UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

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HYDRAULIC LABORATORY REPORT HYD. 1.1

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

by

E. W. LANE, RESEARCH ENGINEER

Denver, Colorado,
April 28, 1932
MEMORANDUM TO CHIEF DESIGNING ENGINEER

SUBJECT: RESULTS OF HYDRAULIC MODEL STUDIES ON THE CLE ELUM DAM SPILLWAY.

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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.*
A. THE LABORATORY

B. THE SUPPLY RESERVOIR FOR THE LABORATORY

C. 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

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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 1\(\frac{3}{4}\) 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
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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 headgates 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.
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.

Prototype scale of feet

0 50 100 150

CLE ELUM
SPILLWAY EXPERIMENTS
FLOW IN
TROUGH WITH PARALLEL SIDES
A - FINAL DESIGN 40,000 SEC FT FL
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
A- 40,000 SECOND FEET
ALL GATES OPEN

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

c- RESERVOIR AT MAXIMUM LEVEL
GATES 2, 3 AND 4 OPEN
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 g 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 60 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,
Development of a Trough with Narrow Throat

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Use of Control Section at Throat

The grade of the trough section of the Clo 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 could 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 50, 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 narrower throats 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 these 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
PROFILE OF WATER SURFACE ALONG SIDES
40,000 C.F.S

FINAL DESIGN

ORIGINAL DESIGN

60 FT. THROAT DESIGN

STATIONS ALONG E OF SPILLWAY
STATION 0+00 = AXIS OF DAM

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
Cle Elum Spillway Experiment
Path of Waves Caused by Gate Piers
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 2, 3 AND 4 OPEN

FLOW IN TROUGH WITH 60 FOOT THROAT
A - CONTROL SECTION AT THE THROAT

B - HYDRAULIC JUMP BETWEEN CONTROL SECTIONS

C - FINAL DESIGN WITH BAD GATE COMBINATION
A - 40,000 SECOND FEET
ALL GATES OPEN

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

C - DESIGN WITH 100 FT. TIP/0.01 T
STILLING POOL 120 FT. //BE
40,000 SECOND FEET
A - Gates 1, 3 and 4 wide open
Gates 2 and 5 open 5 ft.

B - Gates 1 and 3 wide open
Gates 2, 4 and 5 open 5 ft.

C - Gates 1 and 3 wide open
Gates 4 and 5 open 5 ft.

Flow in final design trough with bad gate combinations
A-ALL GATES WIDE OPEN

B-RESERVOIR AT MAXIMUM FLOW LINE
ALL GATES OPEN THE SAME AMOUNT

C-SPILLWAY WITHOUT GATES OR PIERS

FLOW IN FINAL DESIGN THROUGH 10,000 SEC. FT.
Cle Elum Spillway Experiments
Mechanical Analysis of Sand
Used to Test Scour

Percentage Passing Through Sieves

Linear Dimension of Sieve Openings in Inches
A-ALL GATES WIDE OPEN

B-RESERVOIR AT MAXIMUM FLOOD LINE
ALL GATES OPEN THE SAME AMOUNT

C-SPILLWAY WITHOUT GATES OR PIERS

FLOW IN FINAL DESIGN TROUGH 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 ample 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
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 permanganate into the stream. 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 these 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 these 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 $V = \sqrt{gD}$, 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 $\sqrt{gD}$ is shown for a discharge of 40,000 second feet to be approximately $0.75 \sqrt{gD}$ and for a discharge of 20,000 second feet, approximately $0.70 \sqrt{gD}$. Experiments by Whitney M. Forland on models of beaches have shown that waves in very shallow water do not reach velocities equal to $\sqrt{gD}$. 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 160,000 second feet. This showed (Figure 10) velocities practically equal to \( \sqrt{\frac{g}{D}} \). 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 \( \sqrt{\frac{g}{D}} \) and in the model at the design capacity reached only 0.75 \( \sqrt{\frac{g}{D}} \), the results of this study throw some doubts on the sufficiency of the determinations of the boat 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 \( \sqrt{\frac{g}{D}} \) and those in the prototype which would probably be equal to the \( \sqrt{\frac{g}{D}} \) 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 \( \sqrt{\frac{g}{D}} \). Such models might be cheaply constructed at the laboratory on the Uncomphreg 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 is 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 be-
low 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池, and extensive experi-
ments were undertaken on this portion of the spillway. The effective-
ness of the stilling pool was determined by means of the scour pro-
duced 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 mechan-
ical 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
plato glass. The width of the pool and sand box was 4.0 ft., cor-
responding 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. down-
stream 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 201 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, tak-
ing 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 par-
tially 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 investi-
gate the scour around the ends of the stilling pool walls, a larger
sand bin was constructed in which the sloping earth banks of the pro-
totype 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
CLE ELUM SPILLWAY EXPERIMENTS

