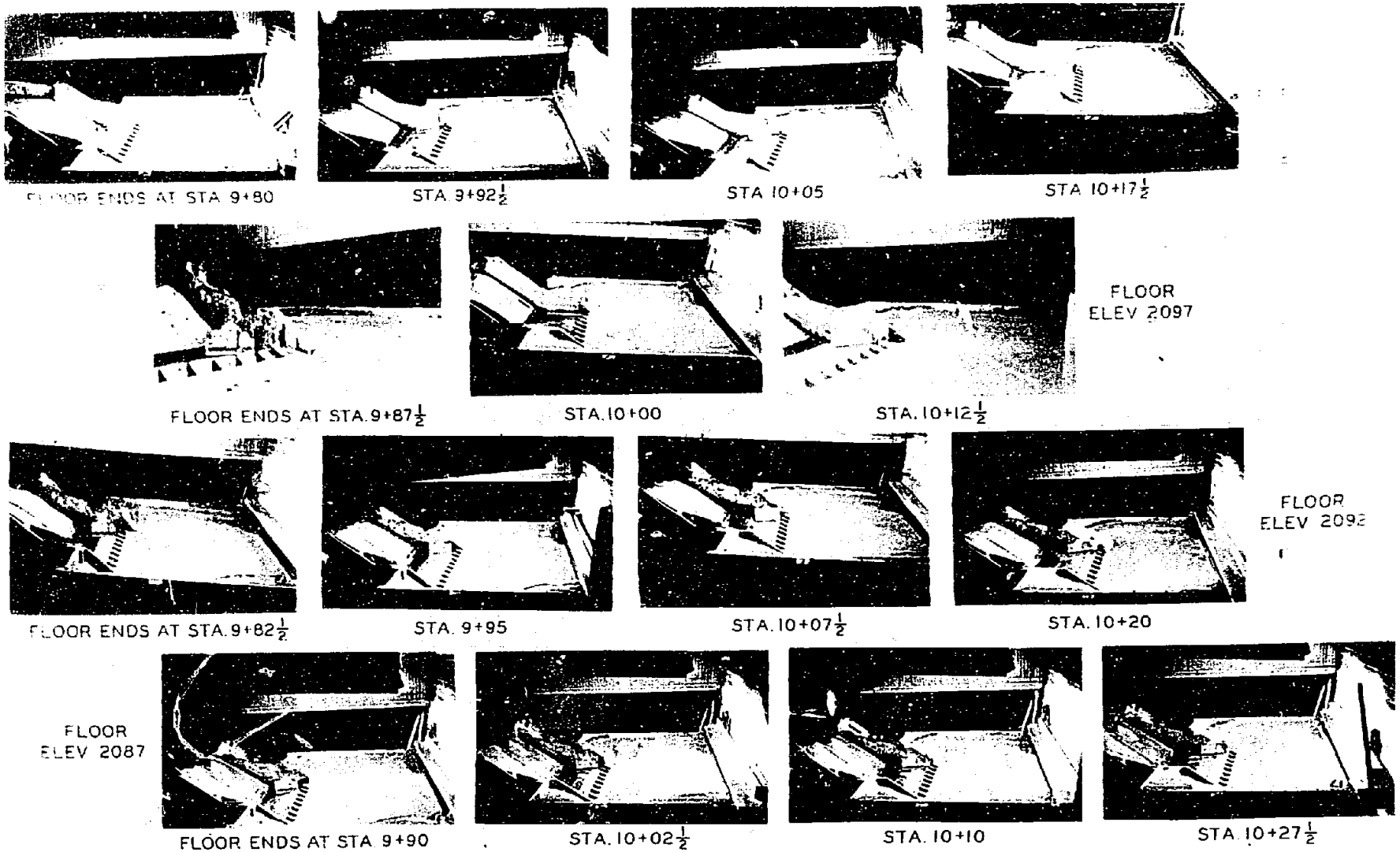
32-D-295

FIGURE 58





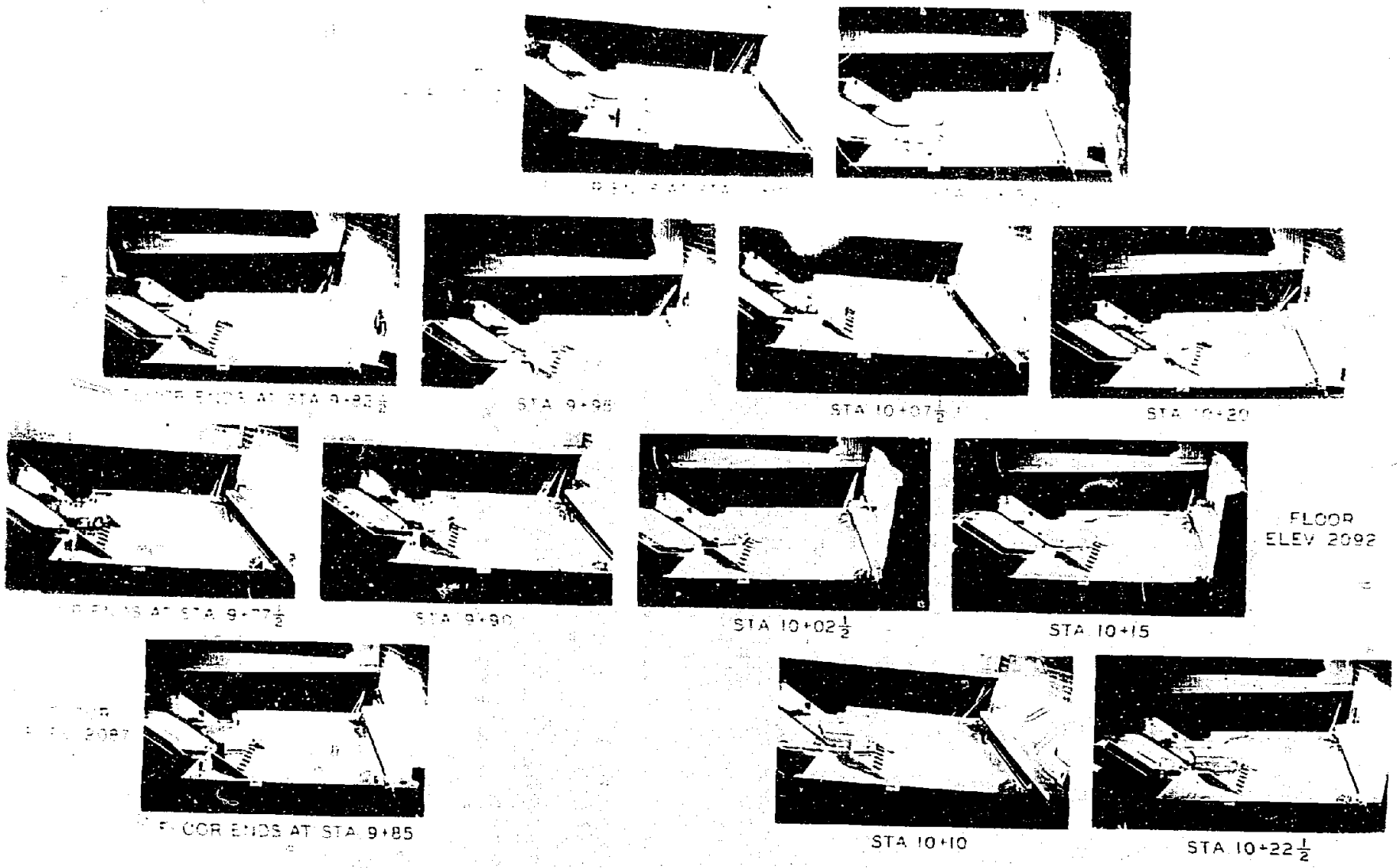
SHAPE OF SAND BIN BEFORE SCOUR TESTS



CLE ELUM SPILLWAY EXPERIMENTS

COMPARISON OF SCOUR WITH VARIOUS LENGTHS AND DEPTHS OF STILLING POOL

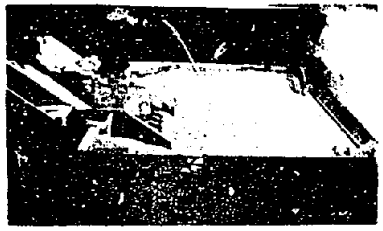
200 FOOT POOL WIDTH



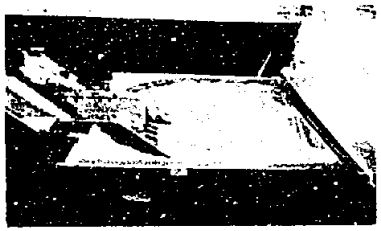
CLE ELUM SPILLWAY EXPERIMENTS

COMPARISON OF SCOUR WITH VARIOUS LENGTHS AND DEPTHS OF STILLING POOL

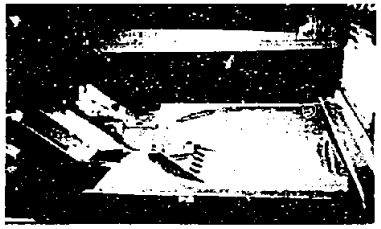
150 FOOT POOL WIDTH



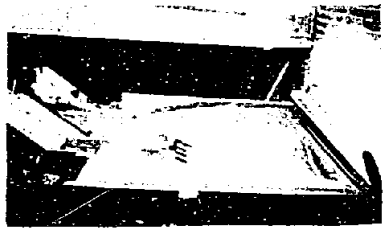
FLOOR ENDS AT STA. 9+95



STA. 10+07½

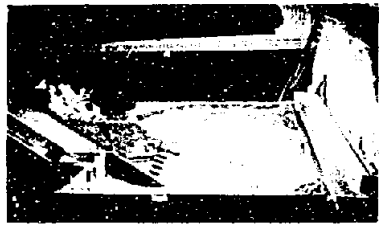


STA. 10+20

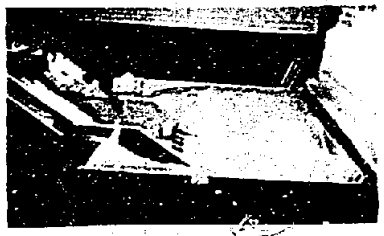


STA. 10+32½

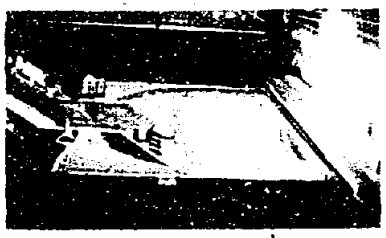
FLOOR ENDS AT STA. 9+90



STA. 10+02½



STA. 10+15



STA. 10+27½

CLE ELUM SPILLWAY EXPERIMENTS

COMPARISON OF SCOUR WITH VARIOUS LENGTHS AND DEPTHS OF STILLING POOL

120 FOOT POOL WIDTH

may show that lower values are unquestionably safe. Quite a number of dams with lower values have been built and have given satisfactory service over a period of years. Some experienced engineers use considerably lower values. In a number of cases however failures have occurred suddenly and without previous warning in structures which have given satisfactory service over a long period, ranging up to twenty years. If a set of values which were somewhat too low came into general use, it might be a long time before their insufficiency would be demonstrated and a large number of insecure structures would be built in the meantime. The author therefore believes that it is better to take a conservative view and make a small step in the right direction, permitting more experience to accumulate before taking another step, than to run the risk of overstepping and having to go back.

The present form of this paper has been a gradual development and is the third step in this process which the author has made. In the first report, which was based upon data from approximately one hundred dams, he recommended the plain creep theory with somewhat lower values than recommended by Bligh. As data from the Deoha Barrage, Kulli Dam and Kulli Bye Wash became available, the weakness of the plain creep analysis became evident and a second report was prepared based on data from 120 dams recommending a set of values with a weight for horizontal creep of one-half. Later data, largely furnished by the firms, Ambursen Dam Company; Ayres, Lewis, Norris and May; and Holland, Ackerman and Holland, which included many dams with a high proportion of vertical creep, gave adequate basis for using smaller relative weight to horizontal creep. The present report was therefore prepared, based on over two hundred dams, using a weight for horizontal creep of one-third.

Low values of creep for dams founded on sand have been recommended by Konig*. The shape of dam and sheet piling he advocated give

*Trans. Am. Soc. C. E., Vol. 73, p. 175.

weighted creeps varying from 8.5 for low dams to 3.6 or less for high ones. It is doubtful however if his recommendations for higher heads were based on actual experience.

An examination of minimum weighted creep ratios suggested as a result of this study shows that they are somewhat less than one-half those recommended by Bligh. A comparison is shown in the following table:

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Material	Weighted Creep Ratio	Bligh's Value
Very fine sand or silt	8.5	18
Fine sand	7.0	15
Coarse sand	5.0	12
Medium gravel	3.5	
Gravel and sand		9
Boulders with some cobbles and gravel	2.5	
Boulders, gravel and sand		4 to 6

This does not mean that Bligh's values are more than 100% too large, or that dams designed according to the weighted creep ratios will have only one-half the percolation distance which dams have had in the past. The difference does not represent as radical change from the present custom as a comparison of the ratios would suggest. The requirements of length to protect against scour, and to secure sufficient bearing pressure will still necessitate considerable horizontal creep. For dams on coarse gravel including cobbles and boulders with some cobbles and gravel, the necessity of conserving all the water may in some cases require greater percolation distances, as the values given may permit too much seepage.

The effect of the adoption of the weighted creep method will be to put the emphasis on deep cutoffs, and lead to their use as far as other limitations will permit. It will tend to eliminate the wide masonry floor type without substantial cutoffs. For example, to meet the weighted creep requirements for a weir on fine sand with no vertical creep, would require an ordinary creep distance of 3 x 7 = 21 times the head, while the ordinary creep distance according to Bligh should be 15 and according to Griffith 12½ to 16.

Because of the emphasis which the weighted creep method puts on deep cutoffs, if the greatest economy is to be secured, it will be necessary to investigate more fully than before the length of apron necessary to prevent scour, and the best shape of dam to accomplish this purpose. The pressure on the designer will be toward using as short an apron as possible and unless careful studies are made there will be a tendency to adopt one which is too short. It should be kept in mind that the weighted creep method deals with piping only, and the requirements of scour may dictate much more expensive structures than safety from piping would necessitate. The recent advances in hydraulic model tests have clearly shown the reliability of this means of determining the conditions necessary to prevent scour. The design of important structures should therefore include a hydraulic model study.

Other Considerations Influencing the Selection of Weighted Creep Ratios

Several factors besides the nature of the foundation material should be considered by the designing engineer in selecting the weighted creep or short path ratio to use in the design of the type of structure under consideration, if the most economical result is to be secured. The safe ratios given on page 25 may therefore be modified somewhat in view of these factors. No definite rules can be given, as it is a matter of judgment. It is believed, however, that the data available do not justify more than slight reductions in the ratios given, except for unimportant structures.

Perhaps the most important of these considerations is the damage to life and property which would result from a failure. For a dam so situated that a failure would result in loss of life an unquestionably safe design is required. If extensive property downstream would be endangered, conservatism of design is indicated. For most dams on porous foundations the conditions are such that little damage would result from their failure except to those directly served by them, and the degree of conservatism should be dictated by a comparison of the damage which would result from failure with the cost of the additional factors of safety. One phase necessary to consider in estimating the damage which is likely to result from failure is the time necessary to make repairs or temporarily patch up the structure. A failure which would put a hydro-electric plant out of service for a considerable time, or shut off the supply of irrigation water for a long enough period to cause the loss of a crop, or more especially of orchards or citrus fruit groves, would be very serious and justify larger ratios than for a structure which could be temporarily patched up in a short time. For this reason small structures of low head can usually be designed with lower weighted creep ratios for very minor structures perhaps as low as 80% as much as those given in the table.

Another consideration is the quality of supervision which the structure would have during and after construction. The weighted creep-head ratios suggested in this paper assume first class inspection during construction. It is doubtful if an engineer should consent to design a dam on porous foundations unless he was satisfied that it would be built with intelligent supervision, but should that not be available, higher values than those suggested should be used.

Most dams on porous foundations require more or less maintenance. Many dams have experienced difficulties from time to time which would have eventually led to failure without competent maintenance. Usually it is necessary occasionally to add riprap below them or repair the flexible apron. If repairs are not made with reasonable promptness scour at the lower edge of the structure may attain such proportions

that the conditions assumed in computing the travel paths for the design may no longer hold and the original security against piping no longer exist. Unless a reasonable degree of maintenance can be counted upon, higher values of weighted creep-head ratios than those suggested should be used.

For dams on coarse gravel, a factor which deserves consideration in selecting a creep head ratio is the money loss which would occur if some seepage took place beneath the dam without endangering the structure. For example, in dams for power plants, any loss would usually result in a decreased plant output, but for an intake dam for an irrigation system where only part of the water in the stream was diverted and water would therefore be continually wasted over the dam, the flow under the dam would be of no consequence.

Grouting beneath Horizontal Portions of Dams

Since the available evidence indicates that there is apt to be a poor contact between the foundation and horizontal portions of dam foundations, improving this contact by grouting is worthy of consideration. This could usually be done without great expense thru holes drilled thru the floor slabs. High pressures could not be used as they would lift the floor. By starting at one end of the structure and working toward the other, it is believed that good contact could be secured. If possible the grouting should be done before the head came on the structure, as otherwise much of the cement might be carried away in the flowing water.

Reverse Filters, Weep Holes and Drains

Another factor which should be considered in selecting the proper ratio to use in the design of a masonry dam on a porous foundation is the possibility of using reverse filters, weep holes or drains. The reverse filter was probably first developed by J. S. Beresford and seems to have been used extensively about twenty years ago. It was used on Zifta and Assuit Barrages on the Nile River and on the Hindia Barrage in Mesopotamia. It has been used in this country on dams Nos. 7 and 13 on the Mohawk River. It consists of a filter placed at the lower edge of the downstream apron, built up of a bottom layer of fine material surmounted by other layers of progressively coarser material. It is called a reverse filter because the position of the layers is the reverse of that in ordinary filters, but the water passes first through the fine and then the coarse material as in other filters. Its only function is to filter the water seeping beneath the dam and prevent the removal of any of the fine foundation material. Although usually covered with riprap, the reverse filter is apt to be damaged or destroyed by the high velocity water passing over the dam.

Holes through the apron of the dam or the wing walls, called weep holes, have been extensively used in America but no examples were found of their use in other countries. Most of the available literature on India and Egypt, however, was published many years ago and it is possible that they have come into use there since then. Their function is usually to relieve upward pressure beneath the apron of the dam and thus permit the use of a thinner apron without danger of its being lifted from under pressure. In recent years pipe drains have been employed in some cases instead of weep holes. The greatest advantage is secured by combining the functions of the reverse filter and weep hole by constructing a filter behind the weep holes or around the pipe drains. In this position the filter is protected from scour.

Since piping failure results from the removal of foundation material from beneath the dam, if a filter prevents the removal of this material, the dam will be safe, even though large quantities of water are passing beneath it. The quantity which passes through the filter depends upon the head acting on it. In ordinary filtration practice, a filter is frequently subjected to a head of water several times its thickness. If an ordinary reverse filter were subjected to such a head it would be lifted bodily. Such filters therefore cannot be subject to much head. If however the filter is weighted down by a sufficient thickness of concrete apron it can be subjected to a much higher head than a reverse filter and the dam be made safe for the passage of a greater quantity of water beneath it. With such a filter near the end of the percolation path of the water flowing beneath a dam, a greater velocity of flow in this water is therefore permissible and therefore a shorter percolation path can safely be used.

Weep holes and drains have been used with very short percolation distances between them and the headwater. In one case a flow through the weep holes sufficient to cause geysers several inches high occured without apparent ill effects.*

*Trans. Am. Soc. C. E., Vol. 88, 1925, p. 319.

A striking example of the effectiveness of vents came to light at the Coon Rapids Dam. This is an ogee structure having a thin downstream apron with steel sheet piling cutoffs under the heel of the dam at the downstream edge of the thin apron. Immediately behind the upstream cutoff a drain was constructed consisting of a reverse filter discharging into a small tunnel built in the dam. Due to an accident, one of the steel sheet piles of the upstream cutoff was pushed down far beyond its proper position leaving an opening in the cutoff, and reducing the creep distance to the drain to 0.6 times the head. In spite of this the flow through the drain does not seem to have been sufficient to excite alarm and the defect in the cutoff was not discovered until a large hole washed below the dam was pumped out during repairs to the apron and a blowout

occurred. Had the drain not existed it seems very probable that the thin concrete apron below the dam would have been blown up.

Table 5 gives data on dams using weep holes, drains or reverse filters, together with the creep head, weighted creep head and short path ratios to the most upstream vent, the foundation material and other pertinent data. The low ratio resulting from the short distance from the headwater to the upstream vent in many cases is striking.

The author does not believe that it is safe to use low ratios for drains or weep holes except where combined with a reverse filter. It is probable that the use of a drain without a filter contributed to the failure of the Corpus Christi Dam abutment.* The U. S. Bureau of Reclamation has for many years used weep holes with filters of gravel beneath with good success. Experience at the Lost River Dam shows that weep holes without filters or other course material behind them are liable to clog. It is the author's opinion that while weep holes

*Eng. News, Vol. 71, April 30, 1914, p. 968.

and drains are valuable devices, the greatest care must be exercised in their construction. A weep hole or drain which will permit the foundation material beneath the dam to pass through in effect, is a pipe already partially formed, and certainly no such weak spot in the protection of the dam should be deliberately constructed with the impression that it was adding to the safety of the structure.

Weep holes or drains have been used on dams on practically all classes of material. The necessity of a well constructed filter is especially evident in the case of weep holes or drains with clay foundations, as the fine clay could readily pass out through them unless restrained by an effective filter. In gravel a filter probably would form naturally by the washing out of the fine material near the weep hole or drain, in the same manner as a filter tends to form around the screen of a driven well. However, in view of the small expense in installing a filter behind the weep hole it is believed that their construction is justified even in gravel. They are absolutely necessary in sand or silt, if weep holes are depended upon to reduce the length of the percolation path required.

The location of the weep holes and filters should be given considerable study in dam design. From the standpoint of the prevention of piping, the best location for the filter is probably beneath the apron at the downstream end just upstream from the cutoff wall or row of sheet piling which is usually placed at the downstream edge of the structure, the weep holes discharging through the wall or piling into the riprap or other scour protection material. The downstream wall or

row of piling in this case does not act so effectively as a cutoff but is very desirable to prevent undermining. It need therefore be no longer than necessary to accomplish this purpose. To reduce the upward pressure on the apron it may be necessary to place the weep holes farther upstream. They should not be placed farther than necessary for this purpose (unless required by safety from sliding) for to do so shortens the percolation path with no advantage, but brings a greater pressure on the filter and therefore increases the danger of failure. For this reason it is believed that the installation of a drain immediately downstream from the upstream cutoff is not a good practice. On sand or silt foundations where flexible block or book slab pavements are used downstream from the solid apron of the dam, it may be desirable to install weep holes and filter at the downstream edge of the apron. With the block protection the water passing beneath the dam is prevented from escaping from a large part of the area just downstream from the apron but is free to escape thru the cracks between the blocks. This tends to produce concentrated flows thru these cracks and causes the removal of fine material, making an incipient pipe.

A word of caution also may not be amiss regarding drains beneath the apron of dams where a hydraulic jump is formed. If the drain discharges downstream from the jump while its position under the apron is upstream from the jump, the pressure beneath the apron will be equal to or greater than that where the drain discharges, which, being below the jump, will be greater than the pressure acting downward on the apron, due to the thin sheet of water upstream from the jump. This unbalanced pressure may be enough to lift the apron. In the Iron Mountain Dam the drain discharges at the toe of the ogee section, upstream from any jump which may form. In this case the drain may be downstream from the jump and discharge upstream, which would cause an unbalanced pressure acting downward on the apron, increasing its stability. Weep holes which discharge directly through the apron equalize the pressure on the two sides, but they should be constructed in such a manner that the high velocities can not suck foundation material through them.

If it can be conveniently done, it is advisable to have drains discharge where the flow from them can be observed, in order that any failure to function properly may become known. In Northern climates reliance should not be placed in weep holes or drains which may freeze up and thus become inoperative.

It is believed that by the use of weep holes or drains constructed with reverse filters which prevent the removal of the foundation material a reduction of the safe weighted distances given on page 25 may be permitted. This assumes that the vents are of sufficient capacity, are free from freezing and are located a sufficient distance upstream from the lower end of the percolation path that practically all of the under-

seepage will reach the tailwater through them. No fixed rule can be given on these requirements, but the exercise of reasonable judgment should prevent serious mistakes. For the location just upstream from the downstream cutoff a reduction of 10% might be permitted, and smaller reductions could be allowed for less favorable locations.

Effect of Silting of Riverbed above the Dam

The possibility of silting of the riverbed above the dam should be given consideration in determining the permissible percolation ratio, as it may greatly reduce the upward pressure beneath the dam and the probability of failure by piping. The upward pressure on the Island Park Dam was almost entirely removed in two years or less, only the upward pressure from ground water remaining. The upward pressure on the Hamilton Dam was practically eliminated by silt deposit, in spite of the fact that a gravel plant continuously excavated gravel 100 to 200 feet upstream. Both of these dams are on the Miami River, which has a gravel bed (See Figure 1.) and does not carry an unusual amount of silt. On the other hand, the upward pressure on the Sherman Island Dam on the Hudson River does not seem to have changed materially over a period of five years.* It is possible, however, that considerable

*Trans. Am. Soc. C. E., Vol. 93, 1929, p. 1576.

silting took place when the pond was first filled, which apparently was about 2 months before upward pressure measurements began. The silt deposits above the Granite Reef Dam cut off extensive seepage which took place when the dam first came into use. For an intake dam, where the entire width is composed gates, extending nearly down to the riverbed, silting could not be relied upon to increase the security, as the flow through the gates might remove the impervious silt bed. This effect has been noted at the Grand Valley Dam on the Colorado River.**

**Trans. Am. Soc. C. E., Vol. 93, 1929, p. 1530.

It would not be advisable to count on the silting up above a dam to reduce the necessary travel ratios and at the same time count on vents for further reduction, since if the silting up was effective, the vents would be inoperative. It must be remembered, moreover, that immediately after closure, before the bed has had a chance to silt up, the dam would be subjected to more severe piping conditions, and, therefore, it would be safe to assume in the design only the limited reduction which the dam might stand until it silted up.

Dams on Pervious Material Underlaid by Clay or Hardpan

A number of cases were found of dams resting on pervious material underlaid by clay or hardpan with the cutoffs extending into the impervious material. These cannot be analyzed according to either Bligh's or the short-path method. In fact, no satisfactory method of treating them has been found and therefore the data only is given in Table 6. In determining this data, the short-path in the clay is assumed to follow the creep line, as it seems unreasonable to consider a path directly through the practically impervious clay.

Consider such a dam founded on a thick layer of pervious material with a sheet pile cutoff extending a few feet into the clay beneath. If well constructed, such a cutoff would nearly entirely stop the flow of the water beneath the dam and result in a head approximately equal to headwater elevation on the upstream side of the piling and to the tailwater level on the downstream side. This condition is strikingly shown by the upward pressure measurements on the Sulemanki Weir in India.*

*Stability of Weir and Canal Works - A. N. Khosla, Punjab Engineering Congress, 1930 - Paper No. 142.

Nearly the entire head of the dam would therefore be concentrated on the few feet of creep along the piling in the clay. This would be a severe test on this contact and if even a minute passageway existed water would rapidly flow through it and would probably enlarge it to a certain extent. However, unless the pervious material was very course, the clay washed out in enlarging this hole would be filtered out in passing through the pervious material and form an impervious coating on the "filter" which would greatly retard flow through it. Also, the loss of head of the water in passing at high velocity through the pervious material in the immediate vicinity of the upstream and downstream ends of the "pipe" through the clay would decrease the velocity and reduce the scour in the "pipe." This action is somewhat analogous to the flow through a nearly closed valve. Under ordinary conditions, therefore, no great enlargement of the opening could be expected.

Another reason why this would be the case is that the flow coming through the small opening upon entering the pervious material would spread out into the greater flow area of the pervious material and lose velocity. By the time it had reached the surface where movement of the particles of pervious material was possible it would have too low a velocity to cause movement. No significant movement of the particles near the bottom of the sheet piling would be possible because they would be surrounded by material which was not moved. Therefore, unless there were enough of these openings to permit sufficient flow to cause piping velocities at the downstream edge of the dam, no danger to the structure would result from them.

It is evident, therefore, that cutoffs into impervious material beneath dams greatly increase the safety of the dam against piping even if the contact between the cutoff and the impervious material is not watertight. No satisfactory method has been worked out for estimating how much reduction of travel path can be allowed under these conditions, and since the data at hand is slight, it must remain for the present largely a matter of judgment. For this sort of foundation however, great care must be exercised to prevent the concentration of flow around the end of the pile cutoff from forming a pipe under of behind the abutment.

Dams on Clay Underlaid by Pervious materials

Structures which rest on a thin layer of clay underlaid by pervious material seem especially likely to cause trouble. Apparently the water frequently enters the pervious layer over a large area and seeks to escape at the downstream end of the structure in a comparatively small area, giving rise to flows of considerable magnitude which are apt to endanger the structure and possibly may blow up the impervious layer due to excessive upward pressure. The pressure in the pervious layer is sometimes due to groundwater conditions of the surrounding terrain and only remotely connected with the local headwater conditions.

The Pittsfield Dam seems to have had this kind of foundation, its failure being probably due to a blow up of the impervious layer. The Nathaura Escape Head was another case of difficulty and final failure under these conditions. As the details of this failure are not accessible to most engineers, a summary is given in Appendix III.

In the report describing the Nathaura failure mention is made of difficulties at the Udra, Rarwa and Jaberia culverts of the same canal under similar conditions. Mr. Khosla* has described in detail the difficulties with the Dugri No. 1 and Jauryan siphons in the Punjab and the remedies unsuccessfully and successfully used to remedy the trouble. Although success is sometimes achieved under these conditions** the number of failures and near failures should serve to warn one contemplating the construction of a dam on such a foundation to look into the matter very carefully. As the conditions may vary so greatly it is probably best not to attempt to prescribe a general rule.

*Punjab Engineering Congress, 1930 - Papers Nos. 138 and 142.

**The Design and Construction of the Bassano Dam, Engineering News, Vol. 72, August 27, 1914, p. 425.

Limitations of Creep Analysis

As has already been pointed out, piping may occur as a result of percolation along the lines of contact of the dam with its foundations, or by flow through the foundation material itself. In a given case failure would result by travel along the path which offered the least resistance.

In order to determine experimentally the length of percolation path necessary to prevent piping from flow through the material Mr. W. M Griffith performed a series of experiments with a masonry trough 1' x 1½ ft. in section, filled with fine micaceous sand, and covered with earth, to determine the head of water necessary* to cause a blow-out to occur. The results showed that a blowout did not occur until

*Data from an unpublished portion of Mr. Griffith's paper previously mentioned.

the ratio of length of creep to head reached values of 3.33 to 2.5 although experience had shown that creep head ratios of about 14½ were necessary to secure safety in dams built on this sand. He found that the flow of water through the sand column increased directly with the head until failure occurred, which came suddenly, without the previous formation of springs to form a channel and carry away the material. He also found that loading the sand decreased the tendency to blow out, and that water brought up suddenly on dry sand increased the tendency.

Much the same results were obtained by Terzaghi**, who found that the water percolating through the sand increased with the head until a

**Technical Publication No. 215, p. 36, Am. Inst. of Min. & Met. Engrs.

point was reached at which the permeability of the sand suddenly decreases, and the velocity through it consequently increases, and a blow-out takes place.

This increase in permeability is the result of a change of position of the sand grains to a less compact arrangement, giving them an increased bulk and greater voids. The effect of the added weight in Griffith's experiments was probably to resist this change of volume and therefore increase the head necessary to cause a blowout. This phenomenon has also been studied by Osborn Reynolds***, and by Dr. Warren J. Mead****.

***Osborn Reynolds Scientific Papers, Vol. II (1901), p. 217.
****The Geologic Role of Dilatancy - Journal of Geology, Vol. XXXIII, No. 7, October, November 1925.

In his paper "On Percolation Under Aprons of Irrigation Works," S. Leliavsky discusses a number of experiments to establish the head ratio at which blowout occurs and explains the difference in the results obtained by different experimenters. This is largely due to the slope of the surface of the material where the water emerges. If the surface is inclined, less pressure is necessary to move the particles than if the surface is level, as gravity assists in the movement and a blowout will therefore take place with lower head.

The data on the head necessary to cause blowouts by flow directly through the material from actual dams is meagre. The only dam in which failure unquestionably took place by flow directly through the foundation material is that at Coon Rapids. There the tailwater level was so far beneath the apron of the dam that percolation could not take place along the line of creep, at least part of the distance. This dam was on gravel and failed with a short path ratio of 2.7. It is probable however that the surface where the blowout occurred was on a slope, and therefore a blowout occurred at a less head than would have been the case if the surface was level.

The failures at the Port Angeles and Puentes Dams may have been directly through the foundation material. In the former this seems especially likely. The situation there however was complicated by the piling which was driven just below the dam, which probably concentrated the flow and caused the blowout, which took place with a short-path ratio of 1.9, to occur at a less ratio than would otherwise have been the case. The Puentes Dam failed with a short-path of 1.4 and probably would have held at 2.4, although it had not sustained this ratio long enough to be sure.

It would seem that considerable light on this subject would be shed by experience with cofferdams, as a great many of these have been constructed. When cofferdams which are sunk in a saturated material are pumped out, water flows up through the bottom tending to raise the foundation material and make it flow into the cofferdam. If the water level outside the cofferdam is at the same level as the ground surface and the excavation in the cofferdam at the level of the bottom of the sheet piling when the water inside the cofferdam is pumped out, the head acting is approximately equal to the length of percolation path, making the short path (also the creep ratio) practically 1.0. The length of path might be somewhat greater than the head, but this would probably be more than offset by the fact that the downward flow of the water outside the cofferdam would take place through a much larger area than the flow upward into the cofferdam. The condition would be that of flow through a channel of gradually decreasing cross section and the velocity at the smaller end (in the cofferdam) would therefore be greater than for flow through a uniform prism of material with a length equal to the head.

Time has not been available to go into this phase of the matter in detail, but two examples which have come to the writer's attention may be mentioned. In the case of a number of bridge pier foundations in the gravel of the Arkansas River bed at Pueblo, Colorado, there was no tendency of the gravel to flow up into the cofferdams. In a number of cofferdams in Florida, with sand of the fineness of table salt, there was a considerable flow. These would indicate that a short path ratio of 1.0 would be sufficient for gravel and not sufficient for fine sand. Mr. W. M. Griffith has mentioned a case of a cofferdam in fine quartz sand with small stones which failed with a short path ratio of 1.95. The author hopes that the readers of this paper will supply instances which will throw light on this point.

The data is insufficient to define closely the safe limit of length from the standpoint of flow directly through the foundation material. It does indicate however in a rough way that the assumption is justified that the flow takes place directly through the material when the distance is less than half the weighted creep distance. On this basis one might assume that the limit of safe short path would be one-half that of safe weighted creep. There is another factor which enters here however. Even with well designed and maintained dams, scour may occasionally take place at the downstream edge. Under these conditions the material on the side of the hole is more easily moved by percolating water than if it were on a flat surface. Moreover, the percolating water reduces the effective weight of the particles at the downstream edge of the dam and makes them more subject to scour. For this reason it is believed advisable to limit the short-path ratios to eight-tenths of those given for weighted creep. This limitation will seldom control the dimensions of the dam.

As previously stated, there is a defect in the creep analysis when rows of sheet piling are too close together. Bligh recognized this and stated that the percolation followed the line of creep as long as rows of sheet piling were not closer together than twice their depth.* Some

*The Practical Design of Irrigation Works, Bligh 2nd. Ed., p. 168.

such limitation of sheet pile spacing is necessary, for it is obvious that two rows of long sheet piling very close together would offer little more resistance to piping than one row, while if the rows were a great distance apart they would offer considerably more resistance than one row. Bligh did not suggest any reason for using the limit of twice the depth rather than some other value, nor did he discuss what would take place if piling rows were spaced closer than his limiting value.

As the flow follows the path of least resistance, Bligh's limit seems to be equivalent to stating that when the piles are spaced twice

their depth the resistance along the creep line is equal to that directly through the material and when closer than twice their depth the resistance directly through the material is less than along the line of creep. When the piling are spaced twice their depth apart the creep distance between their lower ends is twice the short path distance and since for this condition the resistance along these two paths are equal, the resistance per unit length along the short path is twice that per unit length along the creep line. Wherever there are two points on the creep path so close together that a path exists directly through the foundation material which is less than half the weighted creep distance between the points, the flow would take place directly through the material with a resistance of twice that along the creep path. Where several overlapping short paths are thus possible, the one which gives the smallest combined weighted creep and short-path distance for the entire structure is the one on which the travel would take place.

For example, two lines of steel sheet piling were driven for the Middle Loup River Dam, 31 feet apart. One extends 29 feet below the bottom of the dam and the other 39 feet below. One possible short path which fulfills the above conditions extends horizontally from the end of 29 foot piling to the longer piles. Other possible paths extend from various points on the shorter row of piling to various points along the longer row. The path extending between the lower ends of the sheet piling rows gives the lowest combined weighted creep and short-path distance for the structure of all the possible paths, and should therefore be adopted as the path to be used. Where the water follows the short path, the path connecting the ends of the two adjacent sheet piling rows will usually give the lowest weighted creep for the structure.

The above reasoning applies also to the weighted creep. In this case the horizontal creep is considered to have a weight of one-third, the vertical creep a weight of one and the short-path a weight of two. In computing the plain and weighted creeps for existing dams, these rules have been used wherever applicable.

The rule that the flow follows the short-path if its length is less than one-half the weighted creep distance between any two points in the creep path permits somewhat smaller pile spacing than Bligh's rule, since the horizontal creep, which in Bligh's rule has weight of one in computing the creep distance, has a weight of only one-third in computing the weighted creep distance.

If boring records indicate a very pervious layer extending beneath the dam and coming near the surface near the downstream toe, more conservatism in the short path distance will be necessary, for a Terzaghi* has pointed out this condition is conducive to high velocities where the water emerges and therefore to danger of piping.

*Technical Publication No. 215, Am. Inst. Min. & Met. Engrs., p. 31.

Difficulties of the Flow Net and Electro-Hydraulic Analogy Methods

As previously stated, the difficulties in the use of the flow net and electro-hydraulic analogy method are the necessity of a detailed knowledge of the permeability of the various portions of the foundation material and the lack of data showing what velocities can safely be permitted in the various kinds of foundation material.

In order to reach a reliable result, not only must the relative permeability along the contact and through the foundation material be known, but the position and relative permeability of all the layers beneath the dam must also be known, and they must not change materially in short distances along the dam, since the analysis considers flow in two dimensions only, while actually it may take place in three. In connection with the studies for this paper the author had the opportunity to examine the boring records for a number of dams, and while he did not give them the intensive study which one would if he were to design the dams at those sites, it was apparent that it would be impossible to form an accurate idea of the position and relative impermeability of the various layers from boring records even in a single cross section, and still less possible to consider the permeability in three dimensions. A good example of the variability of foundation materials of ten encountered is that shown by the borings for the Oradell Dam*. Some sand foundations are relatively uniform, but even here the permeability would vary considerably. Moreover, even if all of these factors were known, so that the true velocity of the water emerging below the dam was susceptible of accurate determination, it would be difficult to relate this to dam safety, as safe velocities for the various classes of material are not even approximately known. Some experimental work has been done along this line, but much more work and comparison with actual structures is necessary before they can be safely used as a basis of design.

*Trans. Am. Soc. C. E., Vol. 89, 1926, p. 1187.

The Electro-Hydraulic Analogy Apparatus

The extreme variability even of screened sand under laboratory conditions was strikingly brought out by some of the author's experiments with the electro-hydraulic analogy apparatus. The author is indebted to Mr. Ford Kurtz, M., Am. Soc. C. E., for a general description of the Pavlovsky apparatus. As details were not available it was necessary to develop them and the apparatus as built probably differs considerably from that used by Pavlovsky. At the suggestion of Mr. Walter H. Price a salt solution was substituted for the metal plate

and alternating current indicated by earphones was used to observe the potential. The apparatus consisted of a shallow tray 18" x 36" with plate glass bottom and sides, filled to a depth of 1/4"with a 5% solution of common salt as shown in Figure 6 and provided with means for accurate leveling to insure a uniform water depth throughout. Near the center of one side were placed strips of insulating material representing the sheet piling extending from the glass (to which they were cemented with a watertight joint) to above the top of the salt solution. At each side of this "dam" was placed copper terminals at the position of the contact of the headwater and tailwater with the foundation material.

These were connected to a 110-volt alternating current source thru a lamp bank which reduced the voltage drop across the terminals to about 18 volts. At the opposite side of the tray over a meter scale, was a high resistance wire, one meter long, each end of which was connected to one of the terminals by a heavy copper wire. A spring is necessary at one end of this wire to keep it in tension under all conditions of temperature introduced by the current flowing thru it. On this resistance wire was a sliding contact connected thru a set of earphones to a probing pencil. In passing from one terminal to the other the current follows paths as shown by the light line connecting the terminals. The heavier lines crossing these represent lines of equal potential plotted for increments of 5% of the total potential drop. It is the purpose of the apparatus to determine these lines of equal potential. With these determined the current paths can be drawn as will be described later. To determine the position of any desired potential line, the sliding contact on the resistance wire was placed at the point giving this potential and the probing pencil moved around in the salt solution until the telephone receivers indicated that no current flowed thru the wire from the sliding contact to the probing pencil. The position of this point was observed by means of a system of coordinates ruled on a paper, placed beneath the bottom of the plate glass tray. For example, suppose it was desired to find the position of the potential line representing a drop of potential from the "headwater" terminal of 40% of the total drop between the terminals. The sliding contact was placed on the resistance wire at 40% of its length or 40 c.m. from the end connected to the "headwater" terminal. Since the resistance of the heavy wires between the terminals and the ends of the resistance wire was negligible, and the resistance wire was uniform in cross section and resistivity, the resistance from the headwater terminal to the sliding contact was 40% of the total resistance between terminals and therefore the potential drop to this point was 40% of the total potential drop. When the probing point was placed at any position in the salt solution a current would flow from the resistance wire to the solution or visa versa if there was a difference of potential between these points. When the

probing point reached a point where the hum in the receiver stopped, it indicated no current flowed and that the potential at the probing point was the same as that at the contact point on the resistance wire; namely, a 40% of the potential drop. By determining the coordinates of several points of equal potential and joining them with a line, the position of the line of equal potential could be plotted. If it was desired to determine the potential at any designated point in the salt solution, by placing the probe at this point and moving the sliding contact until it reached a point of equal potential as indicated by no flow thru the receiver, the potential of the probing point could be determined from the position of the sliding contact. Experience with this apparatus showed that potential lines could be located with a high degree of accuracy. To check it an experiment was performed with the tray clear and terminals entirely across each end. The potential lines with this setup for equal potential increments should be straight, equally spaced lines parallel to the terminals, and the results obtained agreed very closely with their theoretical position, as shown on Drawing 5 of Figure 7.

The mathematical theory* of the flow net is difficult, but the

*Hydraulic Laboratory Practice, p. 605.

principles of flow of electric current thru the salt solution in the glass tray are easily understood. For example, suppose the setup of the apparatus is such that the discharge thru the apparatus as shown on Figure 6 is 5 amperes and the total voltage drop is 20 volts. If the flow lines drawn on Figure 6 represent the path of flow of the current, each strip leading from the headwater to the tailwater terminals carries one ampere of current, and in flowing between adjacent potential lines there is a drop of one volt in potential. Since the depth and resistivity of the salt solution is the same throughout, where the potential lines are close together indicating a sudden potential drop, there must be a concentration of current since the potential drop for a conductor of given cross section varies directly with the current. Closely spaced potential lines therefore indicate high flow concentrations, and it is not necessary to draw the flow lines to determine most of the facts which the flow net gives. According to the theory of hydrodynamics the flow lines should cross the potential lines at right angles in the same way that water flows at right angles to the topographic contour lines on a hillside.

The ordinary laws of flow of water thru a homogeneous soil is directly analogous to the flow of electrical current thru a homogeneous conductor since in both cases the quantity varies directly with the pressure (voltage) and inversely with the area and length. Therefore, by substituting a uniform soil for the salt solution the distribution of pressure and flow thru the soil should be the same as the flow of

current thru the salt water. By varying the resistance of the electrical conductor in such a way that at all points its resistivity is proportional to the water conductivity of the foundation material under a dam at the corresponding point, it is possible to determine the flow of water beneath the dam for any given conditions. With some modifications the Pavlovsky apparatus can also be used for investigating saturation lines in earth dams and has numerous other hydraulic applications.

Comparison of Results Obtained with Electro-Hydraulic Analogy and a Model Dam on Sand

Extensive experiments on the flow of water under dams were performed by Colman.* He used a model dam formed in a large tank, and

*Trans., Am. Soc. C. E., Vol. 80, 1916, p. 421.

measured the pressure distribution in the sand beneath the dam by means of over forty piezometers placed at various points in it. In order to test the reliability of the electro-hydraulic analogy apparatus the author made a setup corresponding to that used by Colman. The results by the two methods for the runs where screened sand was used and where there was not obvious leakage in the Colman apparatus are shown on Figure 7, those at the right being the results obtained by Colman and those at the left showing the results obtained with analogous setups on the electrical apparatus. In spite of the fact that the results shown for the Colman tests are averages of several runs, they are obviously much more erratic than those obtained with the electrical method, which plot in smooth, regular lines. There was a much greater resistance to the flow of water thru the sand near the entrance and exit of the water. Colman's comparison of this to entrance and exit losses is not sound, as the velocities were so low that such losses would be negligible. It seems certain that greater resistance at those two localities was due to a difference in the state of the sand at these points, probably a difference in the compacting and size and possibly somewhat to silting up or to a growth of plant or animal life. That the sand might vary considerably in resistance with a little difference in compacting can be appreciated when it is realized that a change of voids from 40 to 30% will increase its resistance 150 to 250%. Colman's paper makes no mention of precautions to secure uniform compacting or insure homogeneous material except to remove all particles of over 1/2 inch size. Small differences in size also have a great effect on resistance. Decreasing the grain size from 0.35 to 0.25 m.m. for example would cause an increase of 100% in the resistance. It is probable therefore that differences of compacting and some slight lack of absolute uniformity of the grain sizes throughout the material accounts for the obvious irregularity of the flow thru the Colman apparatus. The regularity of the lines obtained with

the electrical apparatus however indicates that this method can be relied upon to give results with a high degree of accuracy.

The obvious variability of the water transmission properties of the sand used in the Colman apparatus serves to emphasize the difficulty which is inherent in the application of the flow net method to the design of actual structures. If a laboratory experiment using screened sand gives variations within a few feet of several hundred per cent in the resistance to flow, it is unlikelythat in a natural river bed where great variations in size of the material exist, estimates of resistance can be made with sufficient accuracy for safe design except under very favorable conditions.

Upward Pressure

In masonry dams upward pressure is important because of its tendency to reduce the factor of safety against sliding. Sliding is not so important a factor in the design of masonry dams on earth foundations. The principal danger from upward pressure on a dam on earth foundations is that it will be sufficient to lift up some portion of the dam, breaking a new outlet opening for the percolation which will give such a short percolation path that thedam will fail. Upward pressure is therefore a factor in the failure of dams from piping.

As previously shown, dams may fail from piping by flow along the line of contact of the structure with its foundation or by flow directly through the foundation material. In all dams some flow takes place directly through the foundation material and no doubt the usual condition is that most of the flow occurs in this way. If a structure is designed so as to be safe against piping, the upward pressure under practically all of it will probably depend more upon the conditions of flow through the foundation material than along the line of contact. Even if a pipe did form it would cover only a small area and the foundation masonry would bridge over it. Therefore although it is necessary to give creep along the contact the preeminent place in determining the safety against piping, it may have less bearing in regard to upward pressure.

Under conditions where the foundation material and its distribution is accurately known, it seems probable that some light on the upward pressure to be provided for could be obtained with considerable accuracy by the flow net or electro-hydraulic analogy, although these results would be modified somewhat by the case of creep along horizontal surfaces. The author hopes in the near future to investigate this further by means of a comparison of observed upward pressures on actual structures and the results obtained by the electrical analogy method. Where the foundation conditions are complicated or unknown, some approximate rule must be used. Measurement of upward pressure

on actual dams and research by model experiment, flow net, and electro-hydraulic analogy methods seem to indicate that the drop in pressure caused by a cutoff is greater in proportion to the creep distance than for the horizontal contacts. Leliavsky*, after a study of the works

*On Percolation Under Aprons of Irrigation Works, p. 46.

of Coleman, Forschheimer, Terzaghi and Pavlovsky states, "Thus in calculating the hydraulic gradient the depth of the sheet piling should be multiplied by a coefficient = 2, (sic) whereas there is already sufficient evidence to show that this coefficient should be three or four or more." In other words, the horizontal creep should be weighted three-fourths, one-half or less. Until more definite data is obtained therefore it is suggested that where it is necessary to use an empirical rule, the upward pressure be assumed to vary along the contact line in proportion to the weighted creep; i.e., that the pressure drop along the vertical or steeply sloping contacts per unit of length be assumed to be three times that along horizontal contacts or slightly sloping contacts. This is believed to be a conservative rule as upward pressure experiments frequently indicate that a weight for horizontal creep of zero would give reasonably close results.

Where sheet piling is so close together that the flow may be considered to follow the short path between the piling rather than the line of creep, the upward pressure along the creep line between the ends of the short path may be considered to vary according to the weighted creep distance between these points.

Since the weighted creep rule gives less slope of the piezometric line for horizontal creeps than the method advocated by Bligh, and since ordinarily the cutoff walls, which constitute the greater part of the vertical creep are located in the upstream portion of the dam, the weighted creep rule usually indicates less upward pressure under the downstream portion of the dam than the Bligh rule. Measurements of upward pressures on existing dams indicate results nearer to the weighted creep method. In other words, for the portion of the dam where the magnitude of the upward pressure is important, the Bligh rule gives results which are too large. This shows why that rule has given satisfactory results, although frequently the thickness of masonry used has been little if any greater than the theory would indicate necessary. The factor of safety therefore has been in the theory and none was therefore necessary in applying it. It would be much more logical therefore to apply a more correct theory with a reasonable factor of safety. It is suggested therefore that a factor of safety of at least 25% be used in determining the thickness of masonry required, to take care of the variations of the pressure from that indicated by the theory.

Special conditions sometimes cause unusual distribution of upward pressure. A dam founded on pervious material underlaid with impervious material and with a cutoff extending nearly to impervious material, will have a lower uplift downstream from the cutoff. A dam with such a foundation, where the cross section of pervious material under the downstream end of the apron is less than that at the upstream edge, will have a greater upward pressure over the entire base than one founded on pervious material of great depth. This condition was noted in the upward pressure measurements on the Grand Valley Dam*. A contracting of the space through which the water flows

*Trans., Am. Soc. C. E., Vol. 93, 1929, p. 1532.

beneath the dam acts as a partially closed valve and raises the pressure upstream from it and lowers it downstream.

In computing upward pressure it should be remembered that the conservative assumptions may be different from those for safety from piping. In the latter case the creep under flexible aprons and similar construction downstream from the solid portion of the dam, should be neglected in estimating the security of the dam against piping, as their effect is uncertain and cannot be relied upon. In computing the upward pressure they should be given consideration however, as they may cause greater upward pressure to occur at the downstream edge of the solid portion of the dam than the other assumption would indicate. The upward pressure experiments on the Pinhook Dam,**for example, showed a considerable upward pressure at the downstream edge

**Trans., Am. Soc. C. E., Vol. 93, 1929, p. 1551.

of the solid portion of the dam, due no doubt to the flexible apron downstream, although if the ordinary assumption for the design of such structures for safety against piping were used in determining the position of the end of the creep path, there would be no upward pressure at that point.

It is perhaps worth while at this point to suggest that the upward pressure be assumed to be applied at the bottom of the foundation as in fact it really is, and has a magnitude equal to the difference in elevation between this point and the piezometric line. The total weight of the masonry (in air) and of any water which may be above it, can then be considered as resisting this pressure. This will eliminate any uncertainty as to whether or not the masonry ought to be considered as submerged, and therefore as having lost weight, and prevent a mistake sometimes made, of measuring the upward pressure by taking the difference between the elevation of the top of the apron and the piezometric line, while still assuming the full weight of the masonry available to resist it. Experiments by H. de B. Parsons**and

**Trans., Am. Soc. C. E., Vol. 93, 1929, p. 1317.

ors have shown the necessity of assuming that the upward pressure s over the entire area of the base of the dam.

Miscellaneous Causes of Failure

Although a dam may apparently be designed with sufficient length percolation path, it may still fail by piping, due to causes which designer may not foresee. Failure may also occur from lack of e in construction. A number of cases in both classes are described the accounts of dam failures given in Appendix III of this paper, t it is believed to be desirable, by way of emphasis, to assemble s data in convenient form and to group these points together.

Too great emphasis cannot be given to the necessity of having the toff walls absolutely tight, tied into the main masonry of the dam d in intimate contact with the foundation material. A single pile sing caused the failure at the Coon Rapids Dam and leakage between e piles and between their tops and the masonry was a large factor n the failure of the Deoha Barrage. Even with interlocking steel eet piling, rigid inspection is necessary to see that there are no eaks in the interlocking and that their tops are properly embedded n the masonry of the main structure. The author suggests that sheet piles be driven in such a way that at no time is the lower end of a pile more than say 4 feet away from the ends of the piles on both sides of it. When thus driven the adjacent piles support each other and there is less probability of bending out of line or breaking the interlock. This scheme was successfully used in driving piling to 90 ft. depth thru sand and gravel in the construction of the core all trench of the Wanaque Dam*. This method will involve more moving

ng. News-Record, Vol. 89, July 20, 1922, p. 92.

f the pile driver and therefore some additional expense, but it is elieved that the additional security would fully justify it. The utoffs must also be tied into the structures in such a way that the ater cannot find a passage around their ends, as occurred in the eoha Barrage, since the water will always seek the easiest path and ever fail to find a weak point if one exists. Where cutoffs are uilt in trenches, great care should be exercised to secure intimate ontact between the wall and the sides of the trench. If possible the masonry should be built directly against the earth, and if sheeting is used in the trench it should never be allowed to remain, since it will cause passages along the wall through which water can pass with little resistance. This is believed to have been the cause of the failure of the Plattsburg Dam.

Considerable study is necessary in the design of the abutments to be sure that no path exists either under or behind them with a travel distance so short as to cause piping. The same rules should apply to these paths as to those under the dam. Travel along the back of the wall (or along the top of the floor in the case of a buttressed wall) should be considered as vertical creep. The water can usually follow any one of a number of paths, and it is therefore necessary to consider every possible one, for if there is a short cut, the water is sure to concentrate its flow in it. Such a short cut is believed to have been the cause of the failure of the Cornus Christi Dam.

Care must be exercised in the use of puddle aprons to see that they are not located where they can be scoured away or where they can dry out and crack. The former accident resulted at the Narora weir, even though there was a layer of riprap over it, and the latter occurred at the Kitcha Bye-wash. Care should be taken also in the case of a dam on clay or similar earth, if the foundation at any time has a chance to dry out. The failure of the Dolgarrog Dam can probably be ascribed to this cause, and cutoff walls should therefore be carried down to earth which will never dry. It is probable that greater travel distances should be allowed even in gravel or sand if the foundation has a chance to dry out. It is possible that this was a factor in the Kitcha Bye-wash failure. Trouble has also been experienced in securing watertight contact between puddle aprons and masonry dams.

Consideration should be given to the treatment of springs in the foundation, so that they do not wash channels along which piping can later take place. If springs are found in the foundation excavation it is advisable to try to smother them. In case they are due to the fact that the bottom of the excavation is below the surrounding water level, outlets for the springs should be provided until the pressure can be equalized, and they can then be plugged. If they are due to ground water, and are in the form of permanent springs, it may be desirable to provide a permanent outlet with a filter and weephole or drain, to eliminate the possibility of them developing sufficient head to force an exit for themselves. Water from permanent springs can frequently be differentiated from that from the river by a difference in temperature.

Designed creep distances can be materially shortened by a blow-up of the base of the dam from upward pressure. This action was probably a material factor in the failure of the Puentes, Narora and Deoha Dams. The apron must be heavy enough to resist raising.

Although not connected with piping, it may be worth while to mention here several other causes of difficulty with masonry dams on earth foundations which should be avoided. In many streams there is a constant "flow" of solid material composing the river bottom. If

the backwater from the dam prevents this material from continuing to come down, while the flow downstream from the dam continues to carry material away, a considerable lowering of the river bed often results. (Mullins* states that "a retrogression of the level in the river bed is a nearly invariable result of building a weir.") This

*Mullins Irrigation Manual, (Madras) 1890, p. 16.

may cause greatly increased scour at the downstream edge of the dam, and in the case of the Islam Weir was sufficient to cause a failure. A retrogression of as much as 8 feet has been observed on another dam.

In computing the uplift beneath the apron of dams where a hydraulic jump is formed on the apron, the fact that the downward pressure of the thin stream of water upstream from the jump is less than that below the jump, should be given consideration. The upward pressure on the bottom of the apron is influenced by the depth below the jump and the downward pressure on top of a portion of the apron depends upon the lesser depth above the jump. Failure to provide sufficient weight of apron to overcome this difference in addition to the normal upward pressure has resulted in failure in several cases.

In designing a dam on a porous foundation it is necessary to be sure that the hydraulic jump will be formed on the structure where it is expected. A case has occurred where a dam was placed at the top of a steep section of the river and the tailwater level was so low that the jump occurred below the downstream end of the apron. In a short time it scoured out the riprap and destroyed the structure.

In several structures difficulty has been experienced with aprons formed in layers, due to the tendency of the swiftly flowing water to get between them and "peel off" the upper ones. If possible, aprons should be cast in blocks rather than layers, but if layers are necessary they should be thoroughly bonded together.

Although the bed of a stream may be considerably higher near the banks than in the center, the downstream apron of the dam should be level all the way across. The construction of the dam is usually such that the discharge is spread nearly uniformly over the entire width of the river, and if the apron is higher at the ends the flow over the dam at the sides of the river, after passing off the apron tends to run along the downstream edge of the apron toward the middle of the river. If the slope in this direction is considerable, severe scour is likely to be set up. This result has been observed at several dams. Even if the downstream apron is level, it is advisable to plan the earth excavation below the dam in such a way that severe currents are not set up along the toe from the ends of the dam toward the main channel. In long, low, overflow dams difficulty has also

frequently been experienced from a tendency of the water to form a channel parallel to the dam upstream from the crest and undermine the upstream portion of the dam. To prevent this, groins are sometimes built extending upstream from the crest at intervals along it.

Conclusions

The ordinary method of analysis of a masonry dam on an earth foundation to secure safety against piping, which is usually ascribed to Bligh, is faulty in that it does not consider the greater probability of percolation along level or slightly sloping contacts between the dam and its foundation, than along vertical or steeply sloping contacts.

The flow net and electrical analogy methods are faulty in neglecting the lesser resistance along the line of contact of a dam with its foundation as compared with that directly through the foundation material.

Piping may occur in two distinct ways, (1) by flow along the line of contact of the structure and its foundation or (2) by flow directly through the foundation material. Flow ordinarily takes place along both of these paths in inverse proportion to their relative resistances.

Considerable light on the probability of failure from the first of these causes can be obtained by experimental methods, but the second must be studied largely by an analysis of the action of actual dams.

From the result of an analysis of the action of more than two hundred dams, it has been found that creep along contact surfaces having slopes with the horizontal of less than 45 degrees should be considered to offer only one-third the resistance to piping as those with slopes of 45 degrees or over.

Analysis on this basis may be called the weighted creep analysis, the creep along surfaces under 45 degree slopes called horizontal creep and that of 45 degrees or over vertical creep. It should be noted that these slopes are the slopes of the surface of contact. The slope of the path taken by the water may be different. The weighted creep distance of a dam cross section is the sum of the vertical creep distances plus one-third of the sum of the horizontal creep distances, and the weighted creep head ratio is the weighted creep divided by the effective head.

The following values are suggested as the safe weighted creep head ratios for use in the design of major structures:

SUGGESTED WEIGHTED CREEP RATIOS

WEIGHT OF HORIZONTAL CREEP $\frac{1}{3}$

Very fine sand or silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Coarse gravel including cobbles	3.0
Boulders with some cobbles and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay or hardpan	1.6

These values can only be used for solid masonry cutoffs built directly against the earth or interlocking steel or concrete sheet piling driven so that the interlock is not broken and with the top of the piling satisfactorily embedded in the masonry of the dam. Competent supervision during construction and efficient maintanance are assumed. If all these do not exist higher values must be used.