VELOCITIES OF TRANSVERSE WAVES
IN TROUGH FOR VARIOUS DEPTH OF FLOW

Q = 7.93 SEC FT ON MODEL OR
140,300 SEC FT ON PROTOTYPE

Q = 2.26 SEC FT. ON MODEL OR
40,000 SEC FT. ON PROTOTYPE

Q = 1.13 SEC FT. ON MODEL OR
20,000 SEC FT. ON PROTOTYPE

V = \sqrt{gD}

V = 0.9 \sqrt{gD}

V = 0.8 \sqrt{gD}

V = 0.7 \sqrt{gD}

V = 0.6 \sqrt{gD}

TRANSVERSE VELOCITY V IN FT. PER SEC IN MODEL

DEPTH OF WATER "D" IN FEET IN MODEL
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 the 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 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 3.14 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 tests, 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
EXPLANATION

- - - - - - - - - - -
SAND BEFORE RUN

- - - - - - - - - - -
SCOUR LOOSELY PACKED SAND

- - - - - - - - - - -
SCOUR NORMALLY PACKED SAND

- - - - - - - - - - -
SCOUR TIGHTLY PACKED SAND

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

0 25 50
Prototype scale of feet
EXPLANATION

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

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

STATIONS ALONG E OF SPILLWAY

DEPTH 30 MIN. RUN
DEPTH 60 MIN. RUN
DEPTH 90 MIN. RUN
DEPTH 120 MIN. RUN

PROTOTYPE SCALE OF FEET

0 25 50

0
EXPLANATION

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

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS

EFFECT OF RUNS OF VARIOUS LENGTHS
ON THE FINAL DESIGN

FLOOR EL 2097

STATIONS ALONG EL OF SPILLWAY

10+00 10+25 10+50 11+00 11+25 11+50

9+75

PROTOTYPE SCALE OF FEET

0 25 50
EXPLANATION

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

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

0 25 50

PROTOTYPE SCALE OF FEET

STATIONS ALONG E OF SPILLWAY

ORIGINAL DESIGN

60 FT. THROAT DESIGN
EXPLANATION

- Sand before run
- Straight sides - 200ft. width
- Curved sides - 100ft. throat
- Curved sides - 60ft. throat

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

PROTOTYPE SCALE OF FEET

0 25 50
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 flumes 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 those 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. Rohbock 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 19 (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 standpoint 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 riverted 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 (1/2:1 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 25) 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 leadoff channel, the question arose as to whether the transition from the floor level to channel level could be made abruptly 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 these tests are shown on Plates XX and XXII 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.
EXPLANATION

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

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

STATIONS ALONG % OF SPILLWAY

9+50 10+00 11+00

0 25 50

PROTOTYPE SCALE OF FEET
Figure 19

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

EXPLANATION

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

+25 +50
STATIONS ALONG SPILLWAY

0 25 50
PROTOTYPE SCALE OF FEET
SCOUR WITH NORMAL TAILWATER LEVEL

SCOUR WITH LOWERED TAILWATER LEVEL

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

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF TAILWATER HEIGHT ON SCOUR

STATIONS ALONG E OF SPILLWAY

0 25 50
PROTOTYPE SCALE OF FEET
EXPLANATION

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

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

FIGURE 21
EXPLANATION

- DASHED - SAND BEFORE RUN
- LIGHT DASH - EXPANSION ENDS AT STA. 7+84 FIRST FORM
- MEDIUM DASH - EXPANSION ENDS AT STA. 9+00 SECOND FORM
- DARK DASH - EXPANSION ENDS AT STA. 9+98 THIRD FORM
- THICK DASH - EXPANSION ENDS AT STA. 9+96 FOURTH FORM

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF EXPANSION ENDING AT DIFFERENT POINTS
FLUME SLOPE 1/2:1 THROAT 100FT.