For less important structures these ratios may be reduced somewhat down to perhaps 80% of the values given for very minor structures.

Additional factors which should be considered in selecting the proper ratio are (1) the loss of life and property which would result from a failure, (2) the quality of supervision during construction and of maintanance after completion, (3) the loss resulting from percolation under a dam, (4) the difficulty in mending a breach if it occurs, and (5) the possibility of the river bed upstream silting up.

Reverse filters, weep holes and drains are an aid to security and weighted creep head ratios may be reduced up to 10% if they are used. The best form is a weep hole with a reverse filter behind it. For best results vents should be located far enough from the end of the travel path to insure that most of the flow passes through them. Except to reduce upward pressure, they should not be located farther upstream than necessary to accomplish this purpose. Usually the best location, from the standpoint of piping, is just above the downstream cutoff, venting through the cutoff. The position of the hydraulic jump should be considered in locating vents. Great care must be exercised in constructing vents or drains.

The security of dams on pervious foundations underlaid by impervious material will be greatly increased if a cutoff is carried into the impervious material. No satisfactory analysis for this condition has been developed. Dams founded on impervious material underlaid by pervious material are apt to give trouble and should be designed with care.

In all cases care must be exercised that cutoffs are properly tied in at the ends, so that the water will not outflank them, and that there is no short route behind or under the abutments through which a channel may be formed.

In order to prevent failure of dams by percolation directly through the foundation material the short path head ratios should not be less than eight-tenths of those recommended for the weighted creep.

If any two points on the creep line are so close together that the short path between them is less than half the weighted creep distance between them, flow may be considered to take place directly through the material, the length of this travel being given a weight of two. Where more than one short path is possible between the same portions of the creep path, the one which gives the smallest total weighted creep for the structure should be used.

The upward pressure to be used in design may be estimated by assuming that the drop in pressure from headwater to tailwater along the contact line of dam and foundation is proportional to the weighted creep distance. Between sheet pile lines which are so close together that the short path limitations apply the total pressure drop may be computed as proportional to the short path distance with its weight of two and distributed between the two ends of the short path in proportion to the weighted creep between those points.

Miscellaneous causes of difficulty or failure which can be avoided by proper design and construction are clay foundations or aprons which are located where they can dry out, construction on dry sand, control of springs during construction, apron blowups due to insufficient allowance for upward pressure, apron uplift due to the hydraulic jump, scour below the dam due to jump forming off the apron, peeling of apron due to construction in layers, retrogression of the river bed after construction, and scour below apron due to it not being level across the stream.

Acknowledgments

It would have been impossible for the author to have collected the amount of data analyzed in this paper without the hearty cooperation of a large number of members of this profession. The sources of data on the individual dams has been acknowledged in the Tables 1, 2,

3, 4 and 6. In a number of places in the text other acknowledgments were made. In addition to these the author wishes to express his indebtedness to the following:

To the Ambursen Dam Company for the data on a large number of Ambursen type dams, on earth foundations, with most of which that company was connected as designer or contractor: to Messrs. Ayres, Lewis, Norris and May, for data on dams designed by them or by the late Gardner S. Williams, M. Am. Soc. C. E.; to Mr. W. M. Griffith for many valuable suggestions and ideas and for data on several dams; to Messrs. Holland, Ackerman and Holland for data on a number of dams designed by them; to the Department of Public Works, Philippine Islands, for data on dams built by them, to the Verein Deutscher Ingenieure and the Deutsche Wasserwatschafts and Wasserkraftverband for assistance in securing information on failures in Germany; to the U. S. War Department and its various district offices for data on dams built by the U. S. Corps of Engineers; to Messrs. H. E. Babbitt, J. L. Burkholder, E. L. Chandler, C. H. Eiffert, B. A. Etcheverry, Ivan E. Houk, R. D. Goodrich, John H. Gregory, Ford Kurtz, R. B. McThorter, D. W. Mead, Wm. W. Tefft, Members, American Society of Civil Engineers, A. L. Olin, George R. Clemens, Associate Members, American Society of Civil Engineers, Messrs. P. Bier, G. P. F. Boese, H. N. Eaton, W. H. Frazer, Chas. G. Gablemen, C. C. Noher, Paul A. Nelson, A. D. Williams, and numerous others for assisting in securing data and for valuable suggestions. The author was assisted in preparing the data and drawings by Messrs. Walter H. Price, W. F. Bingham and Richard J. Willson.

This paper represents the result of a study made for the United States Bureau of Reclamation to assist in the design of an intake dam on the Colorado River for the All American Canal. All work of this Bureau is under the direction of Dr. Elwood Mead, Member, American Society of Civil Engineers, Commissioner of Reclamation. All Engineering work is in charge of R. F. Walter, Member, American Society of Civil Engineers, Chief Engineer. The study was carried out under the immediate supervision of J. L. Savage, Member, American Society of Civil Engineers, Chief Designing Engineer.

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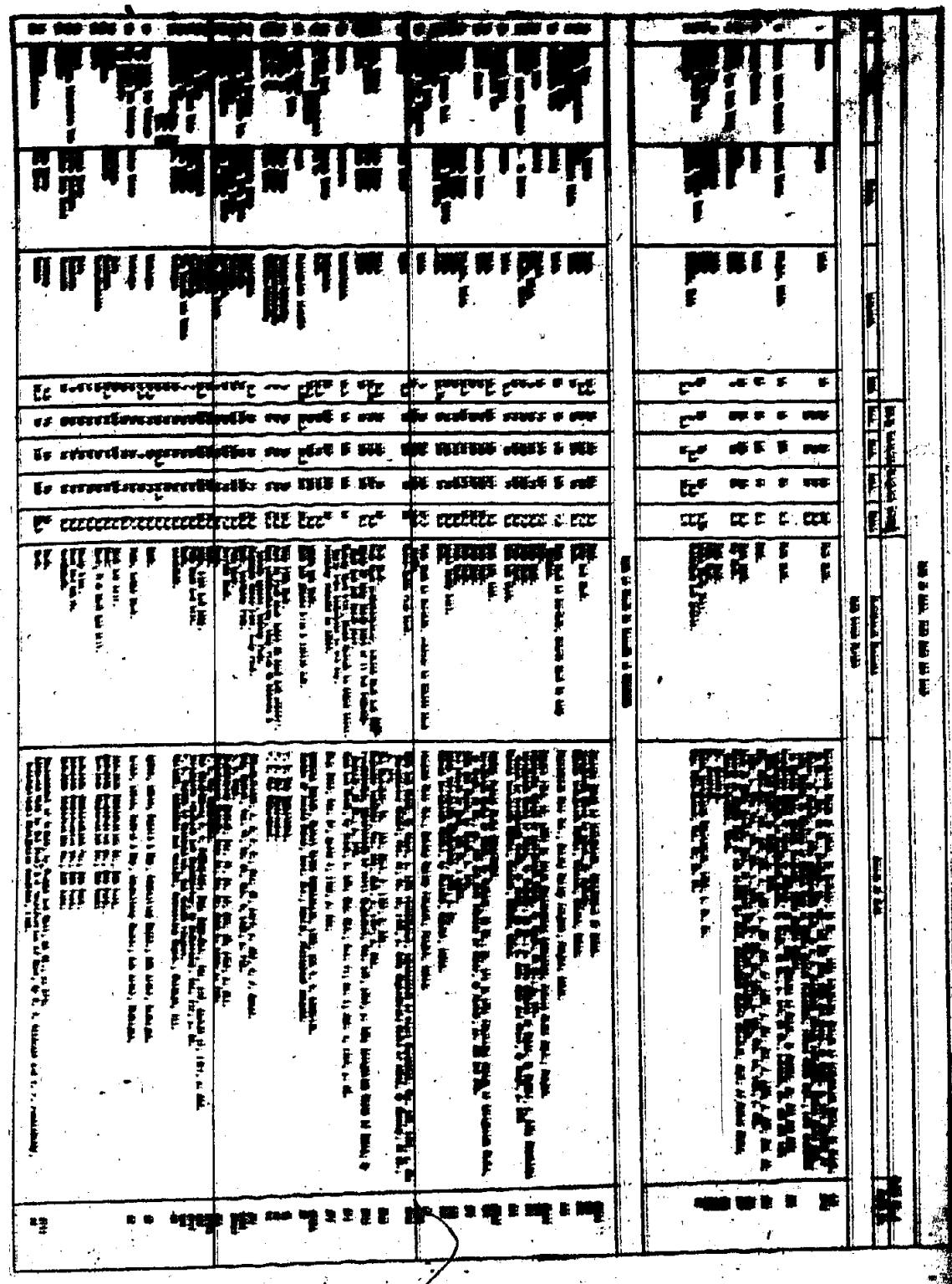

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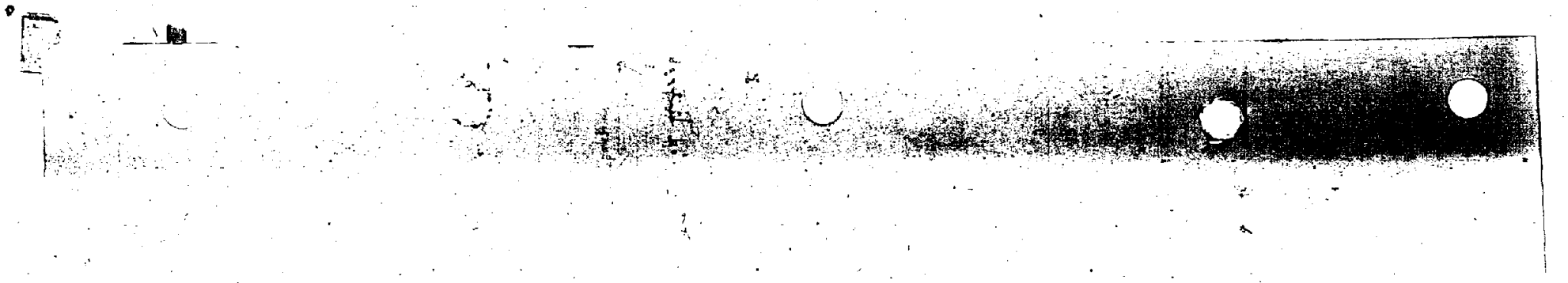

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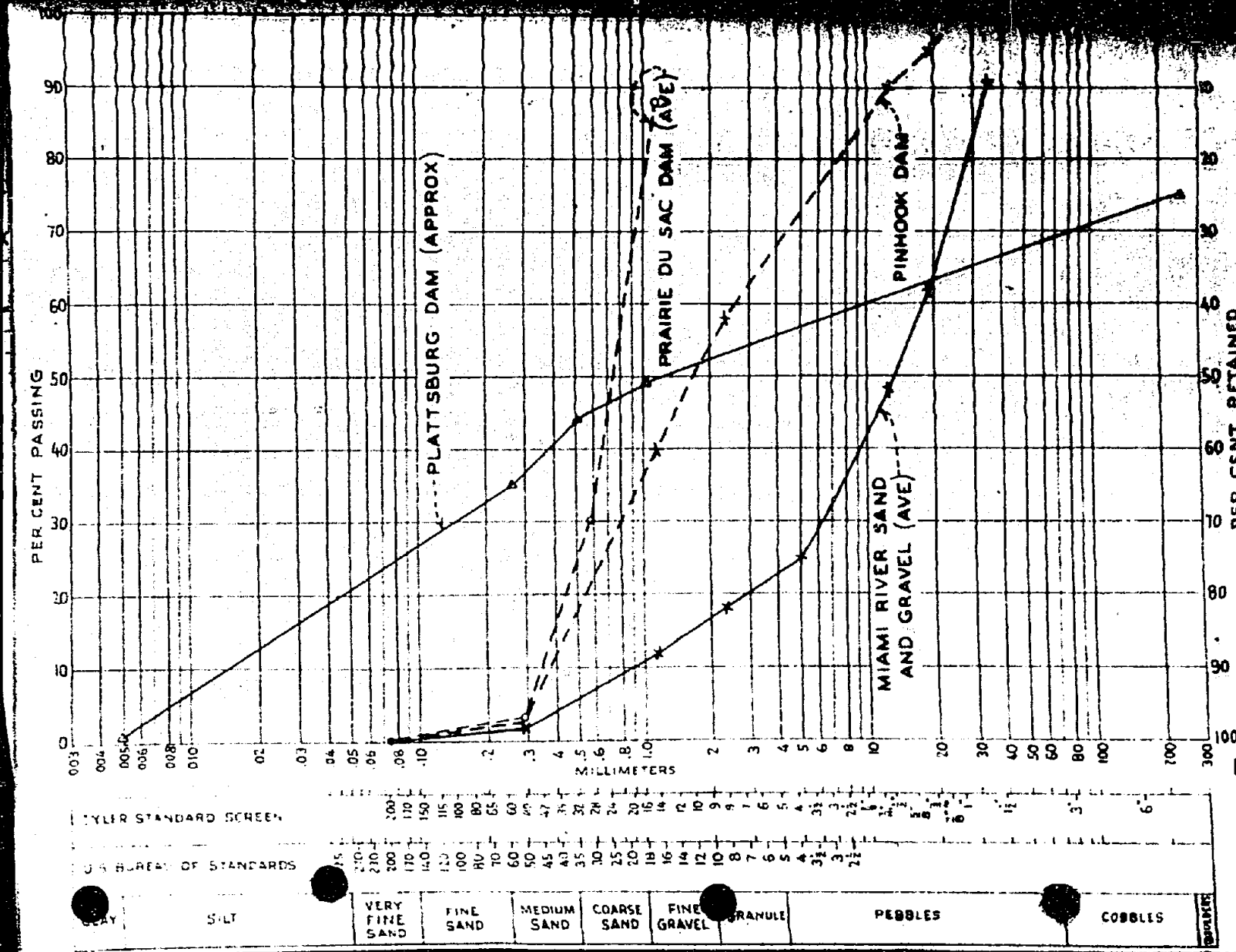
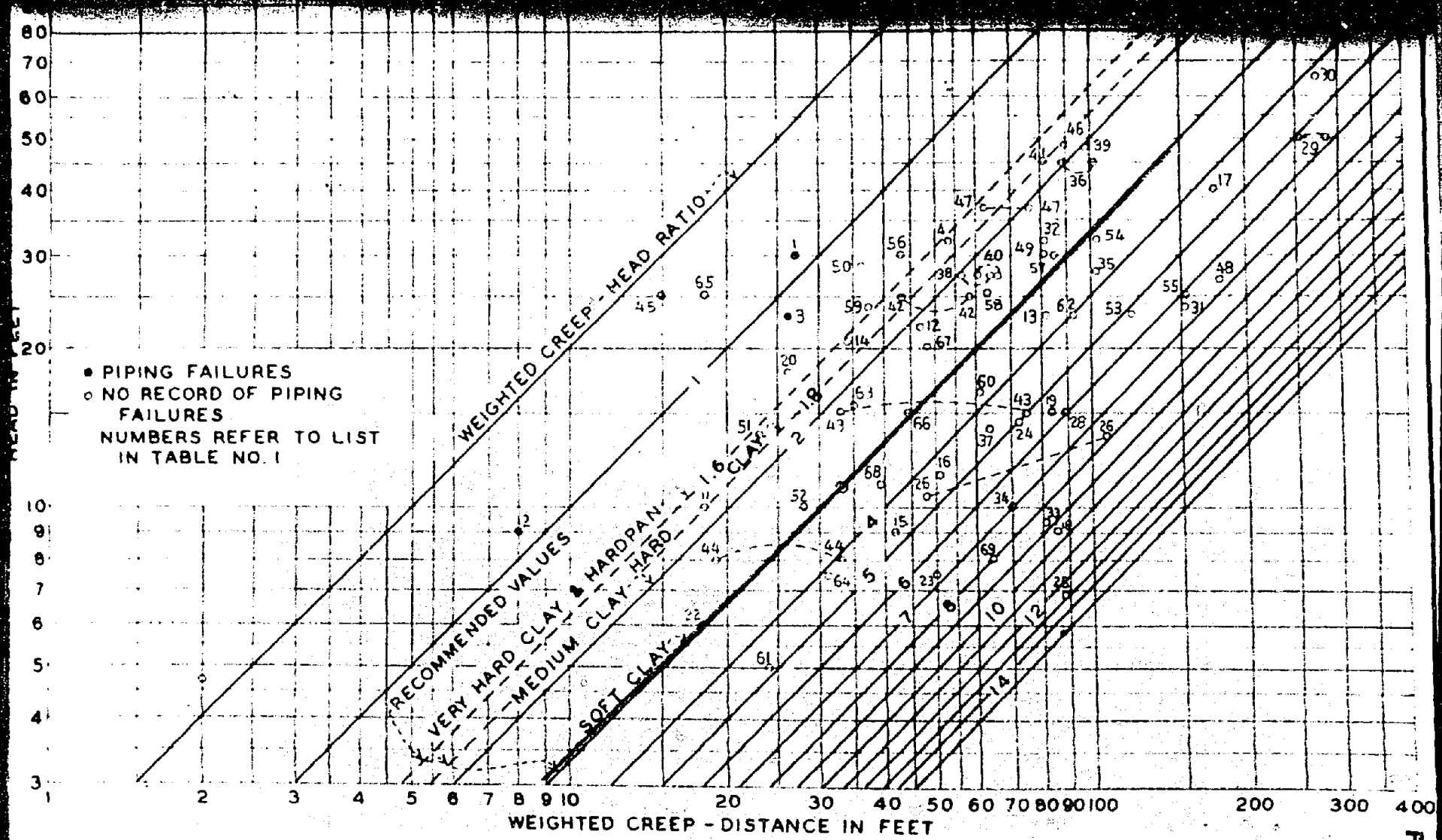