AVERAGE DEPTH OF SCOUR
MAXIMUM DEPTH OF SCOUR

STATIONS ALONG E OF SPILLWAY

0 25 50
Prototype Scale of Feet
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF EXPANSION ENDING
AT DIFFERENT POINTS
FLUME SLOPE 1½:1 THROAT 100FT.
EXPLANATION

- Sand before run
- Scour 1:1 slope
- Scour 2:1 slope
- Scour 3:1 slope
- Scour 4:1 slope

AVERAGE DEPTH OF SCOUR

MAXIMUM DEPTH OF SCOUR

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

STATIONS ALONG % OF SPILLWAY

0 25 50

PROTOTYPE SCALE OF FEET
EXPLANATION

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

STATIONS ALONG E' OF SPILLWAY

AVERAGE DEPTH OF SCOUR

MAXIMUM DEPTH OF SCOUR

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

0 25 50
PROTOTYPE SCALE OF FEET
EXPLANATION

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

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

PROTOTYPE SCALE OF FEET
A - BEFORE RUN

B - DURING RUN

C - AFTER RUN

SCOUR WITH ORIGINAL POOL AND TROUGH DESIGN
SCOUR WITH ORIGINAL POOL DESIGN AND 60 FOOT TROUGH THROAT
A - BEFORE RUN

B - DURING RUN

C - AFTER RUN

SCOUR WITH DENTATED SILL AT TOP OF 1:4 SLOPING FLOOR OF ORIGINAL POOL DESIGN
SCOUR WITH PLAIN SILL AT TOP OF 1:4 SLOPING FLOOR OF ORIGINAL POOL DESIGN
SCOUR WITH DENTATED SILL AT TOP OF 1:1 SLOPE OF POOL FLOOR
SCOUR WITH PLAIN SILL AT TOP OF 1:1 SLOPE OF POOL FLOOR
A-BEFORE RUN

B-DURING RUN

C-AFTER RUN

SCOUR WITH PLAIN SILL IN STREAM
SCOUR WITH LEVEL POOL BOTTOM AND DENTATED SILL
SCOUR WITH LEVEL POOL BOTTOM AND PLAIN SILL AT END OF FLOOR
SCOUR WITH LEVEL POOL BOTTOM AND PLAIN SILL UPSTREAM FROM END OF FLOOR
A-BEFORE RUN

b-DURING 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 dented 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

AVERAGE DEPTH OF SCOUR

MAXIMUM DEPTH OF SCOUR

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

PROTOTYPE SCALE OF FEET
A-EXPANDING SIDES ENDING AT
TOP OF SLOPE

B-EXPANDING SIDES ENDING AT
BOTTOM OF SLOPE

C-SIDES EXPANDING UNIFORMLY
TO END OF POOL

D-SIDES EXPANDING NON-UNIFORMLY
TO END OF POOL
A-EXPANDING SIDES ENDING AT TOP OF SLOPE

B-EXPANDING SIDES ENDING AT BOTTOM OF SLOPE

c-SIDES EXPANDING UNIFORML y TO END OF POOL

D-SIDES EXPANDING NON-UNIFORMLy TO END OF POOL
A - 1/2 I SLOPE - DURING RUN

B - 1/2 I SLOPE - AFTER RUN

C - 2 I SLOPE - DURING RUN

D - 2 I SLOPE - AFTER RUN
A - 3:1 SLOPE - DURING RUN

B - 3:1 SLOPE - AFTER RUN

C - 4:1 SLOPE - DURING RUN

D - 4:1 SLOPE - AFTER RUN
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

A - 2:1 SLOPE

B - 3:1 SLOPE

C - 4:1 SLOPE
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 1:5:1 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 those 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 those two set-ups, as shown on Figure 22. Those 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 1/2 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, tests were made with sand bins 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 in Figure 37. The first two sets of cross sections show 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 10 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, piezomotors were installed at various points in it as shown on Figure 38 and observations of the pressures at those 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
EXPLANATION