MECHANICAL ANALYSIS

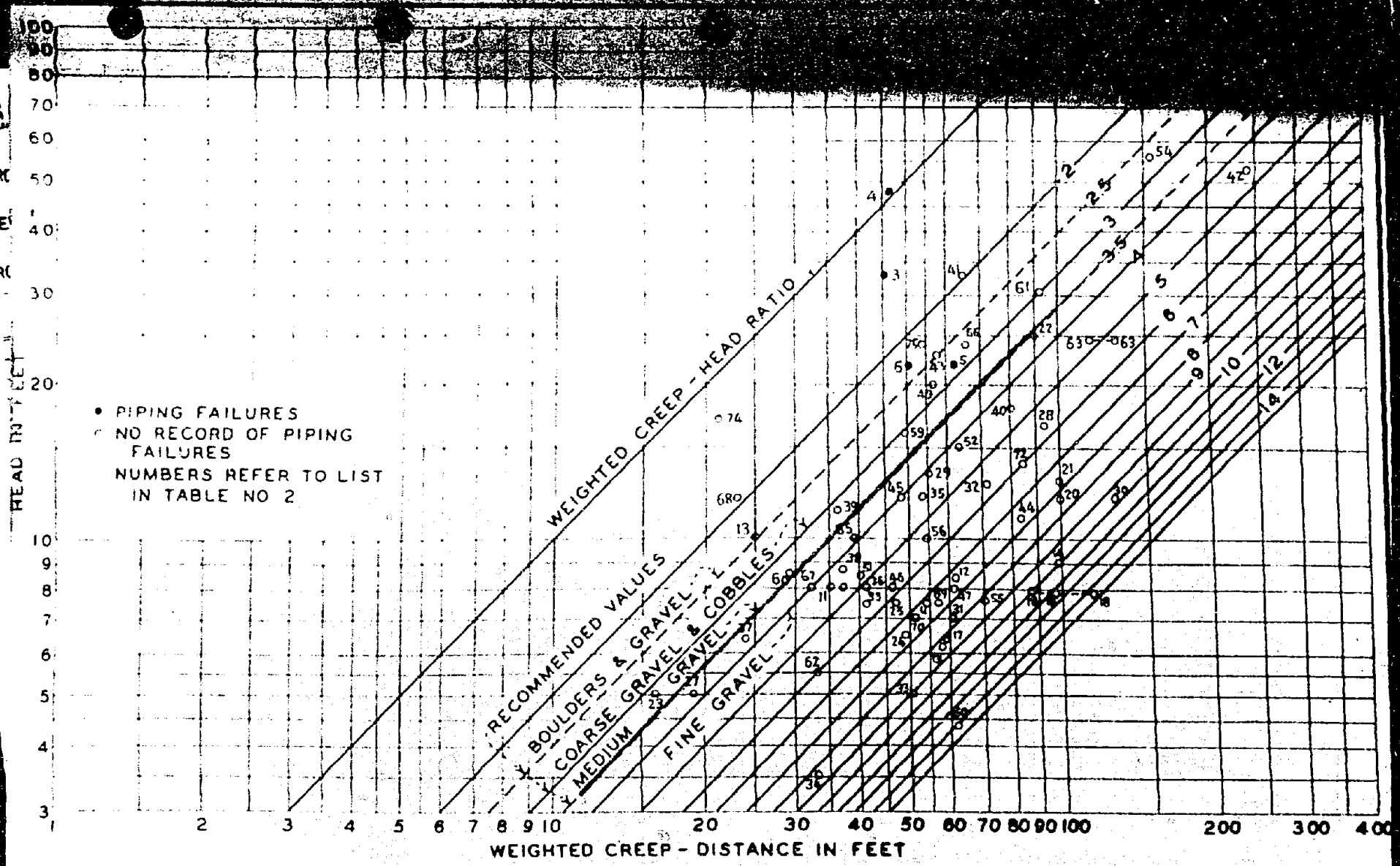
DAM FOUNDATION MATERIAL



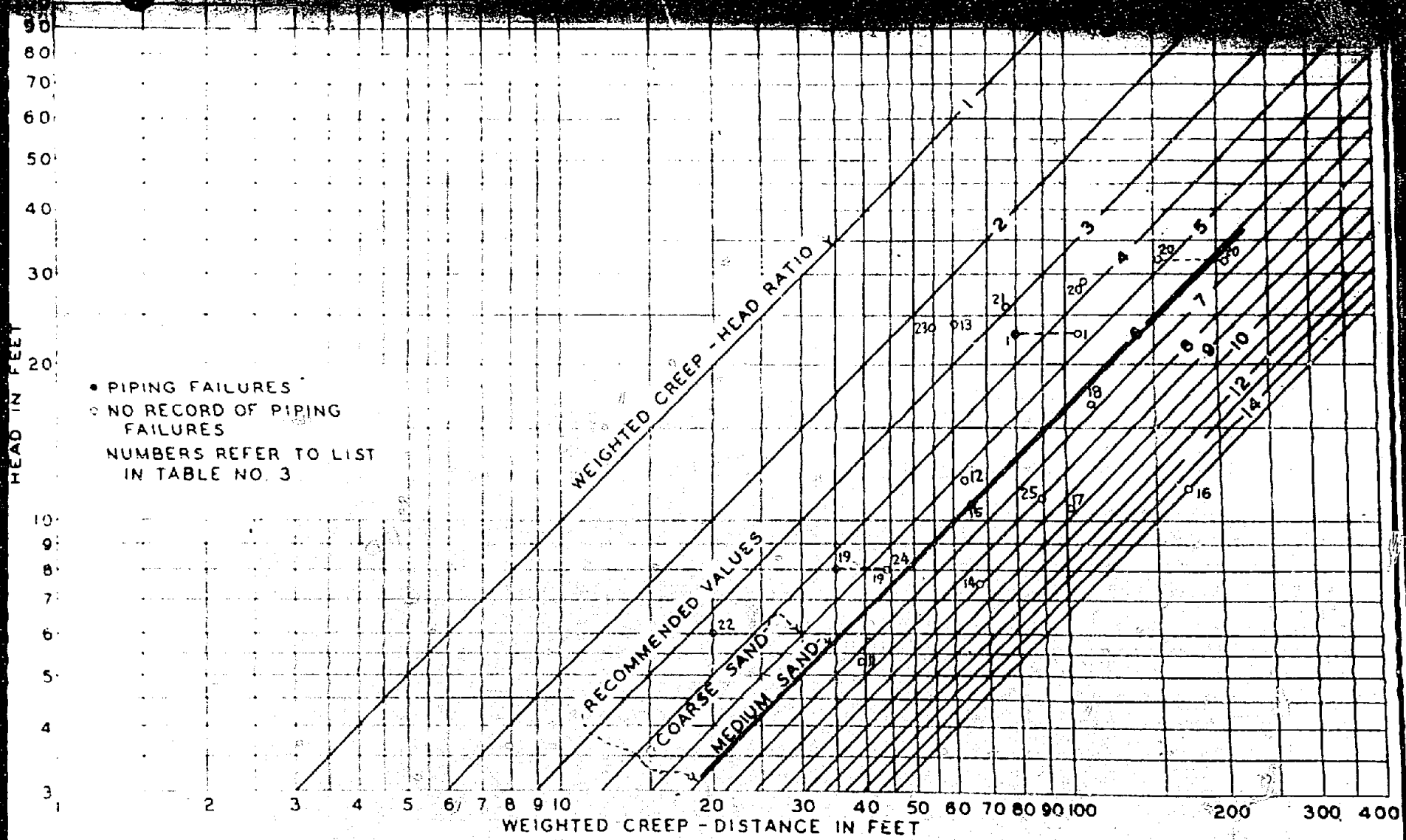
WEIGHTED CREEP - HEAD RELATIONS

DAMS ON CLAY AND HARDPAN

Fig. 2

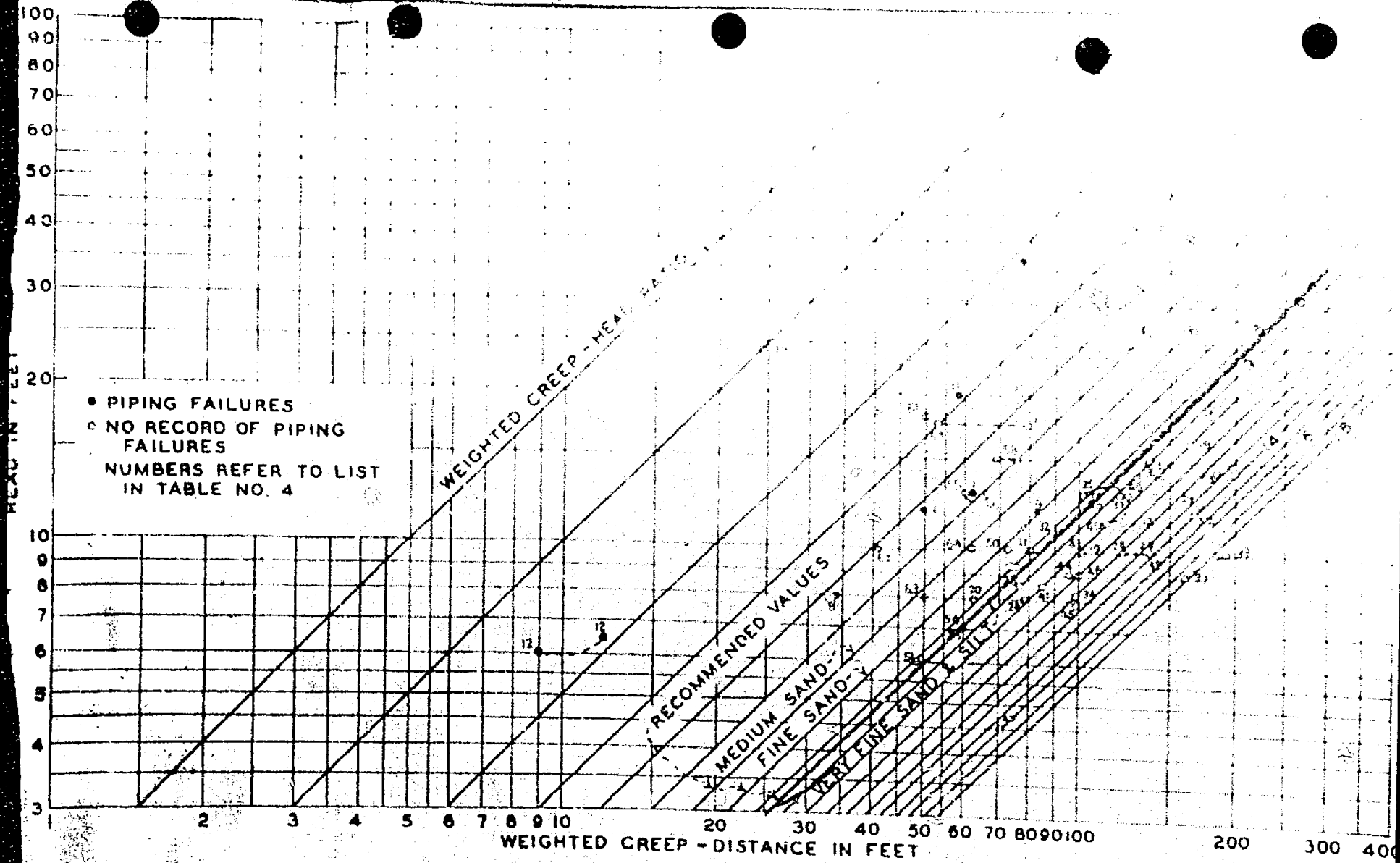


WEIGHTED CREEP - HEAD RELATIONS
DAMS ON GRAVEL, COBBLES AND BOULDERS



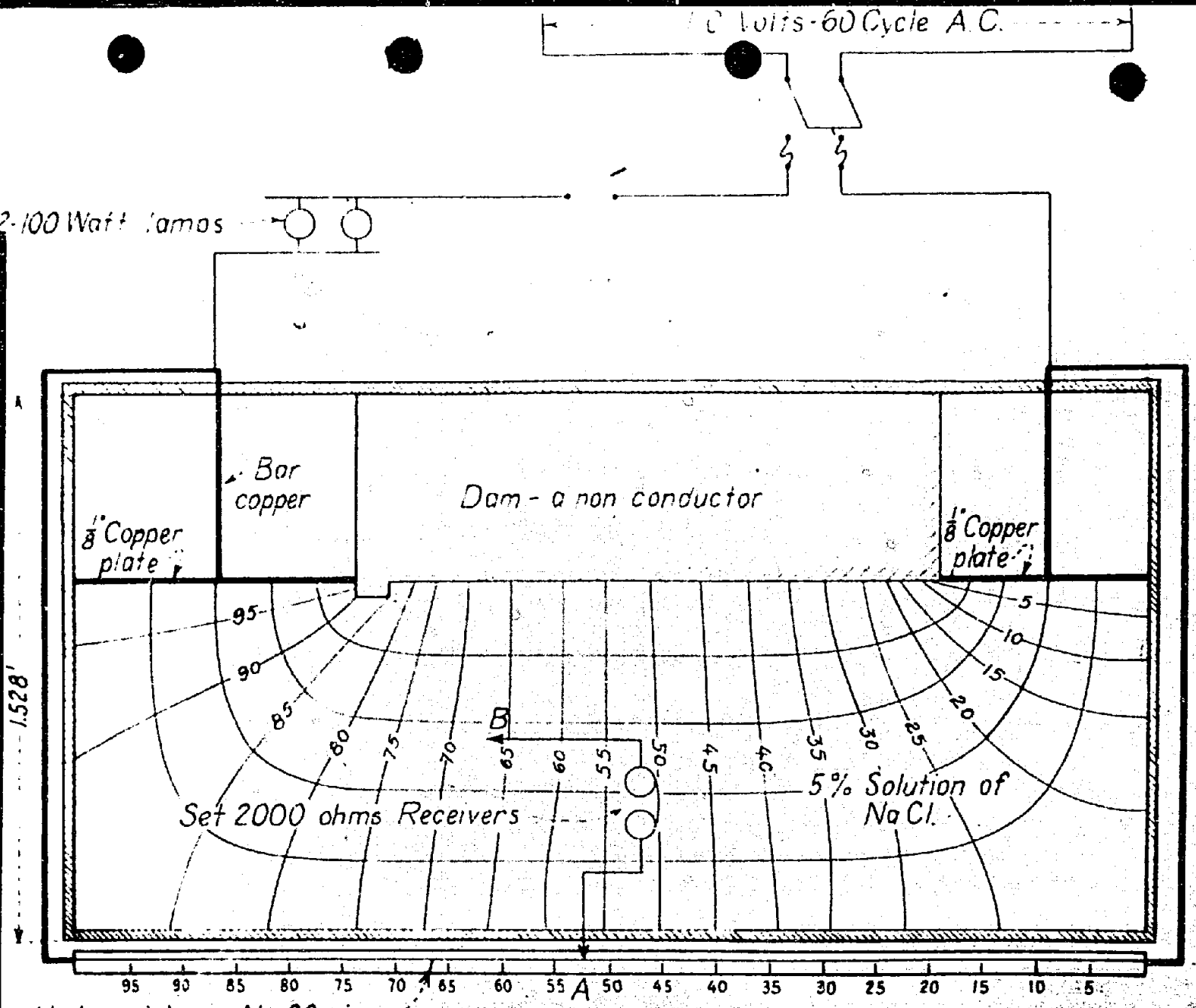
WEIGHTED CREEP - HEAD RELATIONS

DAMS ON COARSE SAND



WEIGHTED CREEP - HEAD RELATIONS

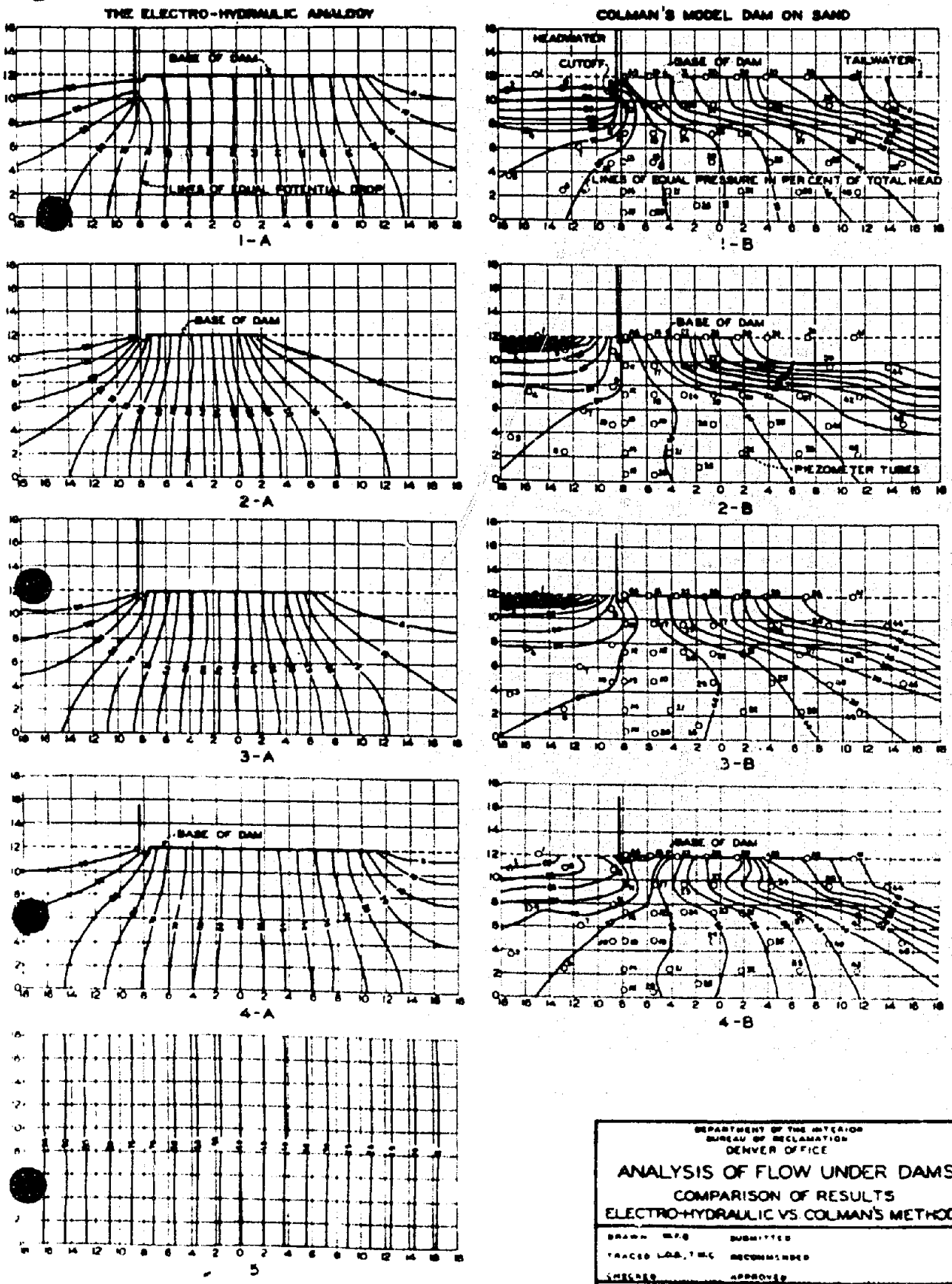
DAMS ON SAND FINE SAND AND SILT



ELECTRO-HYDRAULIC ANALOGY APPARATUS

Fig. 6

Fig. 7



DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE

ANALYSIS OF FLOW UNDER DAMS
COMPARISON OF RESULTS
ELECTRO-HYDRAULIC VS. COLMAN'S METHODS

DRAWN W.F.G. SUBMITTED
TRACED L.O.B., T.M.C. RECOMMENDED
CHECKED APPROVED
DENVER, COLO., JULY , 1938

212-D-181

APPENDIX I

BIBLIOGRAPHY

MASONRY DAMS ON EARTH FOUNDATIONS

APPENDIX I

See also - second Appendix I at back

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85 - 8-19-20 - p. 355 - Floods Wash Out Part of Apron of Granite Reef Dam.
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75 - 6-15-16 - p. 1113 - Building Power House and Dam on Sand Foundations.

74 - 11-25-15 - p. 1032 - $101,000 Hollow Concrete Dam, St. Francis River.

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72 - 8-27-14 - p. 425 - The Design and Construction of the Bassano Dam.

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71 - 2-5-14 - p. 300 - Construction Work for East St. Louis Flood Protection (Cahokia Creek Dam).

70 - 12-18-13 - p. 1258 - Reconstructing the Foundation of the Elwha River Dam.

69 - 1-9-13 - p. 83 - The Sheet Pile Cutoff on the Port Angeles Dam.

69 - 2-6-13 - p. 266 - Lessons from the Failure of a Weir and Sluices on Porous Foundations.

68 - 12-5-12 - p. 1072 - Foundation Failure of a High Concrete Dam - Port Angeles, Washington.

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65 - 1-26-11 - p. 109 - Sheet Piles as a Means of Decreasing Permeability of Porous Foundations.

64 - 12-8-10 - p. 611 - Movable Dams on the New York State Barge Canal.

64 - 12-29-10 - p. 708 - Dams, Barrages and Weirs on Porous Foundations. (See also Vol. 65, 1-12-11, p. 52.)

62 - 10-14-09 - p. 393 - The Failure of the Fergus Falls, Minnesota, City Dam.

62 - 11-4-09 - p. 497 - The Failure of the Dam at Fergus Falls, Minnesota.

61 - 1-7-09 - p. 1 - The Granite Reef Dam and Gate Mechanism.

61 - 4-1-09 - p. 345 - The Undermining of a Reinforced Concrete Dam at Pittsfield, Massachusetts.

61 - 6-10-09 - p. 615 - Turning the Colorado River and Completing the Laguna Dam, Arizona-Colorado.

60 - 10-1-08 - p. 366 - The Granite Reef Diversion Dam of the Salt River Project of the U. S. Reclamation Service, Arizona.

59 - 2-27-08 - p. 213 - The Construction of the Laguna Dam, Colorado River, Arizona.

59 - 4-30-08 - p. 491 - The Break in the Hauser Lake Dam, Montana.

58 - 11-14-07 - p. 507 - The Hauser Lake Steel Dam in the Missouri River near Helena, Montana.

54 - 10-12-05 - p. 374 - Water Power Plants on the Apple River, Wisconsin.

54 - - p. 100 - Pile Foundation for Movable Weir.

53 - 2-9-05 - p. 146 - The Laguna Dam, Yuma Irrigation Project, California.

53 - 2-9-05 - p. 147 - Grouted Rubble Core Walls for the Weirs of the Delta Barrage, Egypt.

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24 - 9-6-90 - p. 204 - Whiting St. Dam, Holyoke, Massachusetts.

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71 - 1-2-15 - p. 24 - Hindia Dam on Euphrates River.

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69 - 3-28-14 - p. 372 - Reconstruction of the Elwha River Dam.

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66 - 11-30-12 - p. 600 - Washout of Base of Port Angeles Dam.

64 - 10-21-11 - p. 481 - A Small Hydroelectric Plant on the Oswegatchie River.

63 - 5-20-11 - p. 560 - Granite Reef Diversion Dam, Arizona.

62 - 10-8-10 - p. 396 - The Las Vegas Irrigation Project, New Mexico.

61 - 4-9-10 - p. 487 - Failure of Canal Headworks, Egypt.

61 - 1-1-10 - p. 24 - The Failure of a Small Concrete Dam.

61 - 6-4-10 - p. 726 - Compensation Weirs, Nepean River.

58 - 11-21-08 - p. 572 - The Hydro-Electric Plant of the Uncas Power Company at Scotland, Connecticut.

58 - 12-5-08 - p. 645 - A Successful Low Timber Dam for Broad, Sandy River Channels.

58 - 12-26-08 - p. 728 - A Low Head Hydro-Electric Development at Berrien Springs, Michigan.

56 - 7-20-07 - p. 75 - The Upper Grays Dam, Consolidated Water Company, Utica, New York.

56 - 10-19-07 - p. 418 - Three Low Head Hydro-Electric Developmentsin Michigan.

56 - 10-26-07 - p. 462 - Three Low Head Hydro-Electric Developments in Michigan.

52 - 10-14-05 - p. 431 - The Riverdale Plant of the Apple River Power Company.

51 - 4-15-05 - p. 428 - The Asyut Barrage Across the Nile.

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41 - 5-20-14 - p. 584 - Structural Features and Construction Methods, Hydro-Electric Plant at Elkhart, Indiana.

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39 - 5-7-13 - p. 533 - Methods of Constructing a Hydro-Electric Power House and Dam on Sand Foundations.

37 - 6-5-12 - p. 639 - The Design and Methods Employed in Constructing the Cooke Water Power Plant on the Au Sable River, Michigan.

37 - 6-5-12 - p. 646 - Three Approved Designs for Weir Dams for Irrigation Canals.

35 - 3-8-11 - p. 290 - Dams on Sand Foundations.

33 - 2-2-10 - p. 117 - What Caused the Failure of the Dansville Dam?

33 - 3-2-10 - p. 207 - The Dam Failure at Dansville, New York.

25 - 6-6-06 - p. 155 - Cost of a Small Reinforced Concrete Dam at Richmond, Indiana.

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APPENDIX II

ADDITIONAL DATA ON DAMS AND DAM FAILURES

APPENDIX II.

Data on Dams and Dam Failures

In order to make available more detailed data than is possible by means of tables for some of the structures which have an important bearing on the percolation distances required for various foundation materials, the following brief statements have been prepared:

Dams on Clay and Hardpan

Woodward Dam at Hill, New Hampshire

This dam was founded on material so hard that if an iron bar was driven into it 12 to 18 inches, it was sometimes necessary to use a derrick to pull it out. The adjacent banks were of a hard cemented sand and clayey material with occasional layers of pure sand. The foundations were said to have been excavated 10 to 12 feet below the natural surface of the earth. The line of creep and the short path were both about 32 feet, which, with the 30 foot head, give ratios of length of path to head of 1.1. A large proportion of the creep on this dam was vertical. The weighted creep head ratio was 0.9.

This dam failed after five years of service with no sign of failure. Where dams fail after a considerable period of satisfactory service with heads that have been carried before satisfactorily, the probable cause of failure is piping along the line of contact of the dam with its foundation. When failure takes place suddenly, at a head which has not been held before, or with a path which is suddenly shortened by an apron blowup, the cause may be percolation directly thru the foundation material. As this dam failed after a long service apparently at no unusual head, the failure was probably due to percolation along the contact of the masonry with the earth. The layers of pure sand may have been responsible for the failure.

Dolgarrog Dam, Lake Eigiau, North Wales.

The Dolgarrog Dam was founded on a glacial deposit of hard blue clay containing boulders, overlaid with a layer of peat. Part of the foundation was not carried as deep as planned. This section failed by a blowout under the foundation after the dam had been in service fourteen years. The reservoir had previously been drawn down below the dam foundation due to prolonged dry weather and the drying out of the clay may have been the real cause of failure. The portion of the dam built according to plan did not fail. The creep distance for the latter portion

was 39 feet and for the part which failed 33 feet which for the 10 foot head gives creep head ratios of 3.9 and 3.3 respectively. The corresponding weighted creep ratios are 1.4 and 1.2 respectively. As the dam held fourteen years with the lower ratio this should be nearly, if not entirely safe.

Fergus Falls, Red River, Minnesota

The Fergus Falls Dam was founded on hardpan. It failed after eleven months service by a blowout under the power house. It appears that a spring was encountered in the power house foundation and no outlet was provided for it. This spring probably forced an outlet for itself and gradually enlarged it. The path from the headwater to the spring was too short and piping occurred from the headwater to the spring and thence along the spring outlet channel. There is no evidence to show that the spillway had insufficient length of path. The creep distance for the spillway was 104 feet, giving a plain creep ratio of 4.5 and a weighted creep ratio of 3.6. The creep distance to the spring was 45 feet, giving a plain creep ratio of 2.0 and a weighted creep ratio of 1.1.

Dams on Gravel, Cobbles and Boulders

The data on dams on gravel, cobbles and boulders is given in Tables 3 and 4. The results are shown graphically on Figure 3. Dams on foundations composed largely of sand but with considerable proportions of gravel are included in this group. They represent an intermediate condition between coarse sand and gravel, and to include them in this classification is on the side of conservatism.

To set forth more fully the basis for the lengths of percolation path required for various materials, the dams having most influence on the conclusions will be described at some length.

Puentes Dam, Guadalantin R., Spain

The Puentes Dam was built in 1785-1791. Most of it was on rock but in the center of the valley it was founded on piles in earth variously described as "earth," "alluvial soil and sand" and "sand and gravel at least 25 feet deep," the latter classification being assumed as being the most conservative. The dam was 151 feet wide on the base and had a masonry apron 7.3 feet thick extending 131 feet further downstream. Nearly all the creep was horizontal. The failure occurred suddenly after 11 years of use. The pressure beneath lifted the apron at the toe of the dam causing it to fail and thus greatly reducing the creep distance,

resulting in a blowout. The reservoir was said to be silted 44 feet above the base of the dam and at the time of failure the head was 143 feet. It had reached 75 feet head in previous years without failure.

Considering the effect of the silt, the line of creep to the end of the apron was 339 feet. With the head of 143 feet at time of failure this gave a creep head ratio of 2.37 and a weighted creep ratio of 1.0. No signs of failure at the end of the apron appeared for this ratio, but might have, had these conditions continued. The blowout reduced the creep distance to 200 feet, the plain creep ratio to 1.40 and the weighted creep ratio to.67. In a previous year with 75 feet head, the respective ratios were 4.5 and 2.0 assuming the same silt conditions at the time of failure, although it is possible that they were less. This indicates that the dam was safe with a 2.0 weighted creep head ratio, possibly safe with a 1.0 weighted ratio, and certainly unsafe with .67 weighted creep ratio.

Port Angeles Dam, Elwha R., Washington

The base of the Port Angeles Dam rested upon a "glacial deposit consisting principally of gravel and small grit or coarse sand." It was 97 feet wide on the base below which a caisson had been sunk 11 feet. An earth fill was constructed on the upstream side with a height of 50 feet and base of about 100 feet. A row of sheet piling 30 to 40 feet long was driven 8 feet downstream from the toe of the dam, but not connected to it. This piling was detrimental, as it tended to concentrate the underseepage into the space between the dam and the piling. The water had been at about spillway level for nine days when signs of increased pressure were noticed below the dam and within two or three hours the entire bottom gave way.

The creep distance was about 169 feet. The head was uncertain, probably 79 feet, but may have been higher. The plain creep head ratio was therefore not more than 2.1. The weighted creep ratio was 1.3. The shortest path was 150 feet and gave a short path ratio of 1.9.

The dam was repaired by filling the canyon with rock blasted from the side hills, reinforced with willow mats, sheet piling and a large concrete block. The short path distance is about 650 to 700 feet and the head practically 100 feet, making a short path ratio of 6.5 to 7.0. Under these conditions it has been in satisfactory service since 1913. On account of the lack of a definite contact between the fill and the original material, no creep distance can be estimated.

Pittsfield Dam, West Brook, Massachusetts

The Pittsfield Dam was founded on an upper layer of gravel and fine material 4 to 5 feet deep, underlaid by a much finer and apparently impervious stratum of clayey material 5 to 6 feet thick, beneath which was a waterbearing gravelly mass containing much fine material with boulders from hens-egg size to possibly a cubic yard in volume. The upstream cutoff wall extended into the impervious layer, but not down through the pervious material beneath. The dam seems to have been designed on the assumption that the foundation material below the top layer was impervious. Apparently the flow thru the pervious material appeared downstream, either lifting the impervious layer or causing piping along some channel leading thru it. The failure was probably more in the nature of piping through the material rather than along the line of contact with the dam. The shortest path was 63 feet and the head 33 feet, giving a short path ratio of 1.9. The plain creep distance and ratio were 76 feet and 2.3 respectively. The weighted creep ratio was 1.4. The dam was repaired with a cutoff wall extending through the pervious layer and has been giving satisfactory service for about 20 years.

Coon Rapids Dam, Mississippi River, Minnesota

The Coon Rapids Dam on the upper Mississippi River failed from a sudden blowout during repairs, after thedam had been giving satisfactory service (from the standpoint of piping) for over three years. The primary cause was a gap one pile wide in the sheet pile cutoff. The hole below the dam caused by scour was cut to build an extension of the apron, causing an increased head on the structure. The head was approximately 48 feet, and the shortest path 130 feet, giving a short path ratio of 2.7. Since the water level below the dam at this time was considerably below the bottom of the downstream piling, the water did not follow the creep line and therefore the creep head ratio cannot be estimated.

The earth below the dam had a sloping surface and would therefore fail more easily than if the ground were level. On the other hand the gap of only one pile would considerably restrict the flow as compared with the ordinary conditions assumed in design.

Plattsburg Dam, West Brook Plattsburg, New York

The Plattsburg Dam was founded on a glacial material of the following composition: 25% boulders, 26% fine gravel, 1% very coarse sand, 2% coarse sand, 4% medium coarse sand, 4% medium sand, 8% medium fine sand, 1% fine sand and 29% sand and silt passing a 100-mesh screen. There was less than 1% clayey material. The base of the dam was stepped up the sides of the hills. The upstream cutoff wall

was said to be 20 feet deep. It is reported to have been built with wood sheeting left between the concrete and the natural earth and that the backfill was made with excavated material and construction debris. The weep holes planned for the structure were plugged. It is also reported that a trench was excavated along the upstream side of the cutoff and refilled with puddle, but this report could not be confirmed. Leakage was evident from the first, and the dam failed from a blowout, probably from seepage along the line of creep, apparently about two months after it was filled.

The length of the line of creep varied for different sections of the dam and the creep head ratios varied from 7.1 to 3.2, the higher ratios being for the lower parts of the dam where it was stepped up the side of the valley. The corresponding weighted creep ratios were 5.1 to 2.0. The shortes t path ratios varied from 5.8 to 2.6. Where the failure took place the head was probably approximately 22' and the creep 87', giving a plain creep ratio of 4.0 and a weighted creep ratio of 2.8. The shortest path at this point was probably 70', giving a shortest path ratio of 3.2.

Pinhook Dam, Maquokota River, Iowa

This dam and power house are founded on what is reported to be "fairly well graded sand, altho there is considerable gravel and occasional boulders as large as 1/3 cu. yd. Another description gives it as "fine sand with sand silt intermixed and almost the nature of quicksand." (A sieve analysis is given on Figure 1.) The head is 25' and the creep distance to the end of the fixed apron below both the power house and spillway is 130', giving a plain creep head ratio of 5.2 and a weighted creep head ratio of 3.5. Below this fixed portion however, there are concrete slabs constructed to allow settlement without cracking, but without watertight joints. Since these slabs no doubt to some extent increase the resistance to piping, it would not be correct to assume that the success of this dam indicated that the 3.5 ratio was safe. The creep distances to the end of this slab apron for the power house and the spillway section are 206' and 231' respectively, which give plain creep head ratios of 8.0 and 9.2 and weighted ratios of 4.5 to 5.9. The shortest paths, not including the block apron are 92' and 91' for the power house and spillway respectively, giving ratios of 3.2 and 3.1. Including the block apron these distances are 161' and 179' and the ratios 5.6 and 6.2 respectively. Some trouble was experienced from sand being carried thru the cracks of the pavement, probably due to the fact that the flexible paving partially cutoff the outlet for the water at the downstream edge of the solid apron and considerable flows escaped thru the cracks between the blocks. The difficulty was remedied by dumping clay on the upstream side of the dam.

Dam on Durrence River, France

This dam failed from blowout thru the central portion. It was founded on gravel. The data is not very complete, but there was apparently considerable erosion downstream before the blowout occurred.

Oswegatchie River, New York

This dam, for part of its length, is reported as being founded on "glacial boulders" but as light sheet piling was driven, the boulders were more than likely combined with sand or gravel. The head was 23 feet. The cutoffs were formed by concrete walls with 2 x 6 inch sheet piling on each side of the wall driven down 4 to 7 feet beyond the bottom of the concrete. Except for springs encountered during construction, there was no seepage and the cutoff walls showed no seepage. The piling is assumed water tight and that the resistance to travel between the two adjacent rows is equivalent to a creep twice the distance between the piles. The creep distance is 82 feet and the head 23 feet., making the plain creep ratio 3.6. The weighted creep ratio is 2.5. The shortest path is 57 feet, giving a short path ratio of 2.5.

Granite Reef Dam, Salt River, Arizona

The Granite Reef Dam is founded on gravel and boulders and sustains a head of 20feet. The total creep distance to the porous apron is 77feet, giving a plain creep head ratio of 3.8 and a weighted creep head ratio of 2.8. The shortest path to the upstream edge of the porous apron is 58 feet, giving a short path ratio of 2.9, and the short path to the weepholes is 47, giving a ratio of 2.3.

When first put into operation there was a seepage of two to three sec. ft. under the 300 feet length not founded on rock, but the pool above the dam filled with sand and silt, which reduced or stopped the seepage. It is possible that the rock fill under the downstream apron acted somewhat as a reverse filter and increased the security of the structure. After some years in service the apron was destroyed by a flood, and the dam stood some time without it before it was repaired, showing that at that time it was not necessary for security.

Dams on Coarse Sand

Riverdale Dam, Apple River, Wisconsin

This dam was founded on torpedo sand, and held a head of 24 feet. It was completed in 1905 and was still apparently in service in 1909. The maximum line of creep is 101 feet, but this includes 20 feet for sheet piling composed of steel rails driven as close together as possible, which would probably not be watertight, and the probable creep was therefore 81 feet, the plain creep head ratio 3.4 and the weighted ratio 1.7. Considering the rails to have made an effective cutoff, the plain creep ratio would be 4.2 and the weighted ratio 2.5. This value will be used as it is more conservative. The shortest path is 71 feet, and the short path ratio 3.0. Downstream from the dam is an apron of large stones which may have acted to some extent as a filter. The foundation was full of springs during construction.

Prairie du Sac Dam and Power House, Wisconsin River, Wisconsin

This dam was built about 1912 and is founded on pure coarse sand. It is hollow concrete, the buttresses are supported by piles and there is no floor under the dam. Seepage is prevented by a 50' row of sheet piling and an upstream clay blanket about 20 feet wide. The inside of the dam is filled with loose rock, through which any water passing under the sheet piling may rise and until the vents were closed escaped thru them in the downstream side of the dam. Water may also seep out thru the sand and under the downstream sheet pile cutoff. Since the plain creep ratio from the inside of the dam to tailwater is much greater than between the headwater and the vent, the underflow undoubtedly escaped through the vents, until they were closed. The head from headwater to vents is 29 feet, and the total ordinary head 32 feet. The creep distance to the inside of the dam, where it formerly could discharge thru the vents, was 124 feet. With the vents closed, the total creep distance is 198 feet. The creep head ratio to the inside of the dam, with the vents open was therefore 4.3 and with the vents closed the ratio for the entire structure was 6.2. The corresponding weighted creep ratios are 3.7 and 4.8. The shortest path with the vents open is 104 feet, giving a ratio of 3.6, and with vents closed 149 feet, with ratio of 4.7. The creep distance under the power house is 301 feet, which, with a head of 32 feet, gives a plain creep ratio of 9.4 and weighted creep ratio of 6.5. The shortest path is 216 feet and short path ratio 6.8.

Dams on Sand, Fine Sand and Silt

Narora Weir, Ganges River, India

This weir was founded on fine micaceous sand, probably falling into Bligh's class "of which 80% of the grains pass a 75-mesh sieve." It sustained a head of about 13 feet. After about 20 years of service it failed and was repaired with a much longer creep distance.

This failure has occupied a large place in discussions of the design of dams on porous foundations, and therefore has been studies in considerable detail. Bligh contended that it was a case of piping, which had been going on for twenty years, and on the basis of this failure, and that of the Khanki weir, built the structure of his widely used methods. A thorough study of the data leads to considerable doubt as to the accuracy of some of Bligh's conclusions. There are many differing opinions on the exact manner in which the failure took place. In Bligh's Practical Design of Irrigation Works, 3rd Edition, Mr. F. W. Woods records several opinions, and Griffith has still another. Woods contends at length, apparently with considerable justification, that the failure was primarily due to undercutting of the upstream portion of the weir by a lateral current and that this occurred in the river channel section where it was built on a fill.

For the purpose of this paper it is not necessary to determine exactly how the structure failed. Certain facts seem to be well established. The greater portion of the structure which was not built on a fill stood without failure for twenty years, although some trouble with springs at the downstream edge seems to have been experienced. This portion had a creep head ratio of 13.0 and a weighted creep ratio of 6.3. The upstream puddle apron in the section where failure occurred had been washed away. In some portions of the weir, springs of considerable magnitude came up through the upstream edge of the grouted riprap. The spaces between the wells forming the downstream cutoff were said to be "indifferently closed" by piles.

Bligh apparently was not in possession of all these facts when he reached his conclusions. He assumed that the line of creep followed the line A C D E F G H I J K L M N. (Plate XIV) This gives a length of approximately 169 feet, and a creep head ratio of 13.0. The upstream puddle is known to have been washed away, and this reduced the creep ratio to 11.0. However it is

very likely from the accounts of the failure that the seepage path ended in springs at the upstream edge of the grouted pitching. If this is the case, the ratio would be still further reduced to 7.3. If, in addition to this, the creep path passed through the indifferent staunching between the wells of the downstream cutoff, the ratio would reach the low value of 5.4. It is apparent therefore that Bligh's conclusion that the failure occurred with a ratio of 13.0 is incorrect, and if the dam failed from piping at all it occurred with a ratio of not more than 11.0, probably with 7.3 and perhaps as low as 5.4. The corresponding weighted creep head ratios are 6.0, 4.8 and 2.8 respectively. The data for the distances and ratios for plain creep, weighted creep and short path are as follows:

	Plain Creep		Weighted Creep		Short Path		
	Path	Dist.	Ratio	Dist.	Ratio	Path	Dist.	Ratio
Conditions Assumed by Bligh	A,C,D,E,F,G,H,I,J,K,L,M,N	169	13.0	82.3	6.3	A,D,E,H,I,M,N.	135	10.4
Puddle Apron Missing	B,C,D,E,F,G,H,I,J,K,L,M.N	143	11.0	77	6.0	B,C,D,H,I,M,N	113	8.7
Exit of Percolation at J	B,C,D,E,F,G,H,I,J,P	95	7.3	61.7	4.8	B,C,D,H,I,J,P	80	6.2
Puddle Missing Exit at J and Percolation Between Wells	B,C,D,E,F,G,J,P	70	5.4	36.7	2.8	B,C,D,E,G,J,P	67	5.2
Reconstructed Weir	- - -	201	15.4	106.3	8.2	- - -	158	12.2

Khanki (Lower Chenab) Weir, Chenab River, Punjab, India

This weir was founded on fine sand and failed about three years after construction. When it was repaired, the apron was extended and sheet piles were added. The data on creep distance from various sources differs considerably and the values used are therefore somewhat arbitrarily selected.

Bligh's method of design and his constants were based principally on the results with this structure and the Narora Weir. There is unfortunately considerable uncertainty regarding the

cause of failure. Bligh contended that it was from piping but Woods argues very convincingly that it was due to subsidence resulting from undermining on the upstream side by a lateral current, that this occurred where the dam was built on a fill and that the weir was safe where not undermined. Buckley however states that there were bad springs below all bays of the weir before it failed, and since failure occurred only three years after construction, it is reasonable to conclude that the original design was not very secure, even if the failure did not result from piping.