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

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

STATIONS ALONG E OF SPILLWAY

AVERAGE DEPTH OF SCOUR

MAXIMUM DEPTH OF SCOUR

SAND BEFORE RUN
CURVE AT BOTTOM OF SLOPE
NO CURVE AT BOTTOM

0 25 50
PROTOTYPE SCALE OF FEET
EXPLANATION

- - - - - - - - SAND BEFORE RUN

- - - - - CURVE AT BOTTOM OF SLOPE

- - - - NO CURVE AT BOTTOM

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

AVERAGE DEPTH OF SCOUR

MAXIMUM DEPTH OF SCOUR

STATIONS ALONG E OF SPILLWAY

9+50  10+00  +25  +50  +75  11+00  +25  11+50

0  25  50

PROTOTYPE SCALE OF FEET
EXPLANATION

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

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

STATIONS ALONG % OF SPILLWAY

0 25 50

PROTOTYPE SCALE OF FEET

RUN 8
SCOUR WITH DENTATED SILL

RUN 11
SCOUR WITH NO SILL
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF DENTATED SILL ON SCOUR
TROUGH SLOPE 2:1

EXPLANATION

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

0 25 50

PROTOTYPE SCALE OF FEET
EXPLANATION

--- SAND BEFORE RUN

AV. DEPTH  60 MIN. RUN

MAX. DEPTH  60 MIN. RUN

0  25  50

STATIONS ALONG E OF SPILLWAY

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

STATIONS ALONG E OF SPILLWAY

9+50  +75  10+00  +25  +50  +75  11+00  +25  11+50

SCOUR WITH NO SILL

SCOUR WITH DENTATED SILL

RAN 21

RAN 22
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF SIZE OF DENTATED SILL
TROUGH SLOPE 1½:1, SET UP B

EXPLANATION

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

STATIONS ALONG E OF SPILLWAY

SCOUR WITH 7.5FT. DENTATED SILL
CORRECTED

SCOUR WITH 10FT. DENTATED SILL

SCOUR WITH 12.5FT. DENTATED SILL

0 25 50
PROTOTYPE SCALE OF FEET
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
TYPES OF DENTATED SILLS INVESTIGATED
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
TYPES OF DENTATED SILLS INVESTIGATED
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

CLE ELUM SPILLWAY TESTS
COMPARATIVE SCOUR
DENTATED SILL - VARIOUS TYPES
LOCATION OF PIEZOMETERS

SECTION A

SECTION B

TAILWATER LEVEL

SILL WITH DOWNSTREAM END
10+00

SILL WITH DOWNSTREAM END
10+12\(\frac{1}{2}\)

SILL WITH DOWNSTREAM END
10+25

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
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
of these observations are shown on Figure 38. The greatest pressures were on the upstream side of the dentates, but they were not as large as might be expected, the maximum being less than 15 feet of water in excess of that due to the tailwater height. The results show decreasing pressures as the sill is moved downstream due to the lesser velocity of the impinging water in these cases. The measurements indicate a downward pressure on the floor just upstream from the sill, due no doubt to the reaction of the force causing the deflection of the water upward and over the sill. This can be used to advantage to increase the stability of the sill, by making the sill and floor in one piece. The pressures between the teeth and on the downstream side of the teeth were less than tailwater pressure. The results of these measurements indicated that no difficulty need be experienced in designing a sill stable against these water pressures.

Study of the Plain Sill

The plain sill was intended to represent a weir of ogee cross section. In most cases the model was somewhat thinner than the weir would be in the prototype but it is not believed that this had any effect on the results. A comparison of the form ordinarily used with an ogee section was made. At the downstream edge of the floor the ogee sill seemed to give slightly better results and 100 ft. downstream the thinner sill seemed better. Neither of these differences was important and it is believed that they will give substantially the same results.

The first plain sills tested were located 25 ft. upstream from the lower end of the pool floor in order to provide an apron for any water which might fall after passing over the sill. Tests under a great many conditions demonstrated that such a sill would be very effective in reducing scour at the downstream edge of the floor. Figures 39 to 42 inclusive show several comparisons of scour with and without a plain sill. The first of these was made with a flume with a 60 ft. throat and 1\(\frac{1}{2}:1\) slope entering the pool. The remainder were made with a trough 200 ft. wide throughout, and entering slop ews as indicated. In all cases with the sill there was very little scour immediately at the end of the apron, except along the sides of the flume, but without the sill there was considerable scour. Figure 39 shows however that for one setup there was appreciable scour near the downstream edge of the floor, although probably not enough to undermine it.