The original design included very little vertical creep. It was repaired by extending the upstream apron and adding a cutoff of wells or sheet piling, the nature of the repairs differing at different parts of the weir. Bligh gives the head as 13 feet. Buckley states that the maximum head may have been 12 feet, and the average 8 to 9 feet. In the following table is given a summary of the analysis of plain creep, short path and weighted creep for both 12 and 13-foot heads. The 12-foot head is adopted as being more conservative.

Before Failure

	Bligh Creep		Weighted Creep		Short Path	
	Dist.	Ratio	Dist.	Ratio	Dist.	Ratio
Using 13-foot head	119	9.2	48	3.7	110	8.5
Using 12-foot head	119	9.9	48	4.0	110	9.2

After Reconstruction

	Bligh Creep		Weighted Creep		Short Path	
	Dist.	Ratio	Dist.	Ratio	Dist.	Ratio
Using 13-foot head	209	16.1	101	7.8	188	14.5

Deoha Barrage, Deoha River - United Provinces, India

This barrage was constructed on fine sand, with a creep head ratio of 17, the entire travel being along the base of the structure, as there was no sheet piling. It was put into service in July 1925. Trouble was experienced with cracking in 1925-26. In 1927 some evidences of piping and subsidence were observed and strong springs with sand discharging down-

stream from the canal headgate developed during a flood but ceased as the flood receded. On account of danger of undermining at the downstream edge of the apron in 1928-29 steel sheet piling 15 feet long was driven, but difficulty was experienced in getting a continuous cutoff properly tied into the floor. Thickening of the floor on account of the increased upward pressure was found to be unnecessary. Interlocking steel sheet piling 20 feet long was also driven upstream from the headgate and no more trouble appeared from the headgate springs. A line of interlocking steel sheet piling, 25 feet long, was started along the upstream edge of the floor.

In September 1929, when it was necessary to carry the full 12 foot head, springs suddenly appeared downstream of the sheet piling in the first bay, cracks soon developed and the floor was slightly raised. A distinct channel formed leading from the end of the upstream line of sheet piling at one abutment to the downstream row of piling, thru which the water escaped. Portions of the floor were undermined and one portion upstream of the gates was forced down by the weight of the water over it, and a section downstream was blown up.

The length of the creep path was more than 320 feet, giving a creep head ratio of at least 26.6. No appreciable part of this path was along a vertical or inclined surface. This distance is so extreme that it is reasonable to assume that the channels along at least a portion of the path had started to develop before sheet piling was driven. As the length of the path which developed under the various conditions is unknown the structure at the time failure occurred cannot be analyzed in a quantitive way but it does serve to bring out several important points. It is evident that the original creep distance giving creep head a ratio of 17 was insufficient. The long distances that creep can develop under a horizontal foundation surface are strikingly shown, emphasizing the value of creep distances along vertical surfaces. The necessity of tight sheet piling well connected to the foundation masonry and of tying in the ends of the sheet piling cutoff at the abutments so that the water cannot find its way around the end of the cutoff are convincingly illustrated.

Kitcha Bye-Wash Weir, Rohilihand Canals, United Provinces, India

This structure was founded upon dry sand above ground water level. It had a puddle upstream apron covered with grouted riprap and a stepped masonry drop, with 12' sheet piling at the downstream edge. There was no upstream cutoff. Normally no water was against this structure. Several times small flows

passed over it, each time causing piping and failure of the floor. The first time a real test of the structure occurred, it failed to such an extent as to require abandonment.

There are differing opinions as to the cause of the failure, evidently it was a case of piping, but the path of the piping is not definite. The water seems to have passed thru cracks in the puddle, which had probably dried out, or thru the contact of the puddle and the masonry. The head was about 8.0 feet and the creep distance which the water actually took, probably about 58 feet, making the plain creep head ratio 7.2 and a weighted creep head ratio of 4.3. The corresponding figures for short path are 47 feet and 5.9. Possibly the settlement of the dry sand when brought into contact with water contributed to the failure of the structure. It is possible also that a head brought suddenly on a dry sand would give rise to greater percolation than sand continuously wet. This failure serves to bring to attention the danger of puddle aprons where they may dry out, and the possible danger of founding a structure on dry soil.

Kulli Dam, Rohiliband Canal, India

This dam was originally constructed with an apron of insufficient weight, which was blown up, necessitating the reconstruction of the weir. The reconstructed weir stood without any signs of trouble for about ten years, when it suddenly failed, apparently from piping along the line of creep. The structure was founded on fine sand, without any cutoff walls or sheet piling. The creep ratio was between 1:14 and 1:16. The structure was rebuilt with much the same section as before, but with an upstream cutoff of interlocking steel sheet piling, and has given no trouble since. The data available is insufficient to give exact data on this structure, but its failure is of major importance as showing the necessity of vertical staunching if short-or creep ratios than those advocated by Bligh and Griffith are to be used.

Kulli Bye Wash, Rohiliband Canal, India

This structure had several failures of the upstream apron which were patched up with reinforced concrete, but after 3 or 4 years it failed beyond repair. It was built on fine sand, without cutoff walls or sheet piling and had a plain creep ratio of between 14 and 16.

Corpus Christi Dam (Abutment Failure) Nueces River, Texas

The author believes that the failure of the Corous Christi Dam abutment was due to the existence of a short percolation path under and behind the abutment. This is the only record of a failure due to this cause which he has found, and the only warning of this possibility of such a failure seems to have been given by Griffith.* The short path was apparently down the up-

*Proc., Institution of Civil Engineers, Vol. 197, Pt. III, P. 229.

stream face of the dam directly over the edge of the abutmont, under the abutment to its rear edge, up over the top of the sheet piling, down to the bottom of the abutment and thence to the drain. There were two lines of tile drain under the dam. The creep path to the end of the upstream one of these was apparently about 103 feet and to the downstream one 154 feet which with a head of approximately 37 feet would give plain creep ratios of 2.8 and 4.2 and weighted creep ratios of 1.6 and 2.1 respectively. The data available does not indicate a filter around the tile drain and fine material seems to have escaped through one of the drains, until a "pipe" was formed leading to that drain. As this enlarged the flow became greater than the drain could carry without pressure, and a pressure under the apron was built up which eventually blew up the apron. The water rushing through this opening and the "pipe" already formed, rapidly enlarged its passageway and undermined the abutment. It may be concluded from this that the weighted creep ratio responsible for failure was not over 2.1 and may have been 1.6. The material at the abutment was sand.

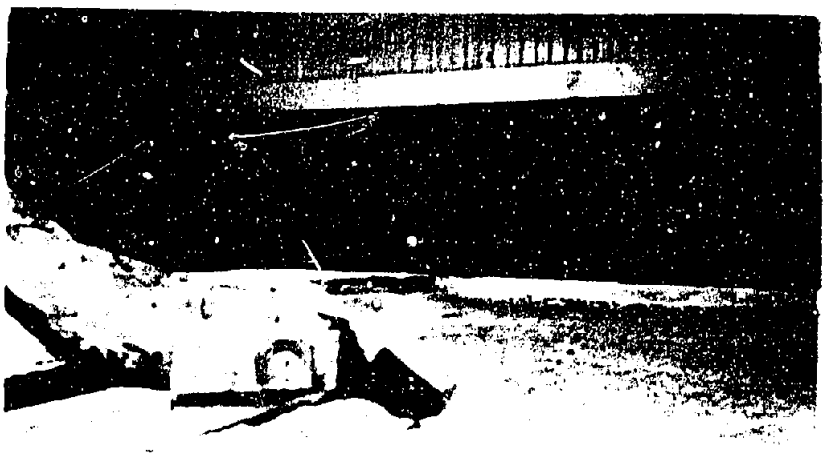
Iron Mountain Dam, Menominee River, Michigan

The bed of the river at the damsite is covered with a layer of sand and gravel, paved with small boulders, beneath which is fine sand. The dam probably rests on this sand layer. It sustains a head of 30 feet with a creep distance of 257 feet, giving a plain creep head ratio of 8.6 and a weighted creep head ratio of 5.5. The downstream apron of this dam is thin and the pressure beneath it is relieved by a drain of tile surrounded by gravel. This dam has been giving satisfactory service since about 1926.

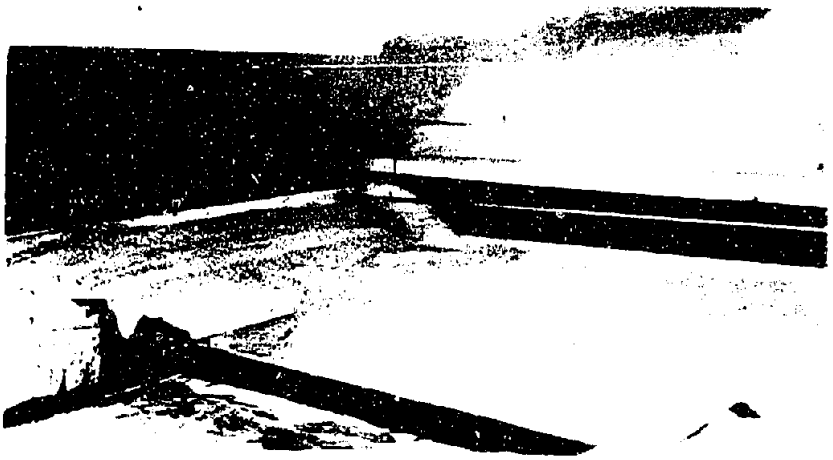
Nadrai Escape Fall, Lower Ganges Canal, India

This structure was built for a 14 ft. drop on light sandy soil, but retrogression of the water level below increased the head to 20 feet. The first trouble was experienced nine years after construction. The downstream apron was extended the next

TYPE I

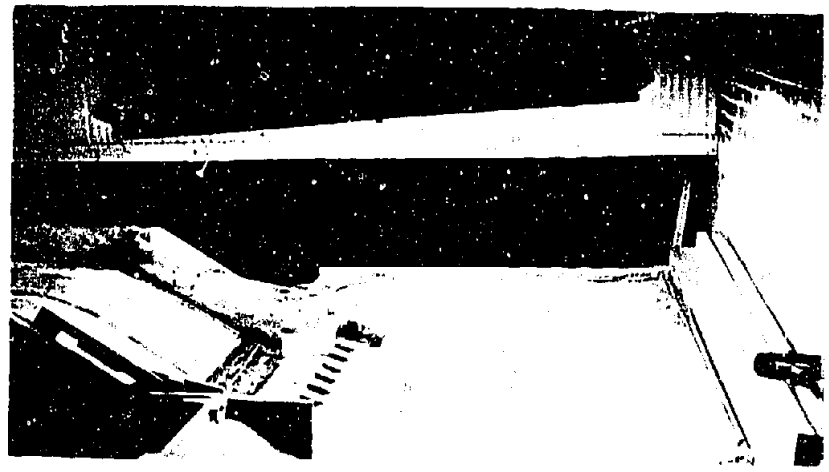


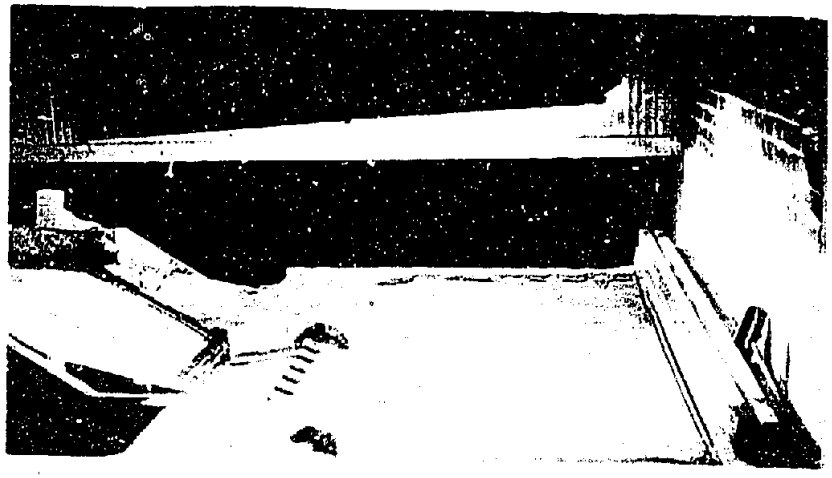
TYPE II



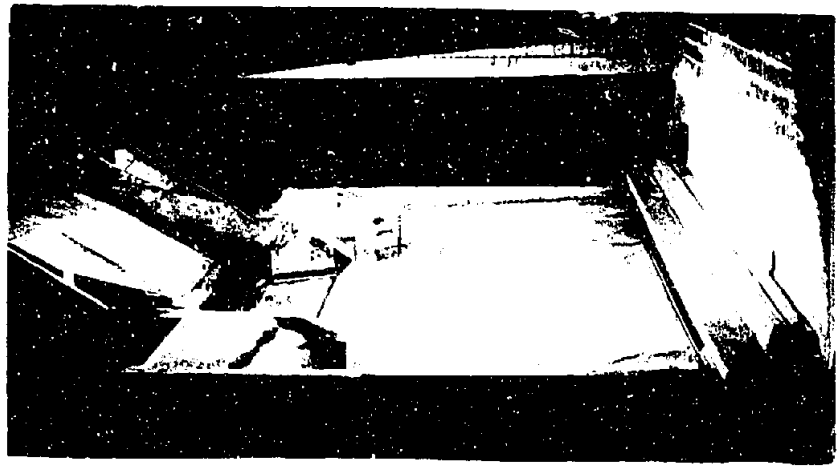
TYPE III

TYPES OF END WALLS TESTED





TYPE VI



TYPE VII

TYPES OF END WALLS TESTED

but would be very expensive to build. Most of the tests were made with Type II, but this gave somewhat too severe scour, so that Type V was adopted for the final design. This consisted of a wall extending out horizontally to the end of the floor where it ended vertically. The earth bank sloped up behind this wall on a 2:1 slope from near the bottom of the wall at its end to the top some distance back. This leaves a triangular section of the wall projecting above the earth which merely acts as a guide wall for the water. As the pressure on the two sides differs little, great strength is not required and therefore it need not be very expensive. This form of wall, together with the slope paving described later, was believed to be best form, considering both the hydraulic effects and cost.

Best Form of Bank Protection

The tests showed that there would be considerable scour of the banks below the pool during high floods. This resulted from wave action rather than high velocity currents. The washing was similar to that of waves breaking on a beach. At the Cle Elum Dam the topography is such that no damage will result from this action except near the ends of the pool walls. Different forms of protection for this area was therefore experimented upon. The first type simulated a concrete slab paving. It consisted of a layer of paraffin cast on the channel side slope as shown on Plate XLIII-A. The scour along the edge of this type undermined the paving, and the next test was run with a cutoff wall eight feet deep along the edge of the paving. This prevented the undermining, as shown on Plate XLIII-B. Better results however were accomplished by the further addition of a strip of riprap 10 feet wide at the outer edge of the paving, as shown on Plate XLIII-C. A paving entirely of riprap of stone to about two feet dimension (on the prototype) was tested. With Type II headwall this protection was not sufficient, as shown on Plate XLIII-D. With the type V walls finally adopted, this protection was adequate. The riprap on a $1\frac{1}{2}$:1 slope however, was not very secure and it was decided to use a 2:1 slope instead.

Downward Pressures

In the design of a stilling pool founded on permeable material, provision must be made to take care of the unbalanced pressure on the floor caused by the hydraulic jump. In a masonry lined pool, the pressure beneath the lining is caused by the level of the tailwater, since water from the tailwater can percolate through the foundation material to the lower side of the lining. This pressure acts upward and tends to lift the pool lining. On the upper side of the lining the weight of the water in the pool acts downward, resisting the upward pressure. Upstream from the jump this downward pressure depends on the level of the water surface flowing at that point, which is much

less than the tailwater level which is below the jump. The upward pressure on the bottom of the pool lining is therefore greater than the downward pressure due to the water on the top of the lining, and unless the lining is heavy enough, it will be raised up. This was strikingly shown on the model at one time when the sheet iron on the wooden floor of the pool was not fastened down securely and permitted a fine film of water to flow under it from the tailwater. When flow through the model was started and the hydraulic jump formed, the excess of pressure beneath the sheet iron lining pulled out the nails with which it was fastened down and caused it to rise up. There are quite a number of records of failures to stilling pools and dam aprons due to this cause.

Extensive observations were made to determine the water pressures which would act to hold the pool lining down in the Cle Elum stilling pool. To record these pressures piezometers were placed in the lining of the pool at various points. The results of the observations for a 40,000 sec. ft. discharge is shown on Figure 59. This shows that the downward pressure on the pool bottom is shown by the piezometers, and the actual water surface elevations. The downward pressure is considerably less than the height of the water surface in the jump due no doubt to the air content of the water in the jump which decreases its density and therefore its downward pressure. This difference between the downward pressure and the water surface may be observed on Plates XIII to XIX and several others by noting the difference of water level in the pool and in the piezometer tubes which are connected to the pool bottom directly behind the point at which they are located.

Experiments were also made showing the pressures for various discharges, tailwater levels, types of sills and entrance slopes as shown on Figures 60 to 65 inclusive. They show that the reduction in pressure due to the jump increases as the discharge increases and as the tailwater level is lowered. Figures 62 to 64 indicate that the existence or shape of the sill has little effect on these pressures, within the range covered by these experiments, and Figure 65 indicates the entrance slope to the pool is not of great importance. Pressures were also observed on the bottom of the pools of 150 and 120 ft. width, for a 40,000 sec. ft. discharge, as shown on Figures 66 and 67.

It should be noted that the pressures shown in the diagrams are average pressures, and momentary pressures vary from these considerably in both directions. The values given should therefore be used with a considerable factor of safety. Probably the best way to provide for this upward pressure is to build an adequate system of drains beneath the pool floor, discharging into the pool at the point

where the downward pressure on the floor is a minimum. Such a system, discharging through the side wall of the pool near its upstream end is to be built at the Cle Elum Dam.

Studies of the Tunnel Outlet

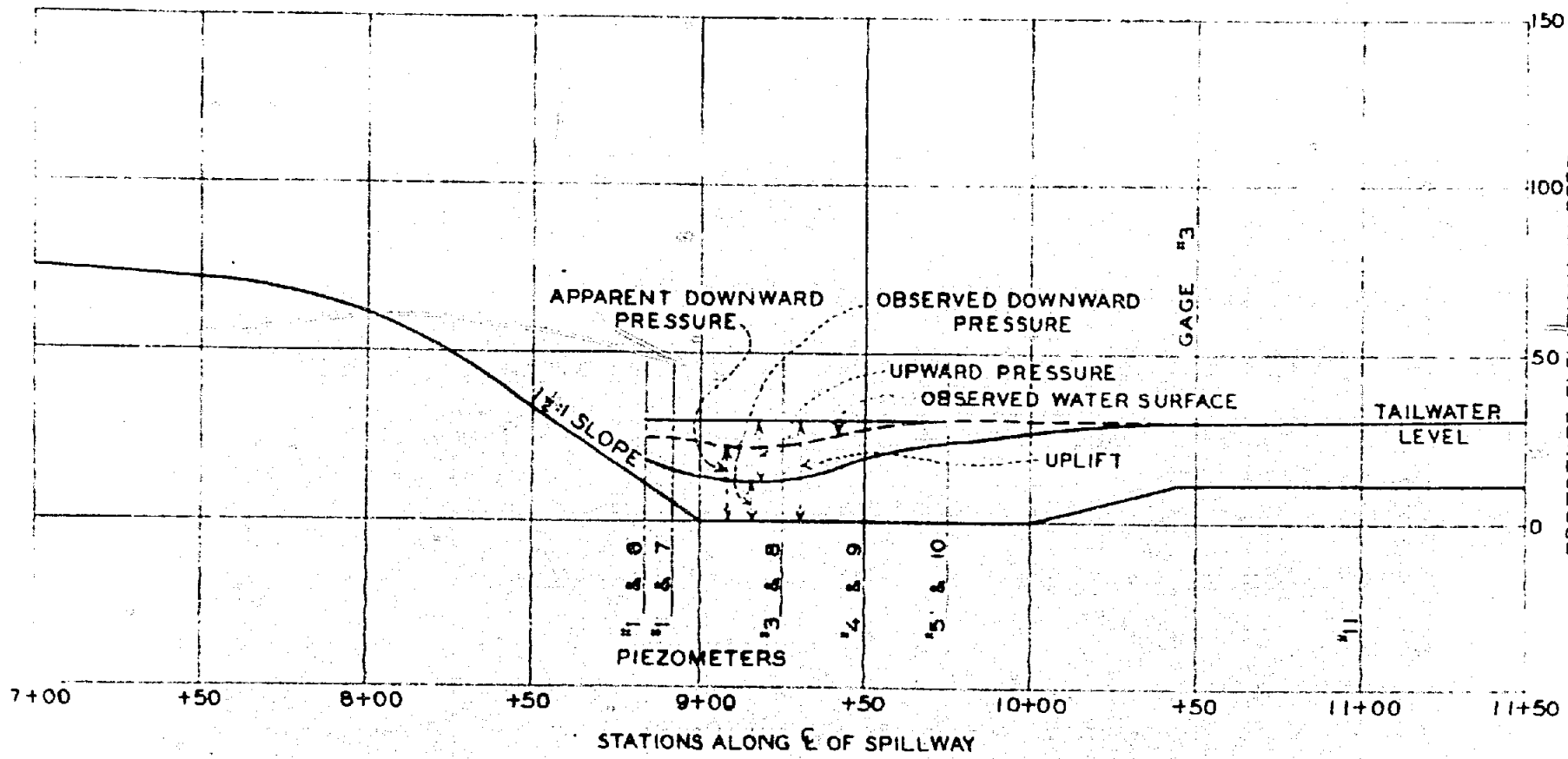
The plan for the outlet tunnel of the reservoir contemplates discharging the water from it into the stilling pool through the face of the steep slope leading into the pool. In flowing over this opening however the water coming down the spillway trough would be unsupported and would drop more nearly vertical than the flume slope, with the result that it would impinge on the tunnel bottom with considerable impact. In order to remedy this a curved deflector was placed on the slope above the tunnel opening as shown on Plate XLIV, to raise the flowing stream sufficiently to make it jump entirely over the opening. Another method was to curve the bottom of the tunnel floor downward, as shown on Figure 68 in order that the water passing over might strike the tunnel floor at an acute angle and thus largely without impact. The latter method was found to be the most satisfactory and was therefore adopted.

Tests of the Final Design

An extensive series of tests were made on the final design in order to detect weakness in it, if any existed. For this purpose a series of runs were made with discharges less than that for which the structure was designed, the tailwater level in each corresponding to the discharge used. The results of these runs are shown on Plate XLV. The first shows the effect of a flow of 10,000 sec. ft. The next three show the condition at the end of similar runs of 20,000, 30,000 and 40,000 sec. ft. respectively. In each case the runs were of one hour duration, corresponding to 7.071 hours on the prototype. The condition at the beginning of each run is shown on Plate XLVII-A.

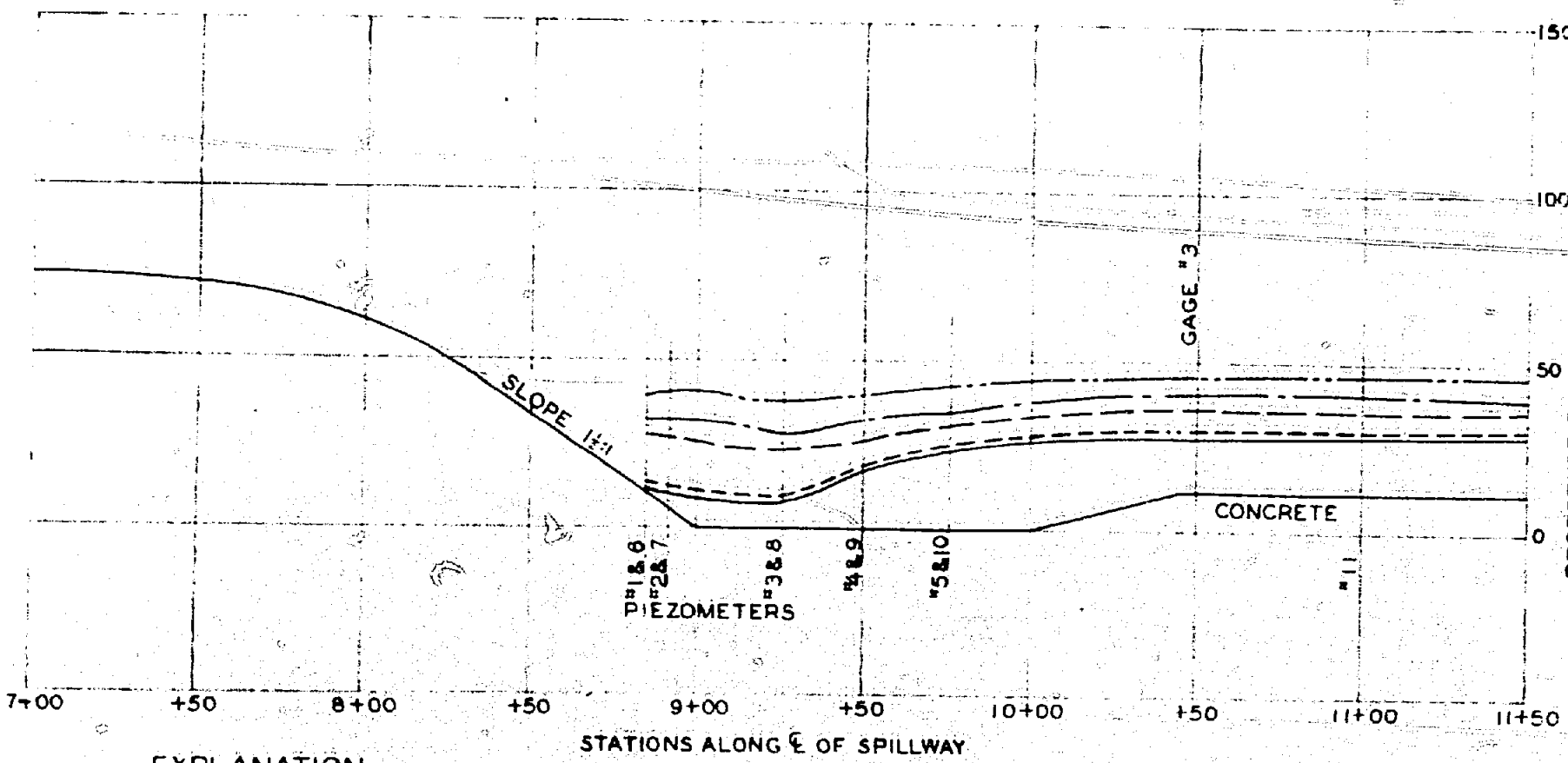
The scour at the downstream edge of the pool floor and 100 ft. downstream is shown graphically on Figure 69. These tests showed greater scour with the increase in discharge, but in no case was there a severe attack on the bottom where it would endanger the pool walls and floor; in fact, there was a deposit at the end of the floor and on the riprap bank protection rather than a scour on the channel sides.

In order to show the efficacy of the sill, a run was made without one, as shown on Plate XLVI-A and B. By comparison with the results of the one hour run with 40,000 sec. ft. discharge, on Plate XLV it will be seen that without the sill there was considerably more scour on the riprap of the channel sides and somewhat more at the bottom. At the end of the 1-hour run (7.071 hours prototype time)



CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
COMPARISON OF DOWNWARD PRESSURE
WITH ACTUAL WATER ELEVATION
40,000 c.f.s. DISCHARGE

FIGURE 59

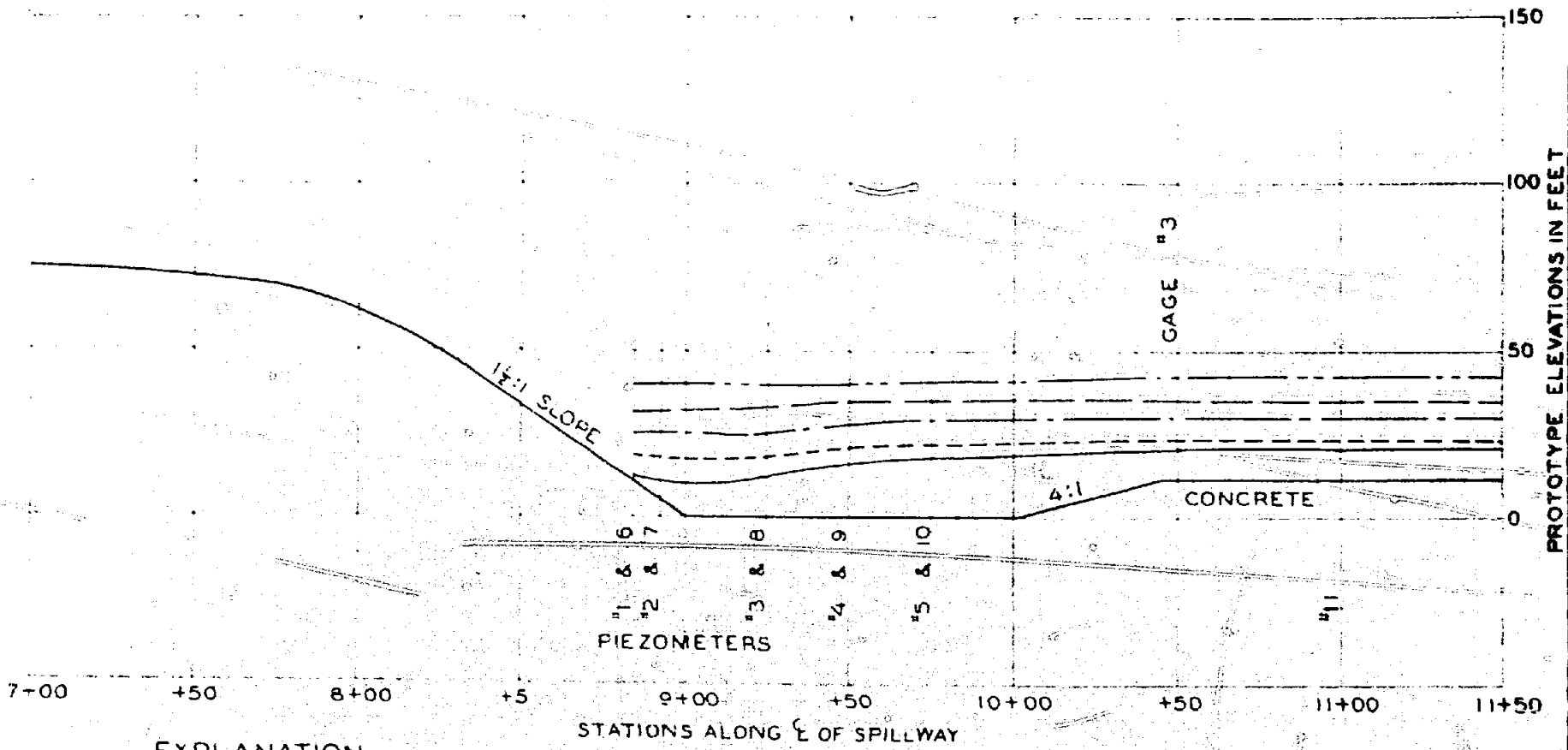


EXPLANATION

- RUN NO. 1
- RUN NO. 2
- RUN NO. 3
- RUN NO. 4
- RUN NO. 5

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
DOWNWARD PRESSURE CURVE
FOR VARIOUS TAILWATER LEVELS
40,000 c.f.s. DISCHARGE

FIGURE 60

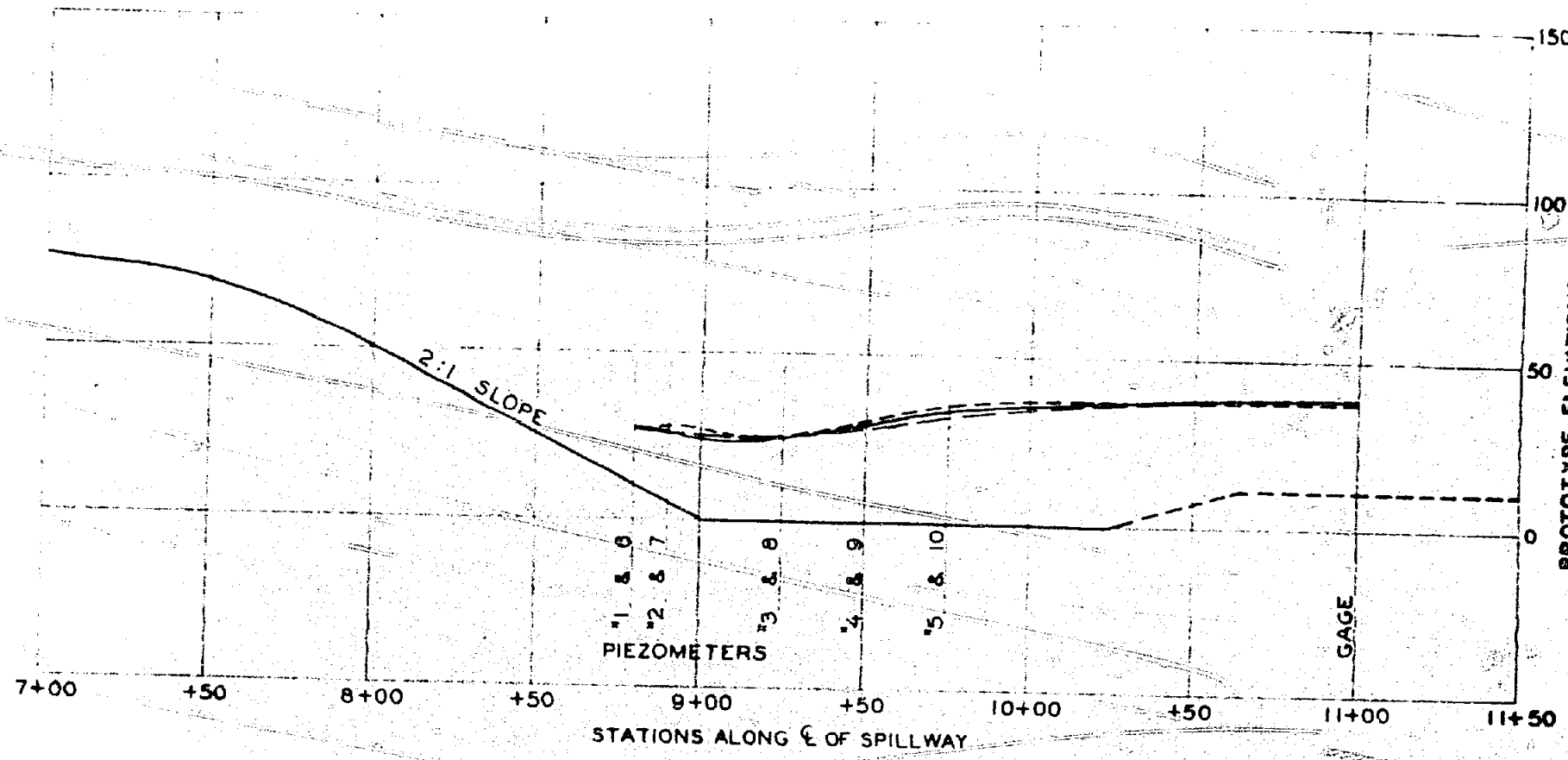


EXPLANATION

- RUN NO. 5
- RUN NO. 7
- RUN NO. 8
- RUN NO. 9
- RUN NO. 10

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
DOWNWARD PRESSURE CURVES
FOR VARIOUS TAILWATER LEVELS
17,600 cfs. DISCHARGE

FIGURE 61



EXPLANATION

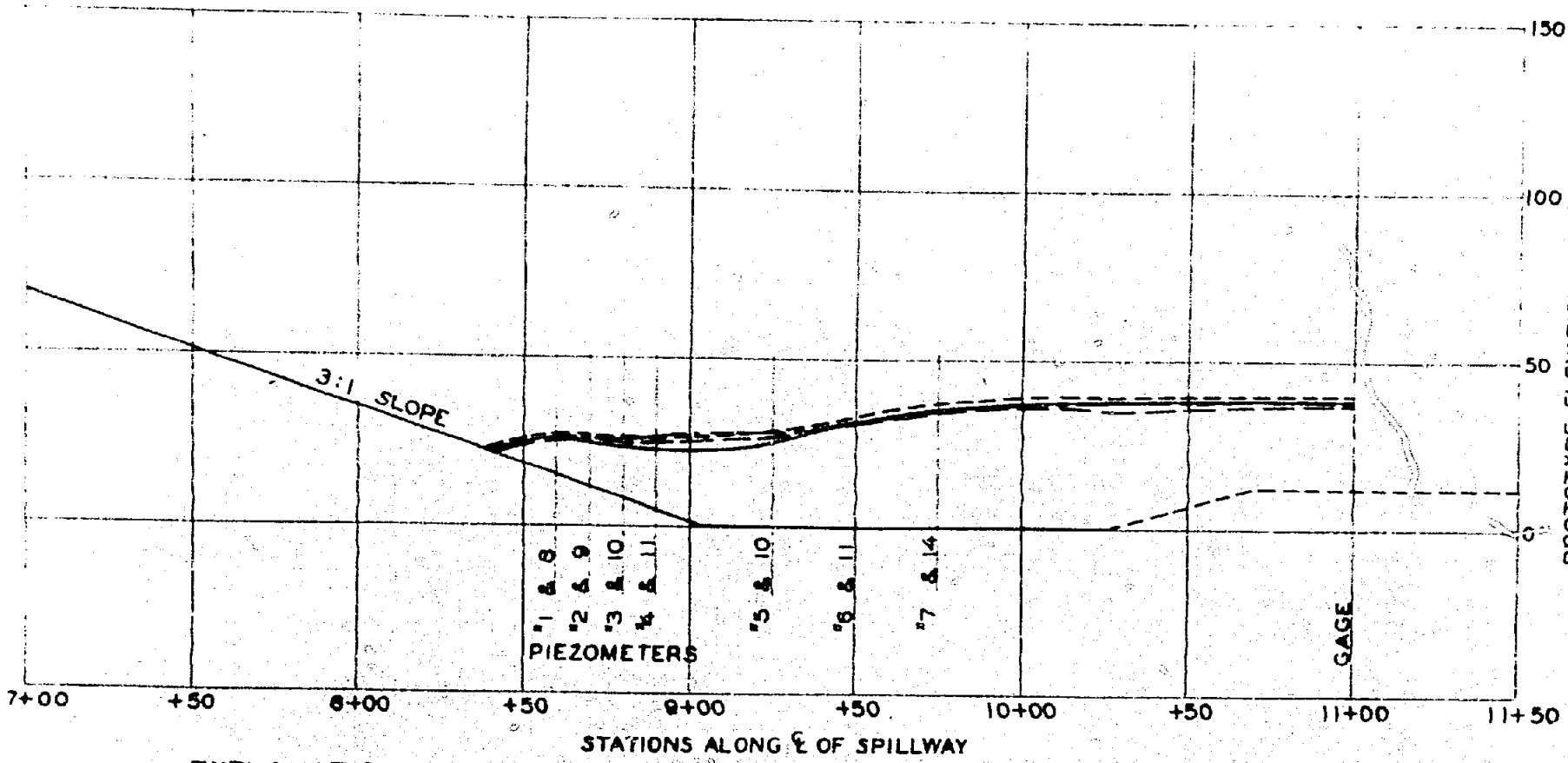
——— WITH NO SILL

- - - - - WITH 10FT. PLAIN SILL AT STA. 10+00

— — — — WITH 10FT. DENTATED SILL AT STA. 10+00

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
DOWNWARD PRESSURE CURVE
FOR DIFFERENT SILLS – 2:1 SLOPE
40,000 c.f.s. DISCHARGE

FIGURE 62

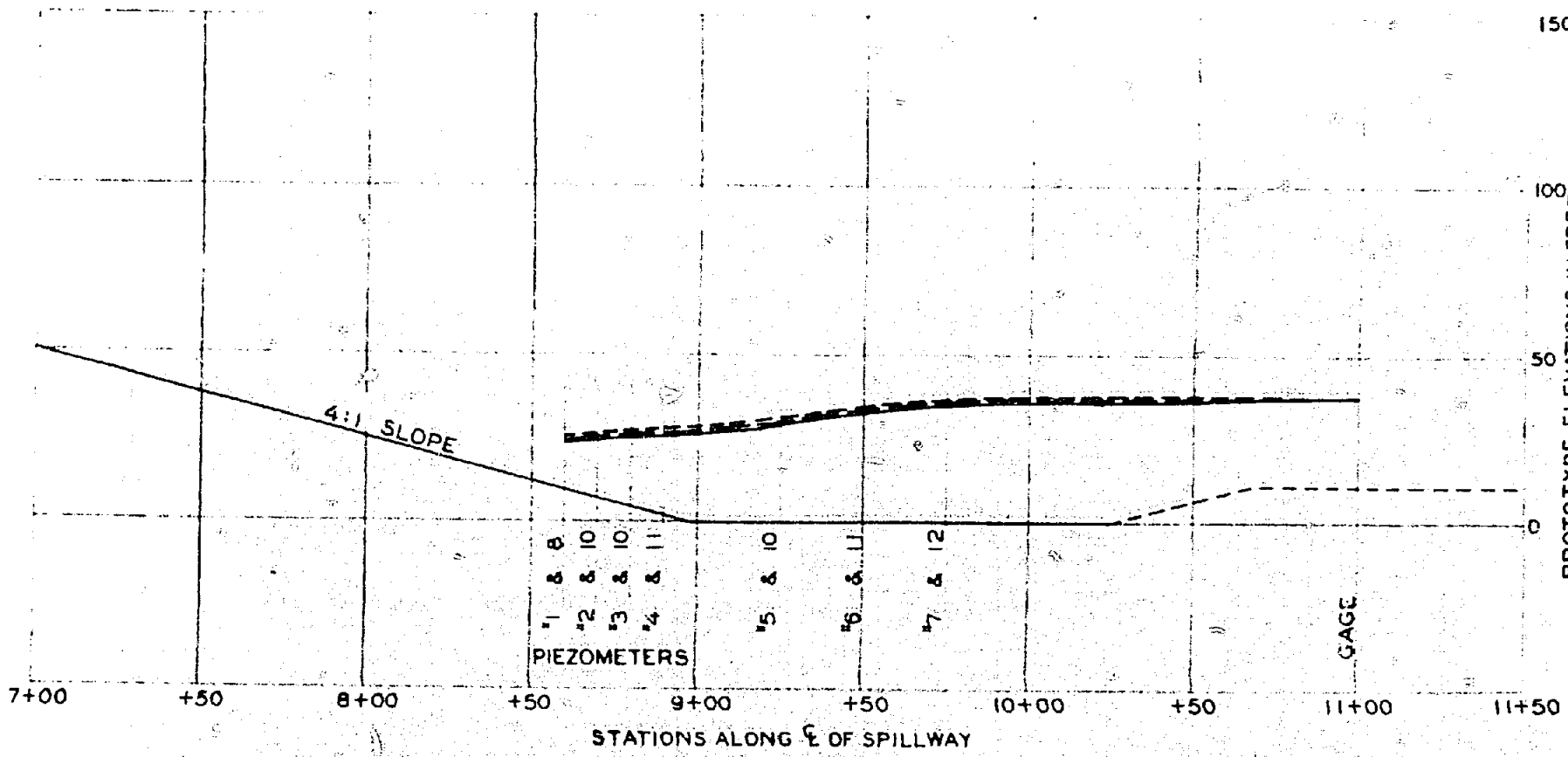


EXPLANATION

- WITH NO SILL
- WITH 10FT. PLAIN SILL AT STA. 10+00
- WITH 10FT. PLAIN SILL AT STA. 10+25
- WITH 10FT. DENTATED SILL AT STA. 10+00

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
DOWNWARD PRESSURE CURVE
FOR DIFFERENT SILLS – 3:1 SLOPE
40,000 c.f.s. DISCHARGE

FIGURE 63



EXPLANATION

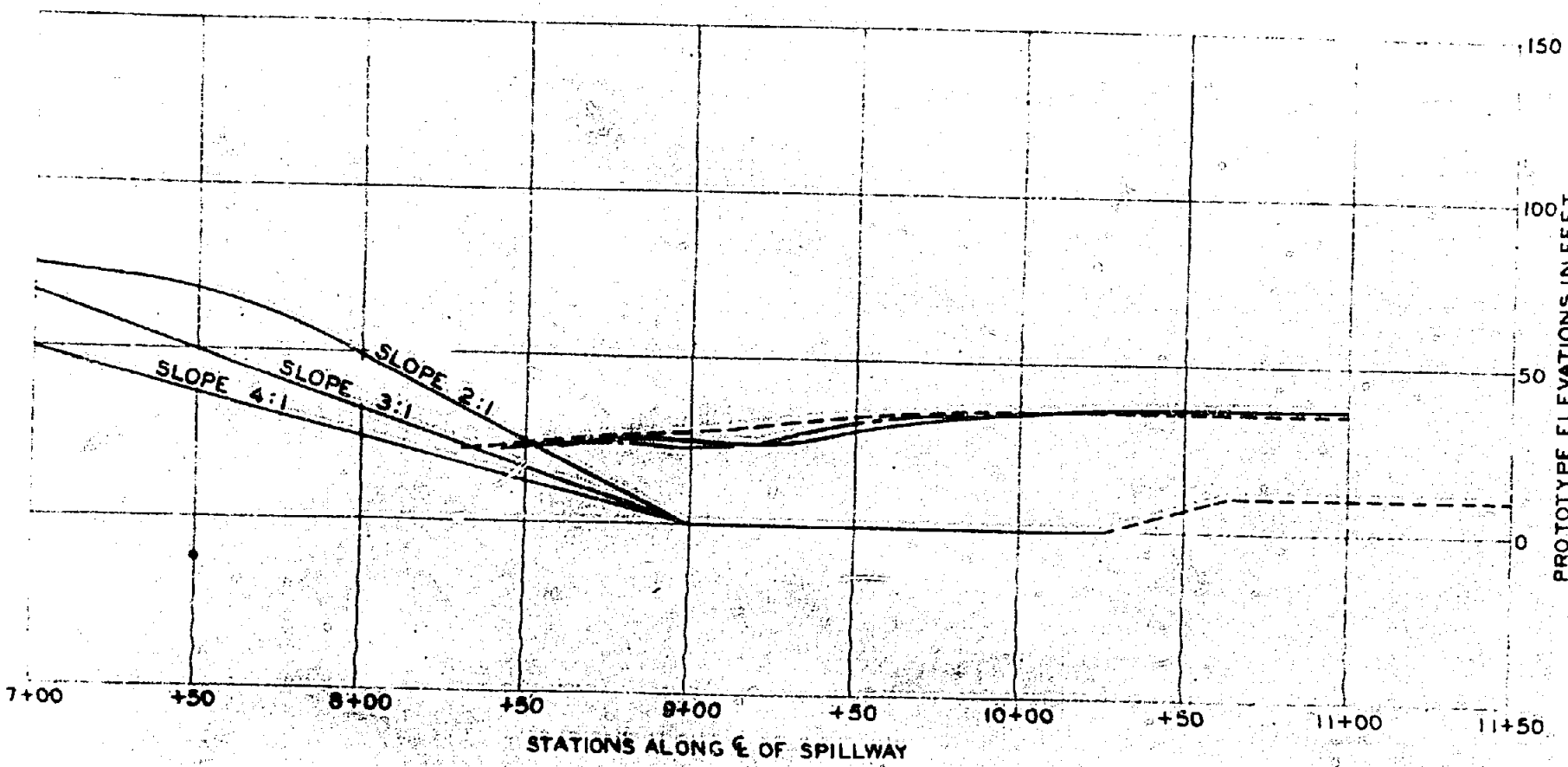
——— WITH NO SILL

- - - - - WITH 10FT. PLAIN SILL AT STA. 10+00

— — — WITH 10FT. DENTATED SILL AT STA. 10+00

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
DOWNWARD PRESSURE CURVES
FOR DIFFERENT SILLS - 4:1 SLOPE
40,000 c.f.s. DISCHARGE

FIGURE 64



EXPLANATION

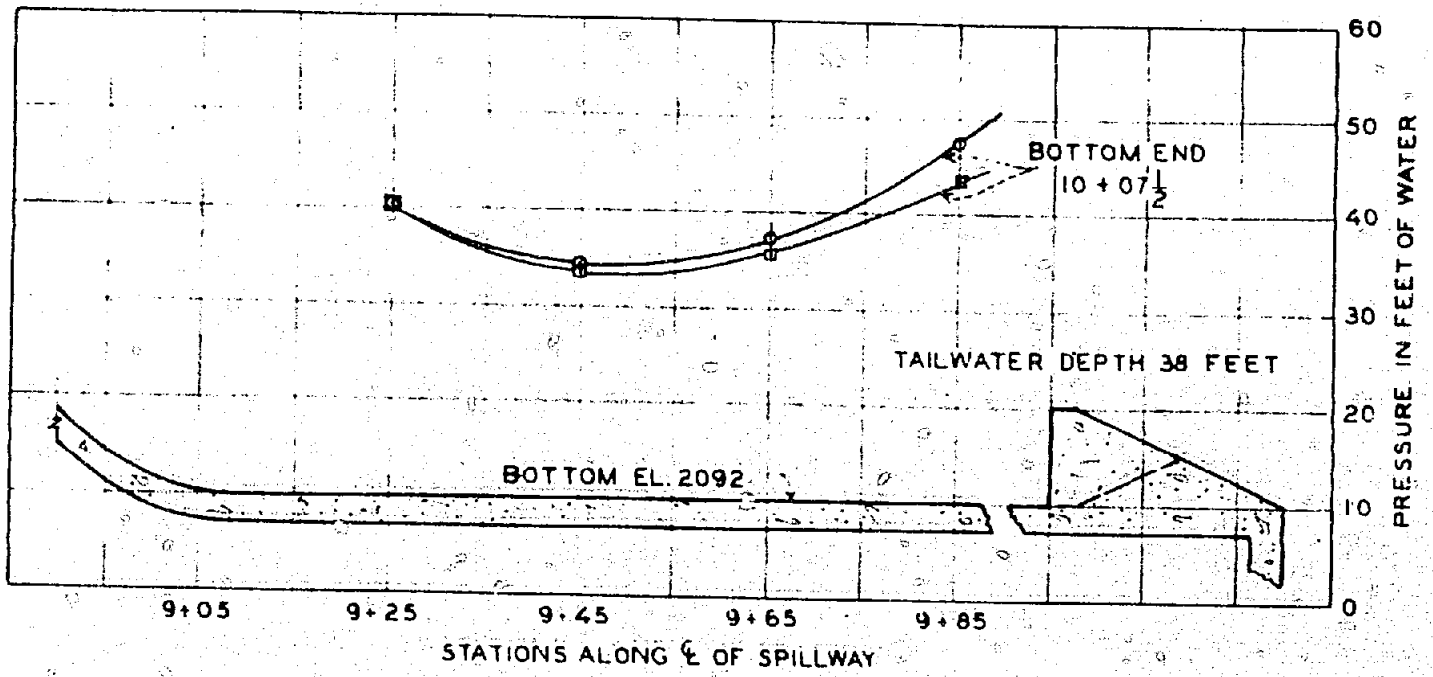
CURVE FOR 2 1 SLOPE

CURVE FOR 3 1 SLOPE

CURVE FOR 4 1 SLOPE

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
COMPARISON OF AVERAGE DOWNWARD PRESSURE FOR DIFFERENT FLUME SLOPES
40,000 c.f.s. DISCHARGE