It was later found that somewhat better results were obtained by placing the sill at the downstream edge of the floor. In this position the water passing over the sill in most cases set up an eddy on the bottom which moved sand upstream and deposited it against the back side of the sill. A comparison of the two sill
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 1:11 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. 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 these 
tests it was decided that the sill should be small, having only suf-
ficient 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 ne-
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 down-
stream edge of the floor, which gave this critical point better pro-
tection 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
SCOUR WITH PLAIN SILL

SCOUR WITH NO SILL

EXPLANATION

--- SAND BEFORE RUN

AV. DEPTH 30 MIN. RUN

AV. DEPTH 60 MIN. RUN

MAX. DEPTH 30 MIN. RUN

MAX. DEPTH 60 MIN. RUN

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

STATION ALONG £ OF SPILLWAY

0  25  50

PROTOTYPE SCALE OF FEET
SCOUR WITH NO SILL

SCOUR WITH PLAIN SILL

STATIONS ALONG E. OF SPILLWAY

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

STATIONS ALONG E OF SPILLWAY

EXPLANATION

--- SAND BEFORE RUN

- AV. DEPTH 60 MIN. RUN

- MAX. DEPTH 60 MIN. RUN

SCOUR WITH NO SILL

SCOUR WITH PLAIN SILL

RUN 21

RUN 23
SCOUR WITH NO SILL

SCOUR WITH PLAIN SILL

EXPLANATION
- - - - SAND BEFORE RUN
- - - - A.V. DEPTH 60 MIN. RUN
- - - - MAX. DEPTH 60 MIN. RUN

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF PLAIN SILL
TROUGH SLOPE 4:1
SILL 25FT. FROM END OF FLOOR

SILL AT END OF FLOOR

EXPLANATION

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

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
EFFECT OF LOCATION OF PLAIN SILL

FIGURE 43
PLAIN SILL 25FT. FROM END OF FLOOR

PLAIN SILL AT END OF FLOOR

RUN 26

RUN 28

STATIONS ALONG & OF SPILLWAY

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
SCOUR WITH 7.5 FT. PLAIN SILL

SCOUR WITH 10 FT. PLAIN SILL

SCOUR WITH 12.5 FT. PLAIN SILL

EXPLANATION

- DOTTED - SAND BEFORE RUN
- THICK LINE - AVG DEPTH 60 MIN RUN
- THIN LINE - MAX DEPTH 60 MIN RUN

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

PROTOTYPE SCALE OF FEET
CROSS SECTION AT END OF POOL FLOOR

CROSS SECTION 87.5 FT. DOWNSTREAM FROM END OF POOL FLOOR

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
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 III 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 56). 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 XXVIII 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 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 evaluating 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.
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS

10FT. DENTATED VS. 10FT. PLAIN SILL
EXPANSION ENDS AT STA. 7+84
FLUME SLOPE 1 1/2:1 THROAT 100FT.

EXPLANATION

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

0 25 50

PROTOTYPE SCALE OF FEET

STATIONS ALONG E. OF SPILLWAY

SCOUR WITH 10FT. DENTATED SILL

SCOUR WITH 10FT. PLAIN SILL

9+50 10+00 11+00 11+50

+75 7+50 11+00 11+50
SCOUR WITH 10FT. DENTATED SILL

SCOUR WITH 10FT. PLAIN SILL

STATIONS ALONG % OF SPILLWAY

EXPLANATION

- - - - - SAND BEFORE RUN

AV. DEPTH 60 MIN. RUN

MAX. DEPTH 60 MIN. RUN

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS

10FT. DENTATED VS. 10FT. PLAIN SILL

EXPANSION ENDS AT STA. 9+00

FLUME SLOPE 1:21 THROAT 100FT.

FIGURE 50

0 25 50

PROTOTYPE SCALE OF FEET
SCOUR WITH DENTATED SILL

SCOUR WITH PLAIN SILL

EXPLANATION

- DENTATED VS. 10 FT. PLAIN SILL
- EXPANSION ENDS AT STA 9+96
- FLUME SLOPE 1:1.5 SET UP 'A'

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS

STATIONS ALONG \( E \) OF SPILLWAY

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 1:2.5
SET UP "B"

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
CURVE AT BOTTOM OF SLOPE
FLUME SLOPE 1:2:1 THROAT 100FT.

EXPLANATION

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

0  25  50
PROTOTYPE SCALE OF FEET
EXPLANATION

- Sand before run
- Avg. depth 60 min. run
- Max. depth 60 min. run

CLE ELUM SPILLWAY TESTS
Stilling pool experiments
7.5 ft. dented vs. 7.5 ft. plain sill
Expansion ends at sta. 9+96
Flume slope 1/2:1 Set up "B"

Prototype scale of feet

Scour with 7.5 ft. dented sill
Scour with 7.5 ft. plain sill

Stations along % of spillway
EXPLANATION

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

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

STATIONS ALONG % OF SPILLWAY

SCOUR WITH 12.5FT. DENTATED SILL
SCOUR WITH 12.5FT. PLAIN SILL
SCOUR AT END OF FLOOR

SCOUR 100 FT FROM END OF FLOOR

EXPLANATION

- - - - BOTTOM BEFORE RUN
- - - - - - - - BOTTOM AFTER RUN 10 FT PLAN SILL
- - - - - - - - - - BOTTOM AFTER RUN 10 FT DENTATED SILL

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

VERTICAL SCALE OF FEET

0 25 50

HORIZONTAL SCALE OF FEET

0 25 50 75 100
EXPLANATION

- --- BOTTOM BEFORE RUN
- --. .- BOTTOM AFTER RUN 10 FT PLAN SILL
- --- --- BOTTOM AFTER RUN 10 FT DENTATED SILL

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

VERTICAL SCALE OF FEET
0 25 50

HORIZONTAL SCALE OF FEET
0 25 50 75 100
CLE ELUM SPILLWAY EXPERIMENTS
COMPARATIVE SCOUR WITH POOLS OF
VARIOUS LENGTHS, DEPTHS AND WIDTHS
SECTION ON CENTER LINE

SECTION ON CENTER LINE

SECTION ON CENTER LINE

SECTION ON CENTER LINE

PLANT

END WALL I

PLAN

END WALL III

PLAN

ENDWALL V FINAL DESIGN

PLAN

END WALL VII

SECTION ON CENTER LINE

SECTION ON CENTER LINE

SECTION ON CENTER LINE

TYPICAL END VIEW

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

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
CLE ELUM SPILLWAY EXPERIMENTS

COMPARISON OF SCOUR WITH VARIOUS LENGTHS AND DEPTHS OF STILLING POOL

120 FOOT POOL WIDTH
TYPES OF END WALLS TESTED

TYPE I

TYPE II

TYPE IV
TYPE V - DESIGN RECOMMENDED

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 art 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 1:1 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 180 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
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
DOWNWARD PRESSURE CURVE
FOR VARIOUS TAILWATER LEVELS
40,000 c.f.s. DISCHARGE
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
DOWNWARD PRESSURE CURVES
FOR VARIOUS TAILWATER LEVELS
17,600 cfs DISCHARGE
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
STATIONS ALONG E OF SPILLWAY

3:1 SLOPE

PIEZOMETERS

1 & 2
3 & 4
5 & 6
7 & 14

CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
DOWNWARD PRESSURE CURVE
FOR DIFFERENT SILLS - 3:1 SLOPE
40,000 c.f.s. DISCHARGE
CLE ELUM SPILLWAY TESTS
STILLING POOL EXPERIMENTS
DOWNWARD PRESSURE CURVES
FOR DIFFERENT SILLS - 4:1 SLOPE
40,000 c.f.s. DISCHARGE
CLE ELUM SPILLWAY TESTS

STILLING POOL EXPERIMENTS

COMPARISON OF AVERAGE DOWNWARD PRESSURE FOR DIFFERENT FLUME SLOPES

40,000 c.f.s. DISCHARGE

EXPLANATION

- CURVE FOR 2:1 SLOPE
- CURVE FOR 3:1 SLOPE
- CURVE FOR 4:1 SLOPE

STATIONS ALONG % OF SPILLWAY

PROTOTYPE ELEVATIONS IN FEET

SLOPE 2:1
SLOPE 3:1
SLOPE 4:1
CLE ELUM SPILLWAY EXPERIMENTS
DOWNWARD PRESSURE ON POOL BOTTOM
150 FT POOL WIDTH
CLE ELUM SPILLWAY EXPERIMENTS
DOWNWARD PRESSURE ON POOL BOTTOM
120 FT. POOL WIDTH
FIGURE 69