FIGURE 65



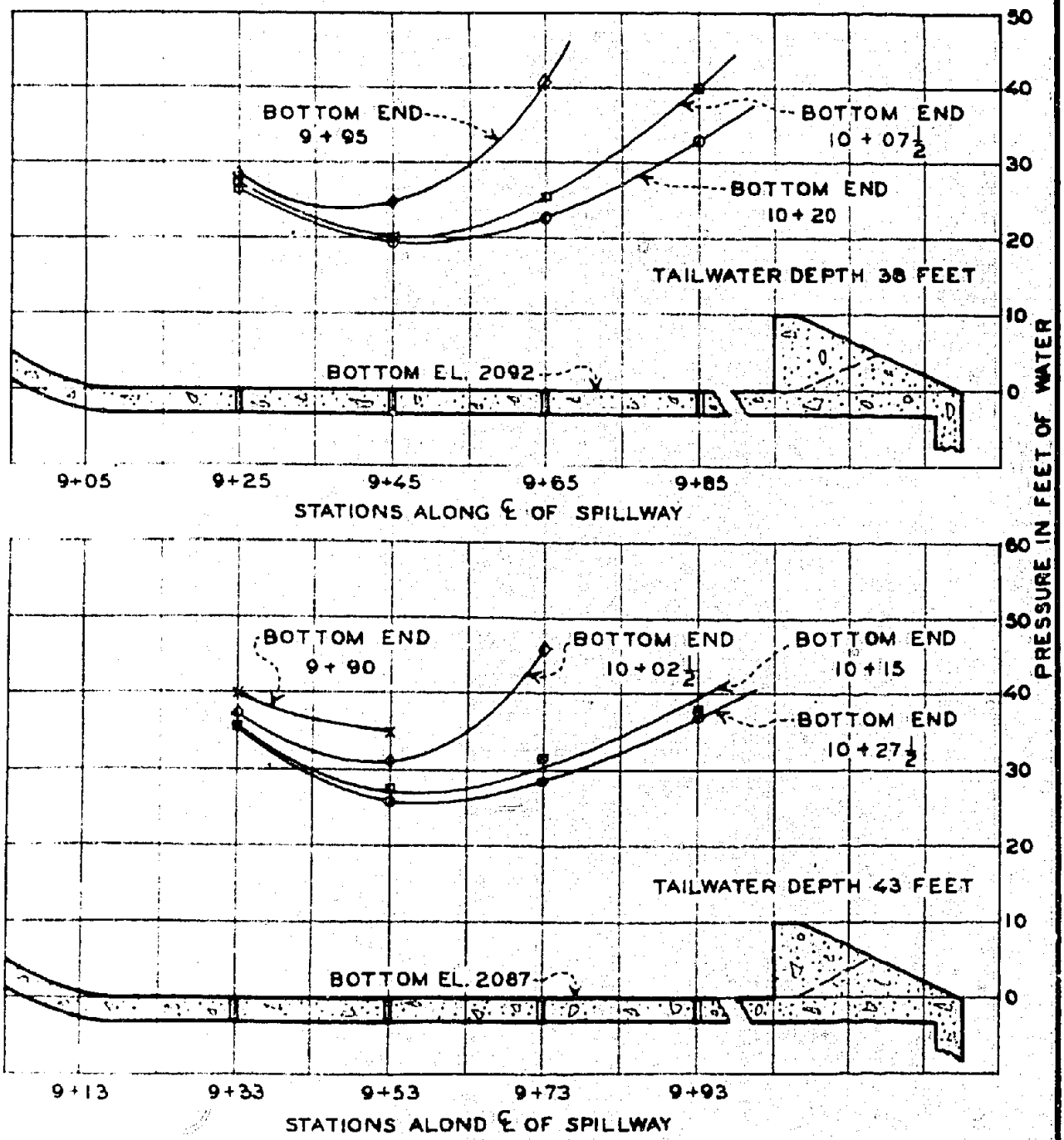
CLE ELUM SPILLWAY EXPERIMENTS

DOWNWARD PRESSURE ON POOL BOTTOM

150 FT POOL WIDTH

FIGURE 66

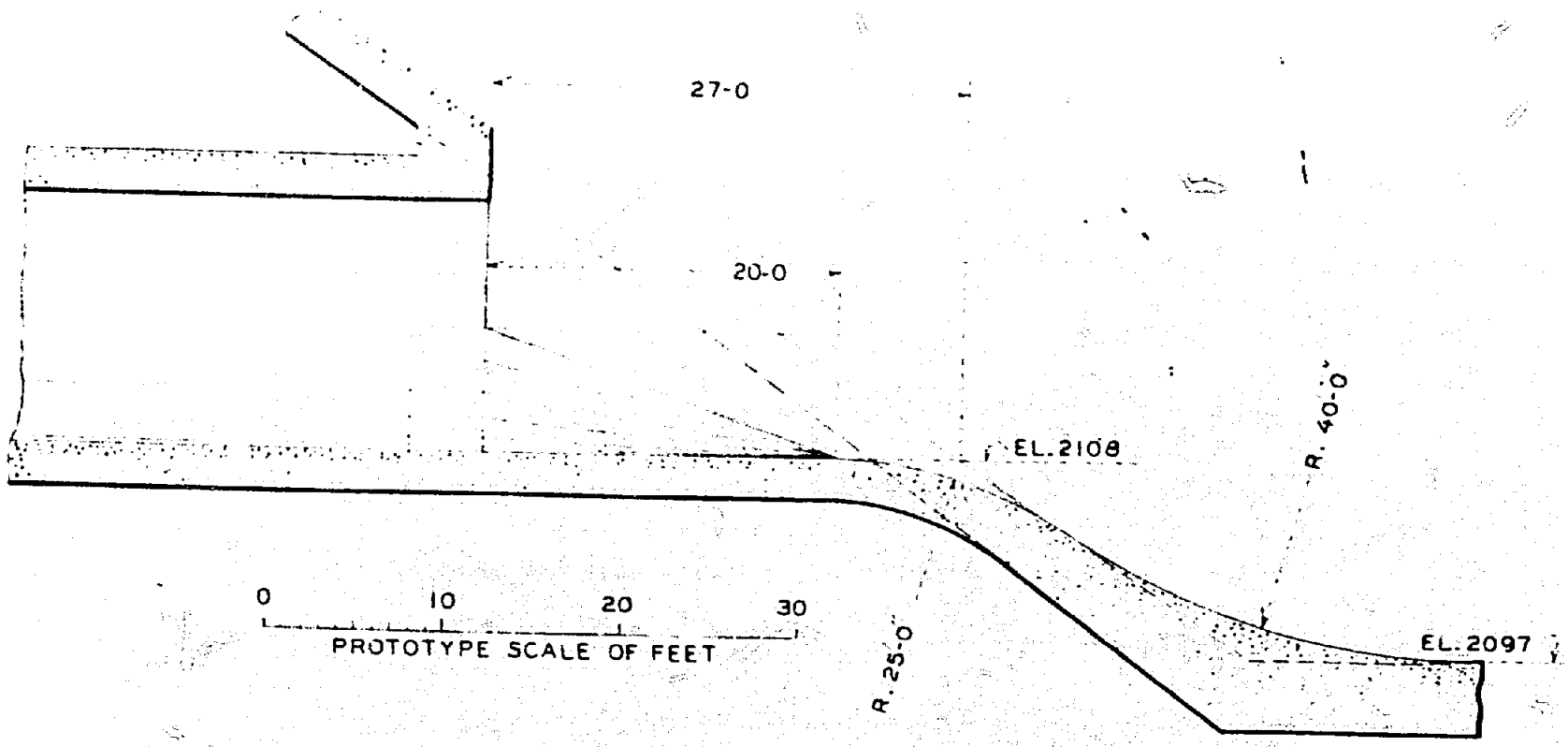
FIGURE 67



CLE ELUM SPILLWAY EXPERIMENTS

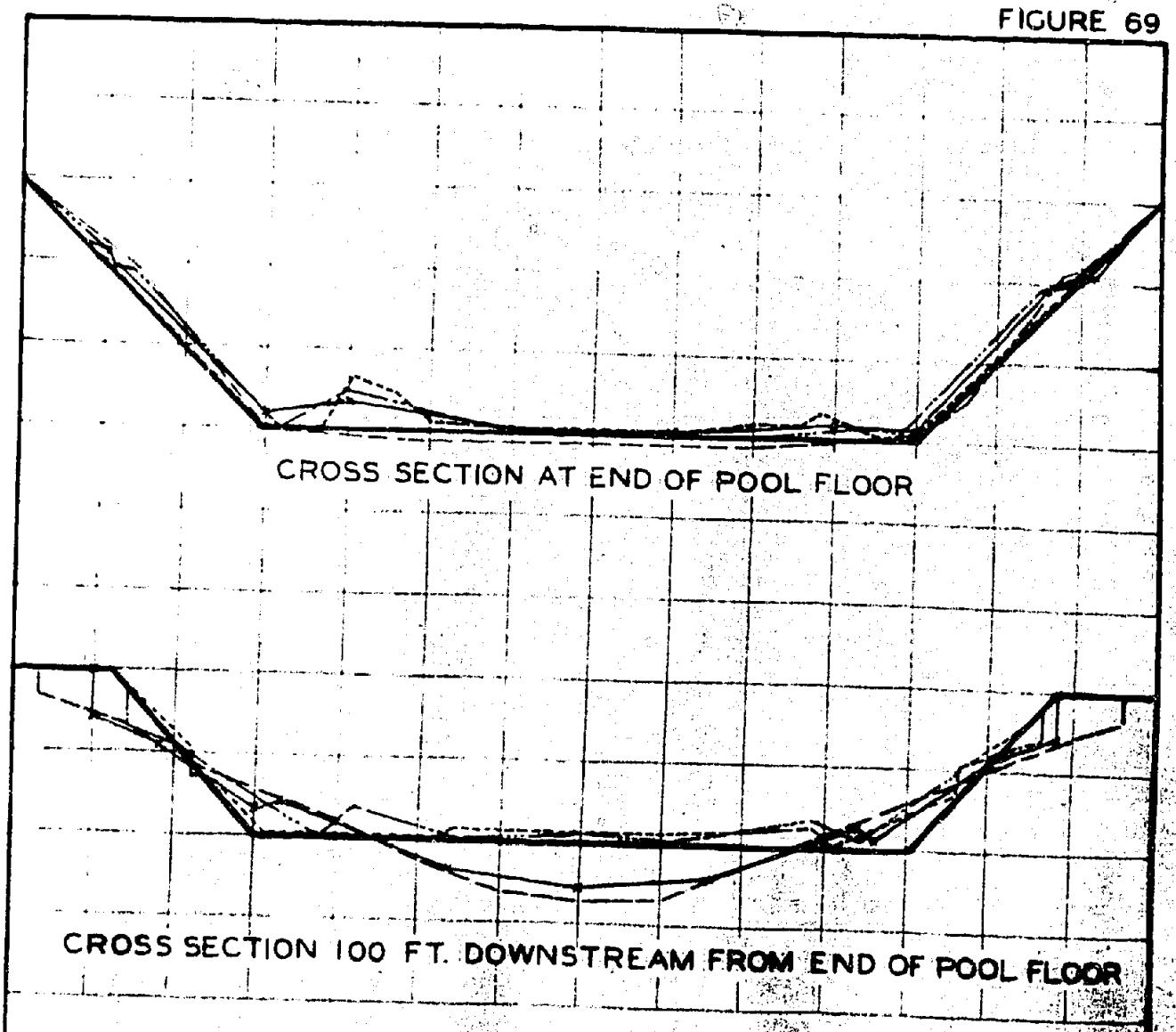
DOWNWARD PRESSURE ON POOL BOTTOM

120 FT. POOL WIDTH



CLE ELUM SPILLWAY EXPERIMENTS
CURVED FLOOR AT TUNNEL OUTLET

FIGURE 68



SPILLWAY BOTTOM

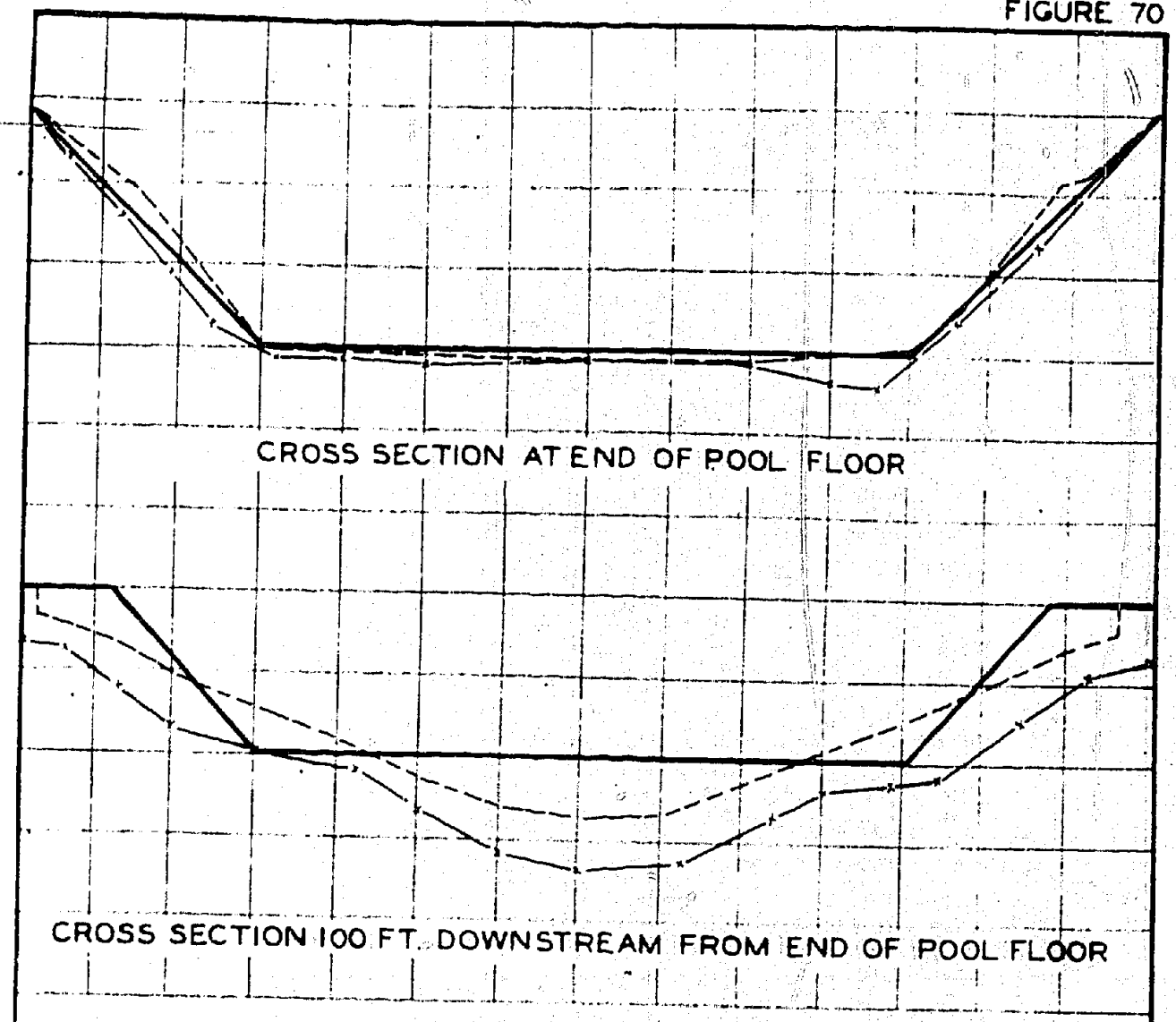
- BEFORE RUN
- AFTER 10,000 SEC. FT. RUN
- AFTER 20,000 SEC. FT. RUN
- AFTER 30,000 SEC. FT. RUN
- AFTER 40,000 SEC. FT. RUN

0 25 50
VERTICAL SCALE OF FEET

0 25 50 75 100
HORIZONTAL SCALE OF FEET

CLE ELUM SPILLWAY EXPERIMENTS
SCOUR WITH VARIOUS DISCHARGES
FINAL DESIGN

FIGURE 70



SPILLWAY BOTTOM

BEFORE RUN

END OF RUN 1 HR. MODEL - 7.1 HR. PROTOTYPE

END OF RUN 5 HR. MODEL - 35.4 HR. PROTOTYPE

0 25 50

VERTICAL SCALE OF FEET

0 25 50 75 100

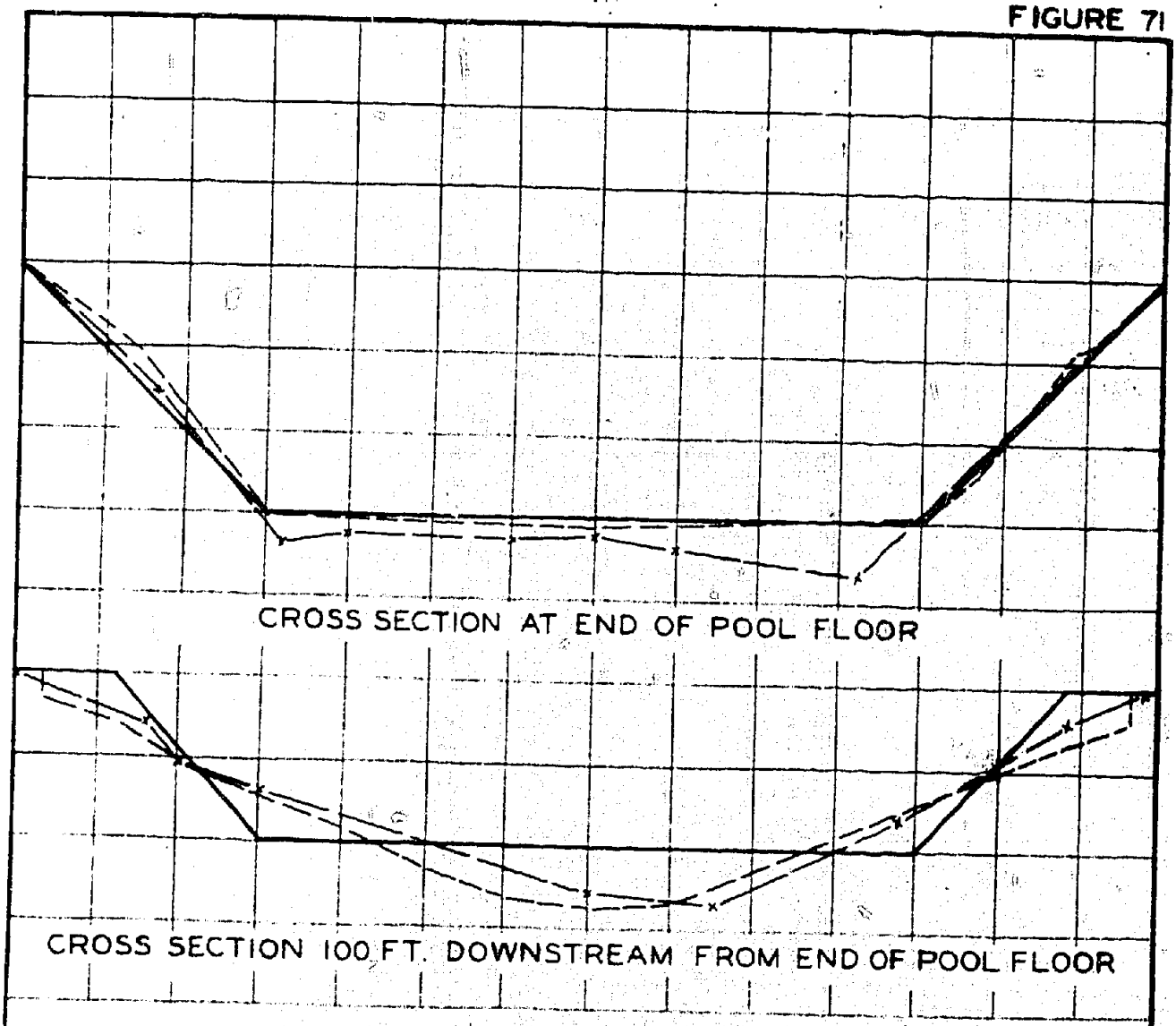
HORIZONTAL SCALE OF FEET

CLE ELUM SPILLWAY EXPERIMENTS

EFFECT OF LENGTH OF RUN ON SCOUR

FINAL DESIGN

FIGURE 71



SPILLWAY BOTTOM

BEFORE RUN

AFTER RUN - ORIGINAL DESIGN

AFTER RUN - FINAL DESIGN

0 25 50

VERTICAL SCALE OF FEET

0 25 50 75 100

HORIZONTAL SCALE OF FEET

CLE ELUM SPILLWAY EXPERIMENTS
COMPARISON OF SCOUR
ORIGINAL AND FINAL DESIGN

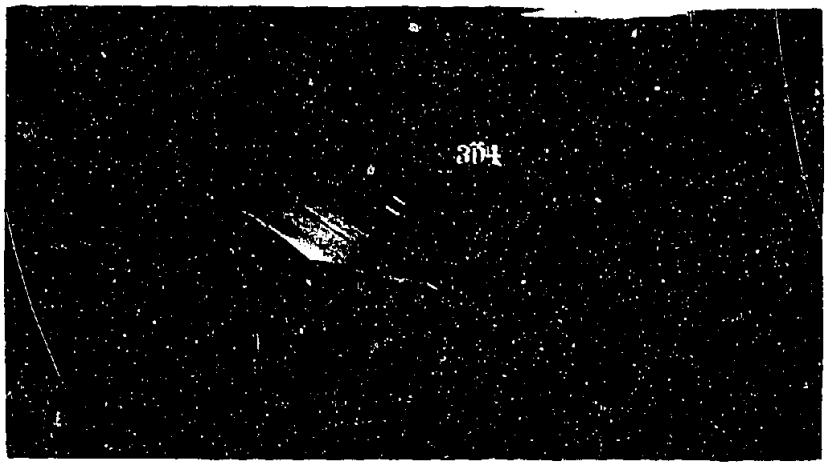
a - CONCRETE PAVING AFTER THE RUN

b - RIPRAP PAVING AFTER THE RUN

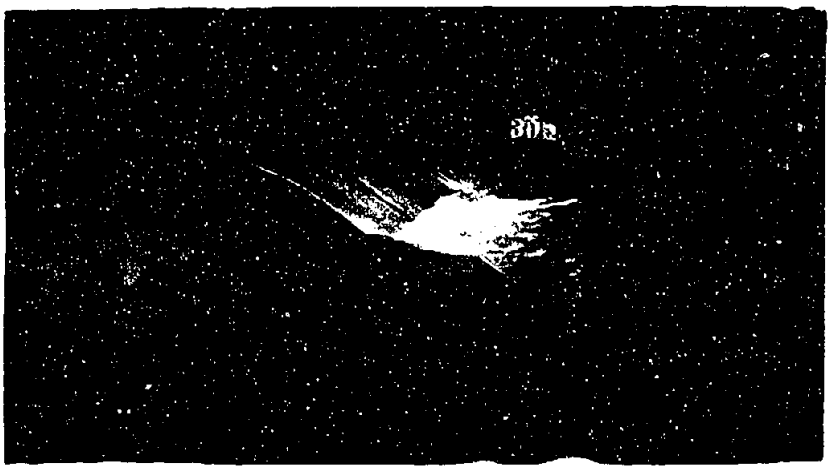
c - [illegible] PAVING BEFORE THE RUN

d - CONCRETE PAVING WITH RIPRAP AFTER THE RUN

TYPES OF BANK PAVING STUDIED

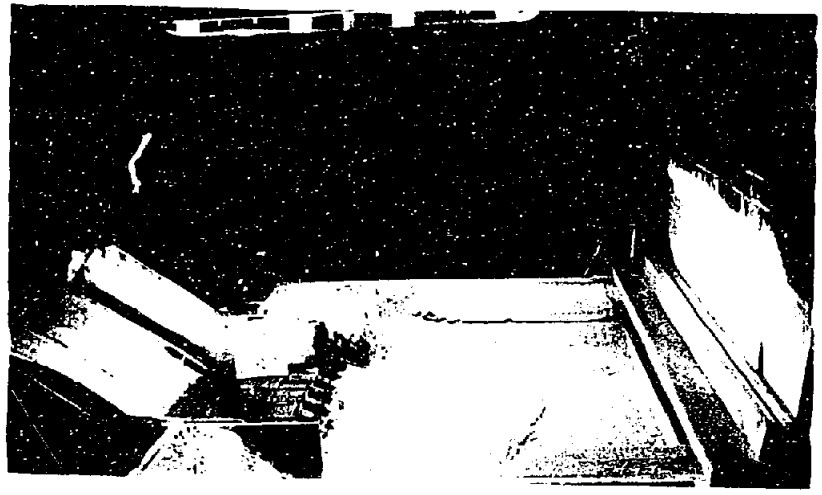


A - SHOWING DEFLECTOR OVER TUNNEL OUTLET

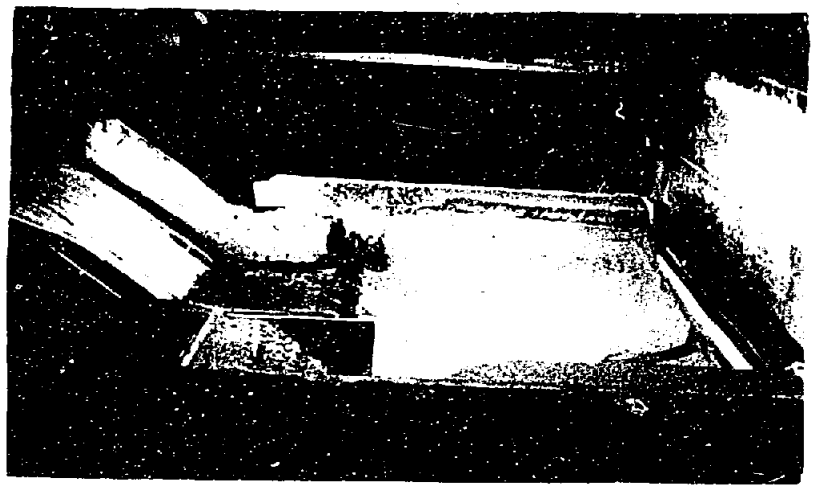


B - SHOWING DEFLECTOR IN ACTION

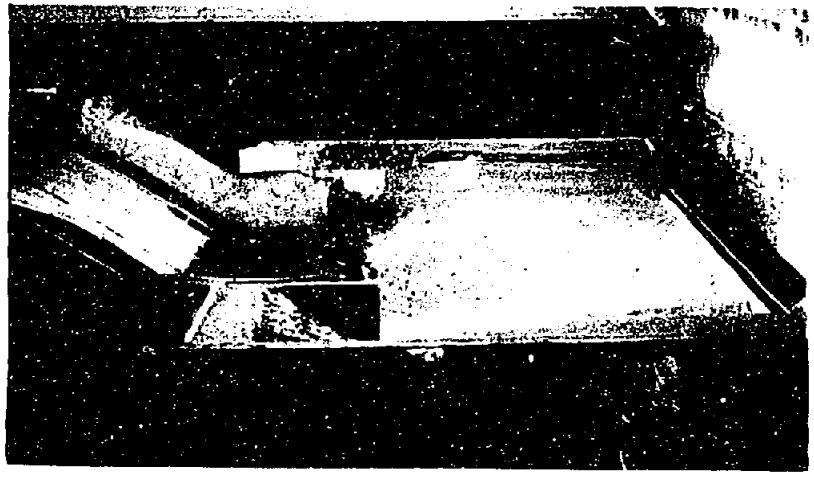
STUDY OF TUNNEL OUTLET



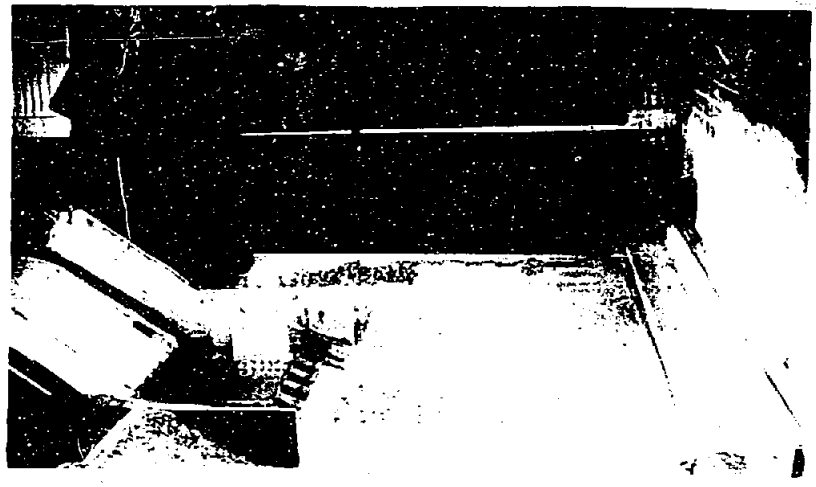
A-10,000 SEC FT FLOW



B-20,000 SEC FT FLOW

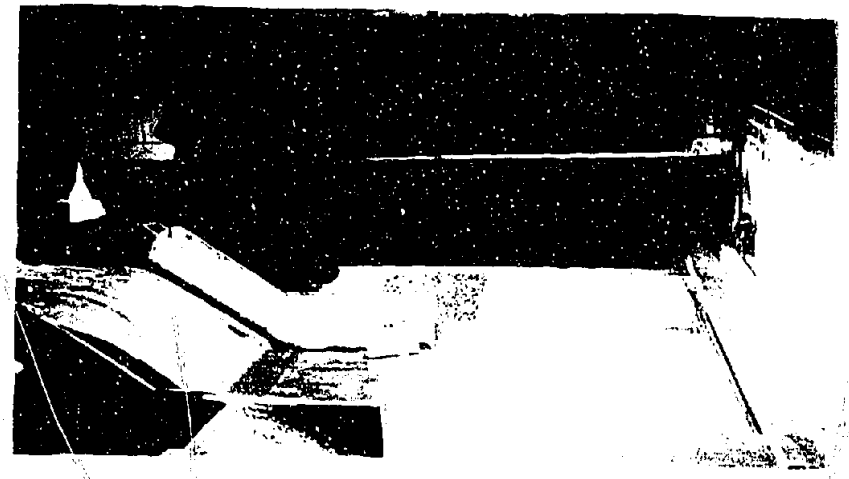


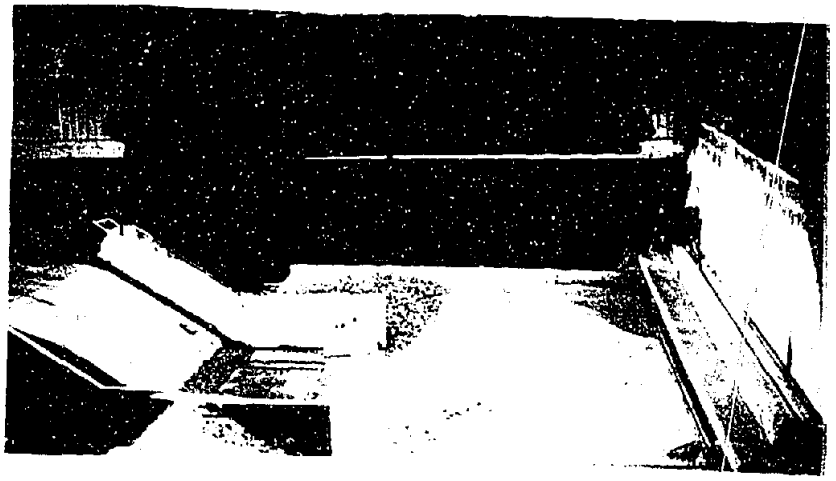
C-30,000 SEC FT FLOW

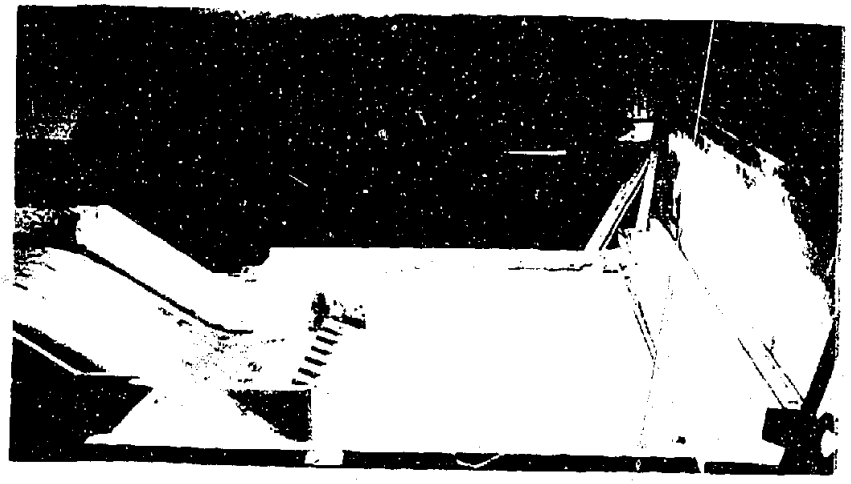


D-40,000 SEC FT FLOW

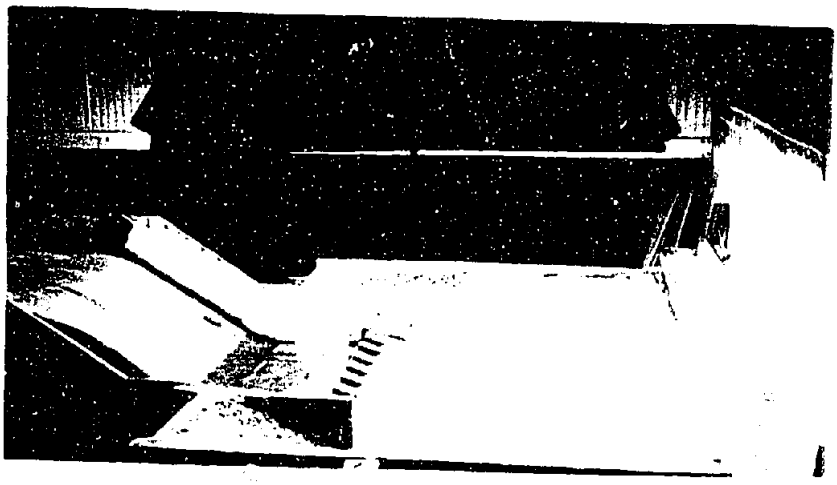
PLATE XLV



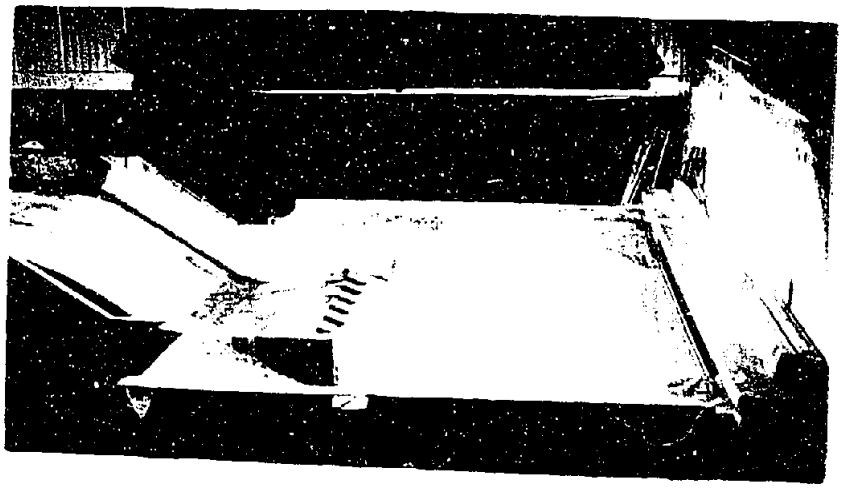




A. SAND PIT BEFORE THE BLOW.

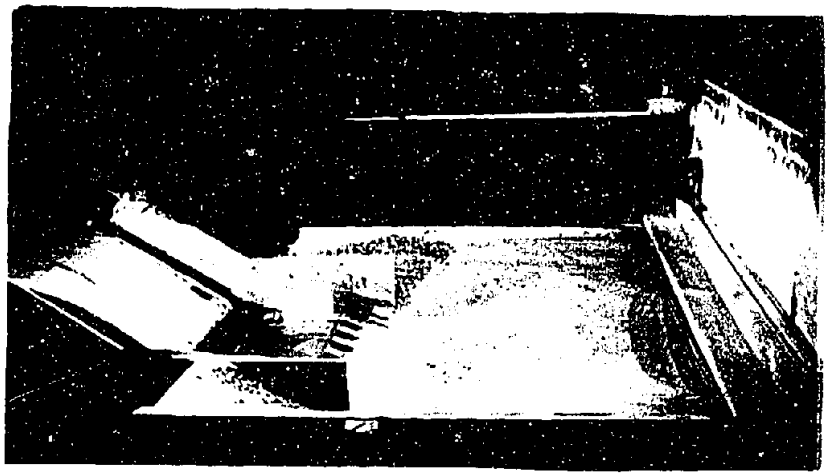


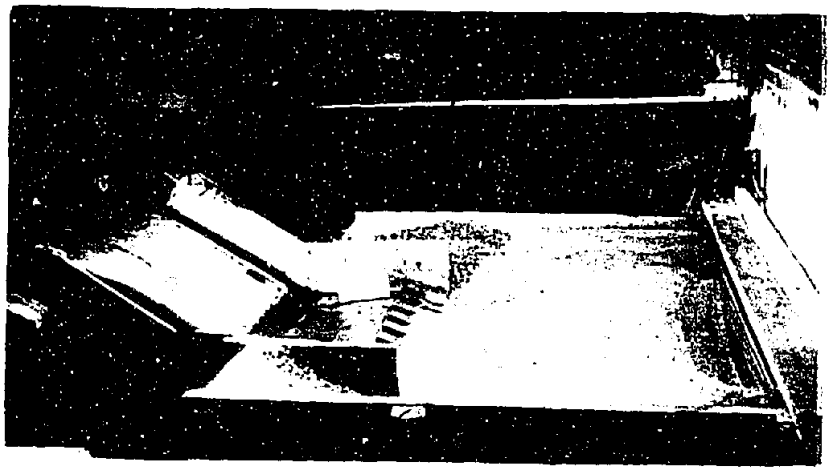
B. CONDITION AT END OF [illegible]

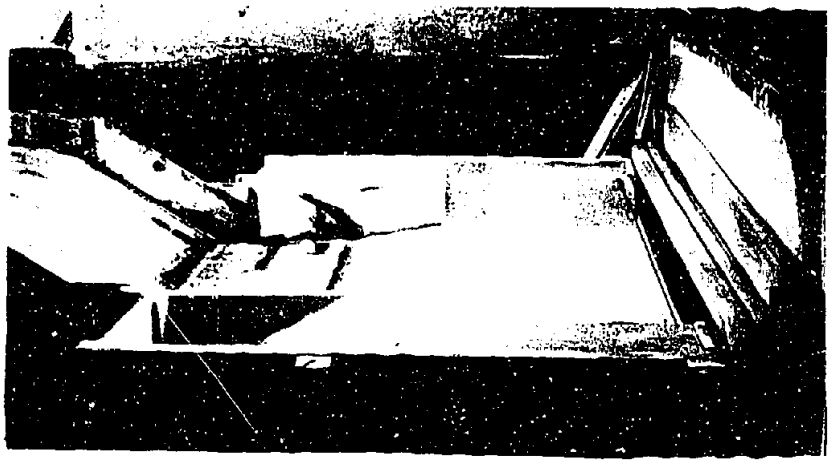


CONDITIONS AT END OF [illegible]

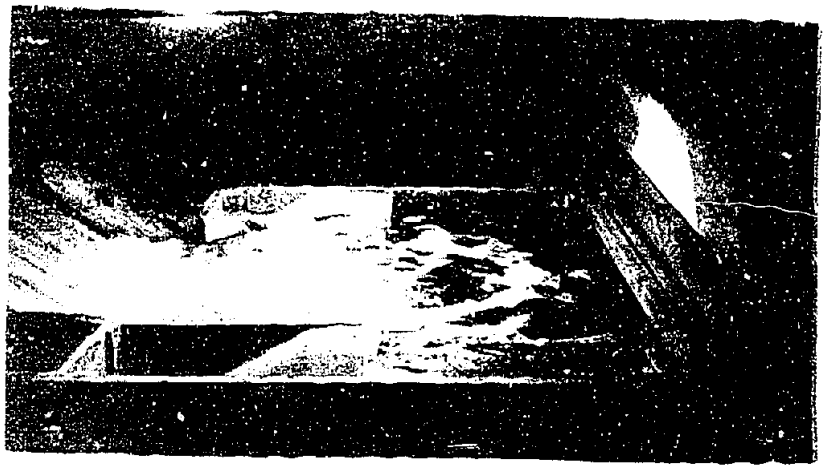
EXPERIMENTS WITH THE [illegible]



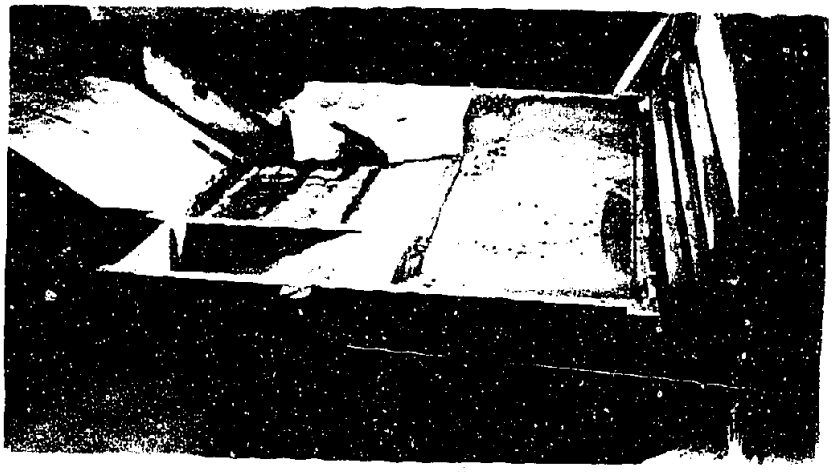




[illegible] BEFORE RUN



[illegible] THE RUN



[illegible]

TESTS WITH ROOF OF ORIGINAL DESIGN

the pool without the sill would still be safe, but it is doubtful if it would be in good condition at the end of a 5-hour run.