CROSS SECTION AT END OF POOL FLOOR

CROSS SECTION 100 FT. DOWNSTREAM FROM END OF POOL FLOOR

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

CLE ELUM SPILLWAY EXPERIMENTS
SCOUR WITH VARIOUS DISCHARGES
FINAL DESIGN
CROSS SECTION AT END OF POOL FLOOR

CROSS SECTION 100 FT. DOWNSTREAM FROM END OF POOL FLOOR

SPILLWAY BOTTOM

- - - - BEFORE RUN

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

\(\times\) -- END OF RUN 5 HR. MODEL - 35.4 HR. PROTOTYPE

VERTICAL SCALE OF FEET

0 25 50

HORIZONTAL SCALE OF FEET

0 25 50 75 100

CLE ELUM SPILLWAY EXPERIMENTS

EFFECT OF LENGTH OF RUN ON SCOUR

FINAL DESIGN
FIGURE 71

CROSS SECTION AT END OF POOL FLOOR

CROSS SECTION 100 FT. DOWNSTREAM FROM END OF POOL FLOOR

SPILLWAY BOTTOM
- - - BEFORE RUN
- - X - 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 - SHOWING DEFLECTOR OVER TUNNEL OUTLET

b - SHOWING DEFLECTOR IN ACTION

STUDY OF TUNNEL OUTLET
A - WITHOUT THE SILL, BEFORE THE RUN

B - WITHOUT THE SILL, AFTER THE RUN

C - SCOUR WITH A LOWERED TAILWATER

VARIous TESTS ON THE FINAL DESIGN
A - Sand pit before the run

B - Conditions at end of 1 hr. run

C - Conditions at end of 2 hr. run

Experiments with flows of long duration
A - CONDITIONS AT END OF 3 HR. RUN

B - CONDITIONS AT END OF 4 HR. RUN

C - CONDITIONS AT END OF 5 HR. RUN

EXPERIMENTS WITH FLOWS OF LONG DURATION
A-C. CONDITIONS BEFORE RUN

B-DURING THE RUN

C-AFTER THE RUN

TESTS WITH POOL 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 25.4 hours on the prototype, 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 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 1/2 ft. 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 capillary 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 streamline 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 surface tension. This would indicate a stream of water ten feet in diameter running along 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 WIDE 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 in 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 those 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 \[ V = \sqrt{\frac{gD}{g}} \]. This
may be due to the fact that the wave motion is somewhat inclined up-
stream. Apparently the abruptness of the contraction influences the
rapidity with which these waves move across, perhaps because it in-
fluences the direction of motion of the wave with respect to the cen-
ter 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 \[ \sqrt{\frac{gD}{g}} \] 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 wheth-
er 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 lim-
itations 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 ex-
ample, with the flume above the throat as for the final design, no
difficulty was experienced when the lower end was narrowed to 150 or
180 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.
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 80 ft. velocity head for a model ratio of 43. 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 that 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 those 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 1:3:1 slope loading 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 the
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 1\(\frac{1}{3}\):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 analyzing the pool results on the basis of a discharge per foot of width, they may be applied to a large range of conditions.
FIGURE 73

CLE ELUM SPILLWAY EXPERIMENTS

DIMENSIONS REQUIRED FOR STILLING POOL 5

AS SHOWN BY
RESULTS OF MODEL TESTS

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

EXPLANATION
DETERMINED FROM EXPERIMENTS ON
200 FT. POOL WIDTH
150 FT. POOL WIDTH
120 FT. POOL WIDTH

200 200 200
150 150 150
120 120 120

90 90 90
80 80 80
70 70 70
60 60 60
50 50 50
40 40 40
30 30 30
20 20 20
10 10 10

4 4 4 4 4 4
3 3 3 3 3 3
2 2 2 2 2 2
1 1 1 1 1 1

100 100 100 100 100 100
50 50 50 50 50 50
20 20 20 20 20 20
10 10 10 10 10 10

5 5 5 5 5 5
4 4 4 4 4 4
3 3 3 3 3 3
2 2 2 2 2 2
1 1 1 1 1 1

1 1 1 1 1 1
0 0 0 0 0 0

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.