In order to be sure that the structure would not be endangered in case the tailwater depth proved to be less than was expected, a run was made with the tailwater depth 4 ft. lower than that assumed in design. The result of this run is shown on Plate XLVI-C. This differs very little from the conditions with the standard tailwater, so that no appreciable effect would be experienced if the tailwater level had been overestimated.

In order to show that the final design would stand up under a high flood of long duration, a run of 5 hours duration was made on the model, corresponding to 35.4 hours on the protytype, with a 40,000 sec. ft. flow. Plates XLVII and XLVIII show the condition of the sand pit before the run and at the end of each hour. At the end of the first hour run there was considerable deposit at the end of the floor and on the riprap bank protection. As flow continued this deposit was gradually washed away leaving at the end of the 5-hour period only a small deposit on the riprap and practically none at the end of the floor, or, in other words, the bottom at the end of the floor had scoured back to its original level. In this condition the pool structure was as safe as before the flood. In order to increase the safety still further however, riprap will be placed across the channel bottom at the end of the floor, as well as on the side slopes. The scour along the center line of the channel is shown graphically on Figure 15 and cross-sections at the end of the pool floor and 100 ft. downstream for the 1-hour and 5-hour periods are shown on Figure 10. They show that at the end of the period some scour was still going on at a distance below the pool, but would not endanger the structure.

Comparison of Original and Final Design.

In order to show the improvement which was obtained from the hydraulic standpoint by means of the model tests, a run was made with a model of the original design under conditions as nearly similar to those of the final design as possible. The results are shown on Plate XLIX and Figure 71. These indicate fairly similar conditions of scour on the banks in the two cases, but shows much more scour on the bottom of the original design at the downstream edge of the pool floor. The scour 100 ft. downstream is much the same for both types. The principal advantage of the final design however is in decreased cost. The length of the pool of the final design is 48 ft. shorter than the original design and the expensive wing walls of the latter are eliminated. These savings together with those made possible by narrowing the trough, represent a saving of several times the entire cost of the laboratory experiments. By means of the laboratory studies therefore it was possible to obtain not only a better and safer structure but a much cheaper one as well.

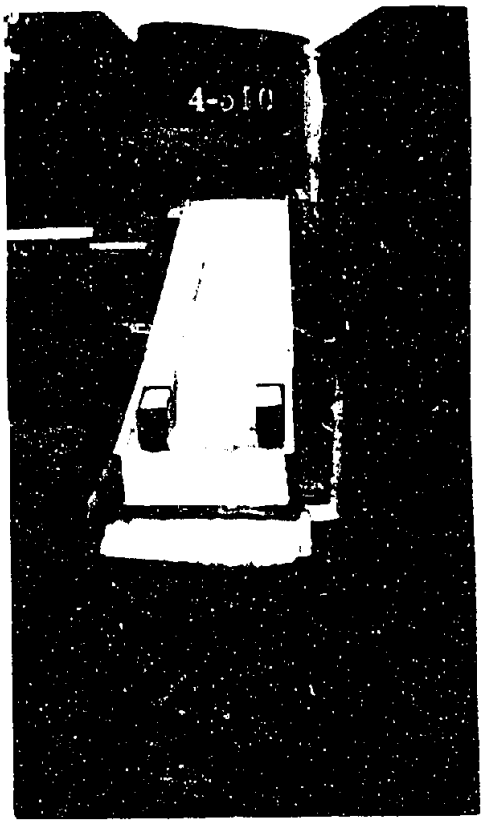
Tests of Small Models and the Law of Similitude

In order to try out several variations in the trough while other experiments on the 1:50 model were in progress, several small models were made on 1:250 scale. The results on some of these are shown on Plates VI, L and LI. A 1:250 model of the original design, as shown on Plate L-A was also made. A Comparison of this with Plate III-A shows that the action of the two models is very similar. The diamond shape wave formation is set up in the same way and in almost exactly the same position in the spillway. Those in the 1:250 spillway however were relatively smaller, due no doubt to the relatively greater effect of surface tension in the smaller model.

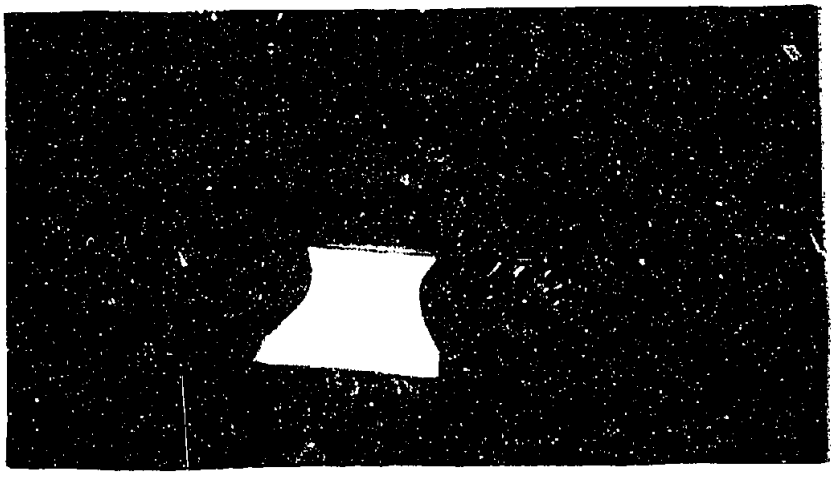
To still further test the law of similitude a 1:500 model was also constructed, as shown on Plate LI-A and B. The wave action in this model was almost entirely absent and did not follow the pattern shown by the 1:50 and 1:250 sizes. Computations show that the Reynolds number at the throat for this size was 2770 which was less than the critical value and since the flow would therefore be in a streamline state, its action would be different. A thin stream of colored fluid introduced through a capilliary tube flowed through the flume in a hair line without breaking up, as shown on Plate LI-B proving that the flow was in a stream line state, as the theory indicated. The 1:250 model however, had a Reynolds number of 7760 which was above the critical value indicating turbulent flow. This was confirmed by the rapid spread of the colored fluid introduced into it, as shown on Plate L-A.

In the 1:500 model the surface tension also had the effect of making the results from it unreliable when applied to the prototype size. As the same fluid would be handled by the prototype as was used in the model the surface tension effects would be of the same magnitude in both, and therefore 500 times greater when compared to the size of the model. A striking example of the unreliability of this small model due to surface tension effect is indicated by the fact that it was possible to make a stream of water about 1/4-inch in diameter run along the flat top of one of the stilling pool walls, held there by the surfact tension. This would indicate a stream of water ten feet in diameter running along on the flat top of a wall, as if it were flowing through an invisible pipe, which is obviously far from the result which would be obtained in the prototype. As previously mentioned, surface tension exerted a considerable effect in the 1:250 model but it is believed that the effect in the 1:50 scale model was negligible. Plates L-C and LI-C show results on small models made for preliminary studies on the Madden Dam, and illustrate the wave action in a short wide trough and in a spillway with the control section at the top of the chute.

A 1:250 SCALE OF ORIGINAL DESIGN TROUGH



B 1:250 SCALE OF ANOTHER TROUGH



C FLOW IN A SHORT W OF TROUGH

TESTS ON SMALL SCALE MODELS

General Application.

Although this memorandum is primarily a report on the hydraulic experiments which led to the particular design adopted for the Cle Elum spillway, one of the strongest reasons for preparing it is to make the information collected on these tests available for studies of similar problems in the future. It will be worthwhile therefore to outline the conditions to which the results of this study are applicable, and certain aspects which may be valuable in considering these results from the standpoint of their bearing on some other problems.

The model scale of 1:50 was selected largely because space limitations prevented a larger scale. Many other scale ratios might have been used and would have shown practically the same results. In the same way, the results of these tests are practically as applicable to a large number of other sizes of spillway. For example, if we consider the model as it was built to be a 1:25 model instead of a 1:50 model it will give the results which may be expected on a structure whose dimensions are just half that of the Cle Elum spillway and whose rated discharge is 7075 sec. ft. Figure 72 shows the principal dimensions of the various spillways and the discharges to which these tests could be directly applied by assuming various model ratios. For conditions approximating any of these sets of values, the results could be applied without further investigation. In most instances however the conditions will be too divergent from those given on Figure 72 and the results will be only partly applicable.

Trough Design

The results obtained by experimentation on the trough will not have as general an application as those obtained on the stilling pool. For a trough of uniform width with entrance conditions similar to those experimented upon, no difficulties need by anticipated. If it is desired however to have a variable width the problem is complex and unless the conditions closely approximate those on Figure 72, model studies will probably be needed. The model studies indicate that for troughs of the shapes shown on Table I a less discharge than those given on Figure 72 will operate satisfactorily and probably also a larger one, up to possibly twice that given. If it is necessary to use a greater discharge or a different slope or length, the results cannot be directly applied.

The experiments did not cover a wide enough field to enable the principles of the design of such flumes to be worked out in detail. A great many things were discovered however which should be valuable to one having a similar problem. The ideas on trough design expressed below are speculative, but are given as an aid to the

preparation of preliminary designs. They should not be used in the design of important structures unless they will later be tested by model experiments.

It appears that the shape of the flume which will work most satisfactorily is intimately related to the laws of wave motion. If a contraction of width is made in the middle of the flume, waves are set up where the contraction of the walls begins, which travel across the flume. If the trough is long enough, they will cross and impinge on the opposite side and be reflected back. The wave set up by the contracting sides seems to move across the flume with a somewhat greater velocity than would be computed for a wave moving at right angles to the direction of flow, using the formula $V = \sqrt{gD}$. This may be due to the fact that the wage motion is somewhat inclined upstream. Apparently the abruptness of the contraction influences the rapidity with which these waves move across, perhaps because it influences the direction of motion of the wave with respect to the center line of the flume. It would be unsafe to assume that the wave can move across the flume at a greater velocity than could occur with a wave moving with a velocity of $\sqrt{gD}$ inclined at the angle with the center line of the flume which would give the greatest resultant cross velocity. Probably the best results are obtained when the wave moves across, intersecting in the center and reaching the opposite side at the lower end of the flume. If the wave does not reach the opposite side, a condition shown on Plate L-C occurs. If the wave reaches the opposite side before the end of the flume is reached the condition is that obtained with the original Cle Elum trough design shown on Figure 4. It could probably be shown by computations whether or not the wave could be made to reach the other side with the depth and velocity conditions assumed in the flume, and thus the limitations of a good solution could be determined. A model study would still be necessary to determine how the contraction should be shaped to give the required cross velocity to the waves.

The experiments showed that the expanding portion was not as sensitive to changes of shape as the contracting portion. For example, with the flume above the throat as for the final design, no difficulty was experienced when the lower end was narrowed to 150 or 120 feet. It therefore appears that any form of expansion in which the waves intersect the side walls at a very acute angle should be satisfactory.

Although a flume in which the width was contracted to the maximum extent above the control section was not applicable to the conditions at Cle Elum, it would probably be in many other cases. As this type would not involve the complicated wave actions, it could be designed much more satisfactorily without model tests than the form developed for Cle Elum. Plate LI-C shows a model of such a type which was developed in connection with preliminary studies for the Madden Dam Spillway.

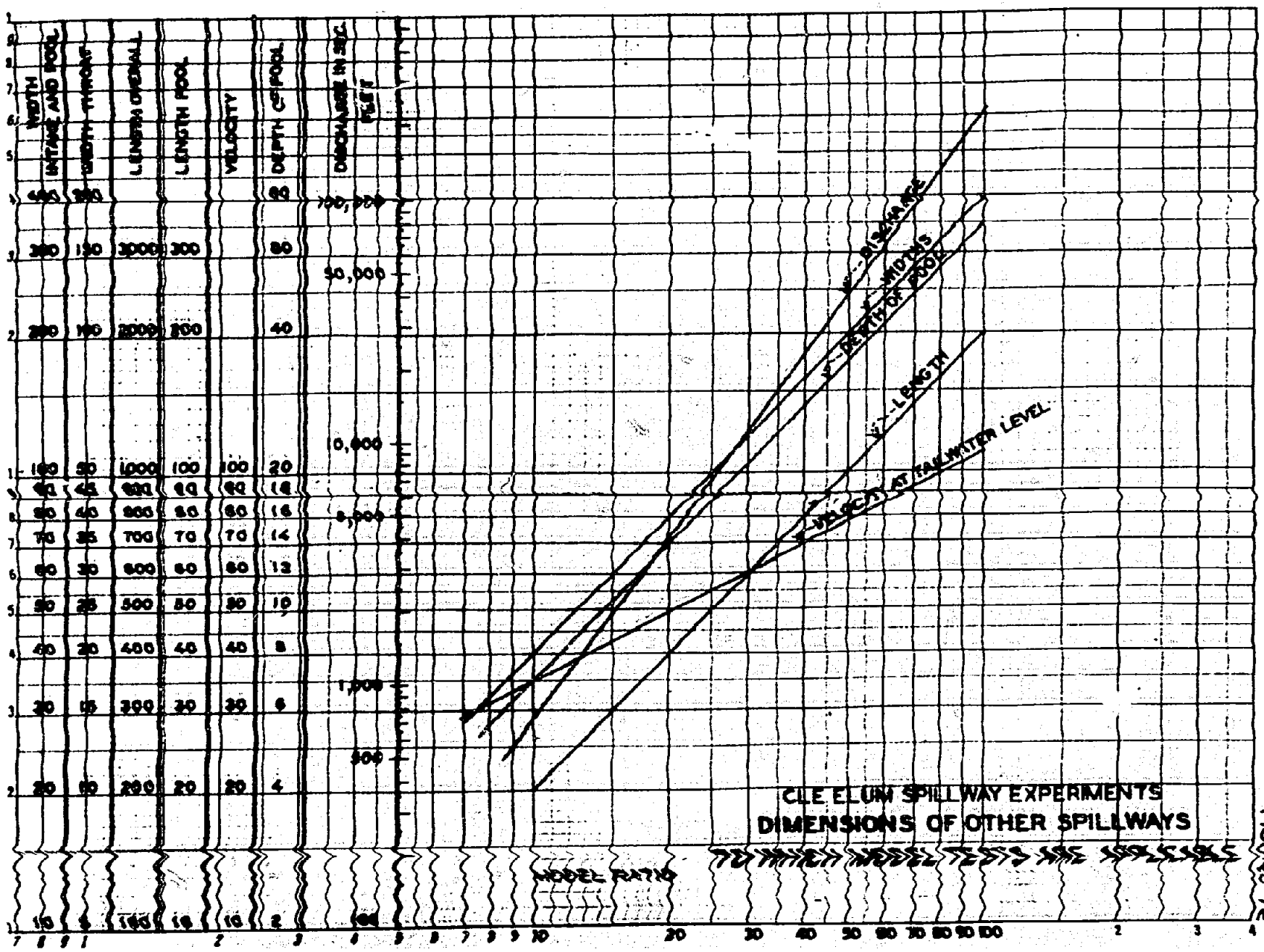
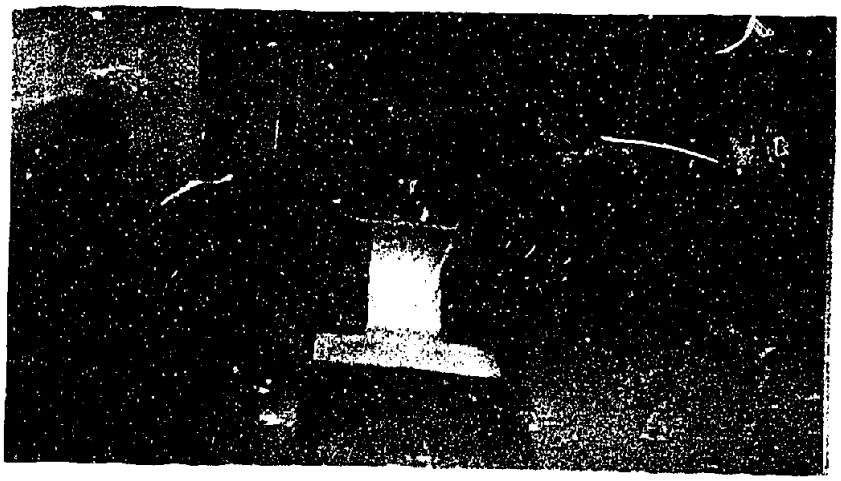

WIDTH INTAKE AND POOL
WIDTH THROAT
LENGTH OVERALL
LENGTH POOL
VELOCITY
DEPTH OF POOL
DISCHARGE IN SEC. FEET
400 200 80
300 150 3000 300 60
200 100 2000 200 40
100 50 1000 100 100 20
90 45 900 90 90 18
80 40 800 80 80 16
70 35 700 70 70 14
60 30 600 60 60 12
50 25 500 50 50 10
40 20 400 40 40 8
30 15 300 30 30 6
20 10 200 20 20 4
10 5 100 10 10 2
100,000
50,000
10,000
5,000
1,000
500
100
DISCHARGE
WIDTHS
DEPTH OF POOL
LENGTH
VELOCITY AT TAILWATER LEVEL
MODEL RATIO
TO WHICH MODEL TESTS ARE APPLICABLE
CLE ELUM SPILLWAY EXPERIMENTS
DIMENSIONS OF OTHER SPILLWAYS

FIGURE 72

A - A 1:500 SCALE MODEL

B - FILAMENT FLOW IN 1:500 MODEL



C - FLOW IN MODEL WITH CONTROL
AT THE THROAT

TESTS ON SMALL SCALE MODEL

Stilling Pool Design

The results of the model tests on the spillway pool have a much wider general application than those on the trough. It would be possible to develop formulas for the dimensions required for the pools, but it would be difficult to indicate the range of these formulas for which their sufficiency was substantiated by the experiments. As the experiments were not sufficiently broad to permit the development of a general formula applicable to all cases, it is believed to be better to indicate the results by diagrams and permit any designer having a problem which cannot be exactly solved by the diagrams to judge how closely it does fit the conditions for which there is experimental data.

On the assumption that the water in the Cle Elum model approached the pool with an equal distribution across its width, and with a direction of flow parallel to the center line of the channel, which is close enough to the truth for practical purposes, a much wider application of the results of the stilling pool experiments can be made. Figure 73 shows the results from the 200, 150 and 120 ft. pool widths reduced to the basis of discharge per foot of width. These diagrams show the depth, and length of the pool required for various discharges per foot of width and velocities and heads at the tailwater level for the entering stream. The pool lengths are measured from the point of intersection of the incoming straight slope with the pool bottom, to the downstream end of the floor downstream from the dentated sill. The length of the pool required could not be determined with exactness but it is believed that the lengths given are conservative. The depth given is the theoretical depth necessary to form the jump. The experiments indicated that for the conditions at Cle Elum a lesser depth could be used, but in order to insure that the depth will be sufficient for other conditions the full theoretical depth is used.

It is believed that these diagrams make the results of these experiments applicable to a wide range of conditions. For example, suppose it is desired to design a spillway for a discharge of 25,000 sec. ft. and a fall of 80 ft. The "velocity head at tailwater level" curve of Figure 73 shows a 60 ft. velocity head for a model ratio of 4^3. With this model ratio, the discharge per foot of width, as determined from the experiments on the 120 ft. pool width, was 220, and for 25000 sec. ft. a pool width of 114 ft. would be required. The depth, for a 43 model ratio, as determined from the 120 ft. pool results is shown to be 36 ft. and the length 99 ft. If it is desired to use a wider pool, the results obtained by experimenting with the 200 ft. pool width could be used, in which case the results would be: discharge per foot of width 140 sec. ft., pool width 179 ft., depth 28 ft., and length 85 ft. Care should be taken not to mix dimensions

determined from the experiments on the various pool widths by using, for example, the width determined from the 200 ft. pool experiments with the depth from the 120 ft. pool tests.

In applying these diagrams to other cases the following facts should be kept in mind. The tailwater rating curve for the model tests was such that there was ample pool depth at all discharges to force the formation of the jump to take place in the pool, and this condition would also be necessary in any installation based on these results. Although the diagrams were developed from tests with the 10 ft. dentated sill, they could also be used with a plain sill corresponding to 10 ft. height at the end of the floor if the tailwater rating curve is such at at all discharges there will be no appreciable fall of the water passing over the plain sill. In other words, the top of the sill must be sufficiently below the channel bottom so that it forms no obstruction at low flows and the pool bottom must therefore be below the channel bottom. A plain sill corresponding in size and location to a 10 ft. sill 25 ft. upstream from the end of the floor might be used also, but this would cause slightly more scour on the bottom and considerably more wave wash on the channel sides. There will be considerable wave wash on the channel sides downstream from the structure in any case and where such erosion would be undesirable, bank protection for a considerable distance downstream will be necessary. The necessary distance would not be determined from these experiments. The results of these experiments are not applicable unless the flow is equally distributed and parallel to the axis when entering the stilling pool. Although the results were derived for a $1\frac{1}{2}$:1 slope leading into the pool, it is believed that they can be used with substantial accuracy for flatter slopes. For steeper slopes the results may not be so satisfactory.

These diagrams were developed primarily for spillways, when floods of the design capacity would be very infrequent and of short duration. In using them for falls in irrigation canals, where the design capacity would be frequently reached and continue for long periods, a cutoff wall and riprap protection at the downstream edge should be used, since the scour over a long period of use would probably be somewhat more severe than the experiments indicated.

Conclusions

The following are the principal conclusions reached from these studies:

In a trough type spillway where the water flows at greater than critical velocity, wave actions are set up which have an important bearing on the capacity. With a trough of uniform width these

are not likely to be serious, but where the width varies they will probably be important. Although rough general dimensions of a satisfactory trough of this type might be determined by computation, unless it was very similar to those tested further model studies would be necessary.

The experiments indicated that very consistent results can be obtained on models in investigating scour by means of a sand bin. For the conditions studied, the greater part of the scour occurred in a comparatively short time, permitting the use of runs of reasonable length.

The tests on the model of the stilling pool indicated that a pool with an upward slope at the downstream end was undesirable. They also show that slopes into the pool of 1½:1 or flatter could be used, the flatter slopes generally being somewhat better from the hydraulic standpoint without sills, but with the dentated sill there appeared to be little difference. A curve at the junction of the slope with the pool floor was found to be desirable.

The dentated sill at the end of the floor was found to be very effective in preventing scour. The shape of the ends of this sill was not important. With the lengths of pool tested, the pressure on the sills were not excessive. The plain sill at the end of the pool floor was also very effective and a plain sill somewhat further upstream was slightly less so. In all the tests of sills the jump formed upstream from it and at all discharges there was a considerable depth of water over the sills. For other conditions the results might have been different.

The best pool width was found to be 200 ft. and length approximately 100 ft., but other widths and lengths could have been used satisfactorily.

The reduction of downward pressure on the pool floor was found to be material and require consideration to prevent the blowing up of the floor.

Small scale models were found to be valuable for preliminary studies and to show general relations. Surface tension and change from turbulent to filament flow limit the size of model and make the results on the larger scales more reliable.

The results on the trough have directly only a small range of application, but by analizing the pool results on the basis of a discharge per foot of width, they may be applied to a large range of conditions.

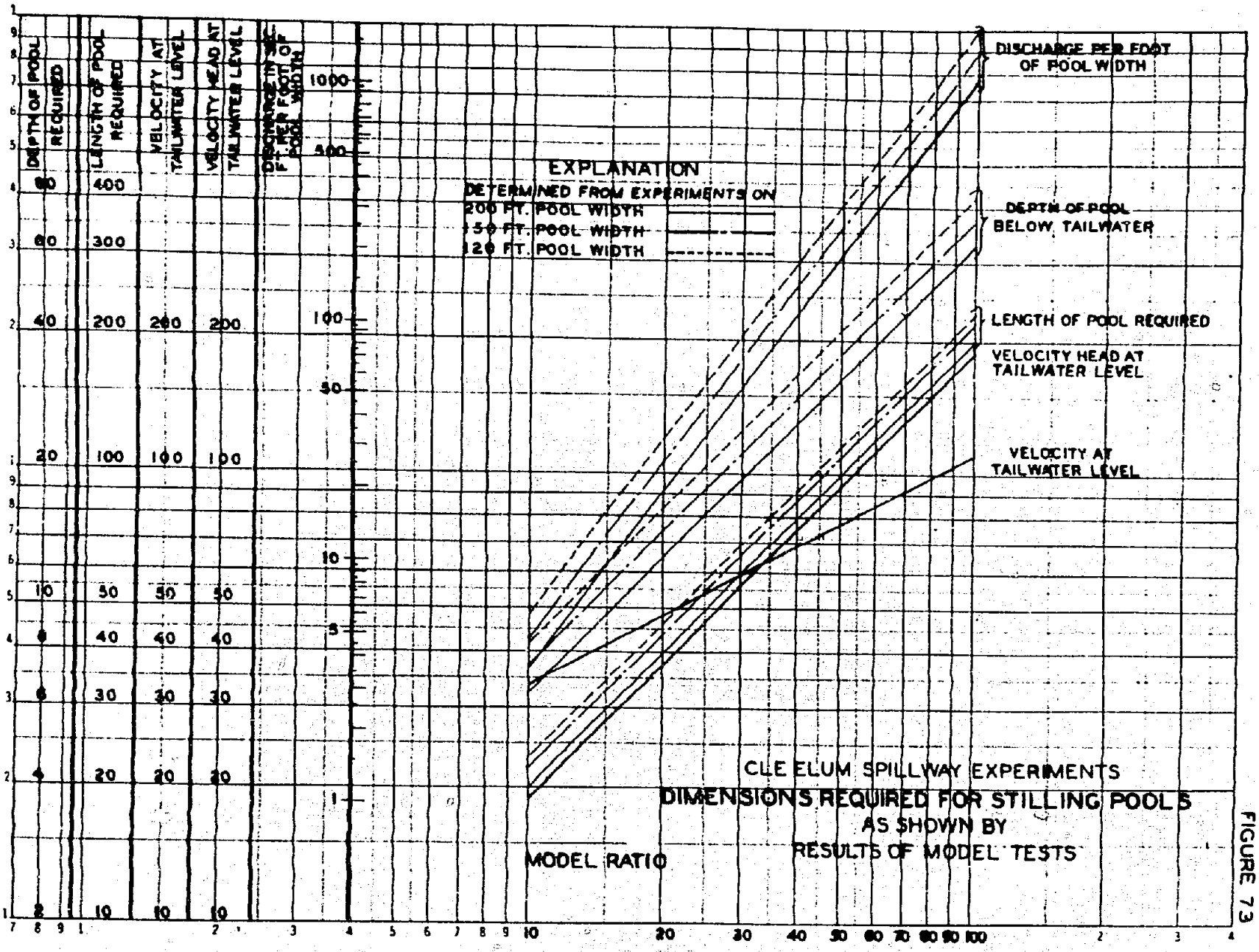

DEPTH OF POOL REQUIRED
LENGTH OF POOL REQUIRED
VELOCITY AT TAILWATER LEVEL
VELOCITY HEAD AT TAILWATER LEVEL
DISCHARGE IN SEC. FT. PER FOOT OF POOL WIDTH
EXPLANATION
DETERMINED FROM EXPERIMENTS ON
200 FT. POOL WIDTH
150 FT. POOL WIDTH
120 FT. POOL WIDTH
DISCHARGE PER FOOT OF POOL WIDTH
DEPTH OF POOL BELOW TAILWATER
LENGTH OF POOL REQUIRED
VELOCITY HEAD AT TAILWATER LEVEL
VELOCITY AT TAILWATER LEVEL
MODEL RATIO
CLE ELUM SPILLWAY EXPERIMENTS
DIMENSIONS REQUIRED FOR STILLING POOLS
AS SHOWN BY
RESULTS OF MODEL TESTS

FIGURE 73

These experiments developed a much better form of spillway with a smaller cost than the original design. The saving in cost alone was several times the cost of the investigation.

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