UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

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Technical Memorandum No. 323

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# PROTECTION AGAINST SCOUR BELOW OVERFALL DAMS

by

E. W. LANE, RESEARCH ENGINEER,
and
W. F. BINGHAM, JUNIOR ENGINEER

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Denver, Colorado
Jan. 23, 1933

Hyd-L.4

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MEMORANDUM TO CHIEF DESIGNING ENGINEER

SUBJECT: PROTECTION AGAINST SCOUR BELOW OVERFALL DAMS

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By E. W. LANE, RESEARCH ENGINEER

and

W. F. BINGHAM, JUNIOR ENGINEER

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Under direction of

J. L. SAVAGE, CHIEF DESIGNING ENGINEER

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(Price $0.85)

A great deal of study has been given to determining the best method of protecting the stream bed below overfall dams against scour resulting from the high velocity and impact of the water flowing over them. A great variety of local conditions have been encountered which has led to many different solutions, descriptions of which are widely dispersed through engineering literature. From a study of those articles and the results of hydraulic laboratory studies on dam spillways performed by the U. S. Bureau of Reclamation, the following analysis has been developed by the writers, which classifies the various conditions and points out the general types of scour protection applicable to each.

Scour below dams results from the erosive power of the water which comes into contact with the stream bottom while moving with the high velocity which it acquires in falling over the dam. Protection is afforded by reducing the velocity of the water, by insuring that the high velocity flows do not come in contact with the bottom, or are diverted to portions of the bottom which will not endanger the structure. Usually a combination of these is used.

On many dams in the past an attempt has been made to reduce the velocity of the water passing over them by having the water fall down a series of steps on the face of the dam, dissipating its energy by contact with the successive steps and reaching the bottom with insufficient energy remaining to seriously erode the stream bed.

Probably the outstanding example of this type is the Gilboa Dam* of

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*Trans., Am. Soc. C. E., p. 280, Vol. 86, 1923.

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the water supply system of New York City (Plate 1-A). While the results on this dam have probably been successful, due to the carefully conducted model tests, without such studies the action on stepped weirs is apt to be different from that anticipated,"" and as the

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**Civil Engineer, p. 623, Vol. 2, October, 1932.

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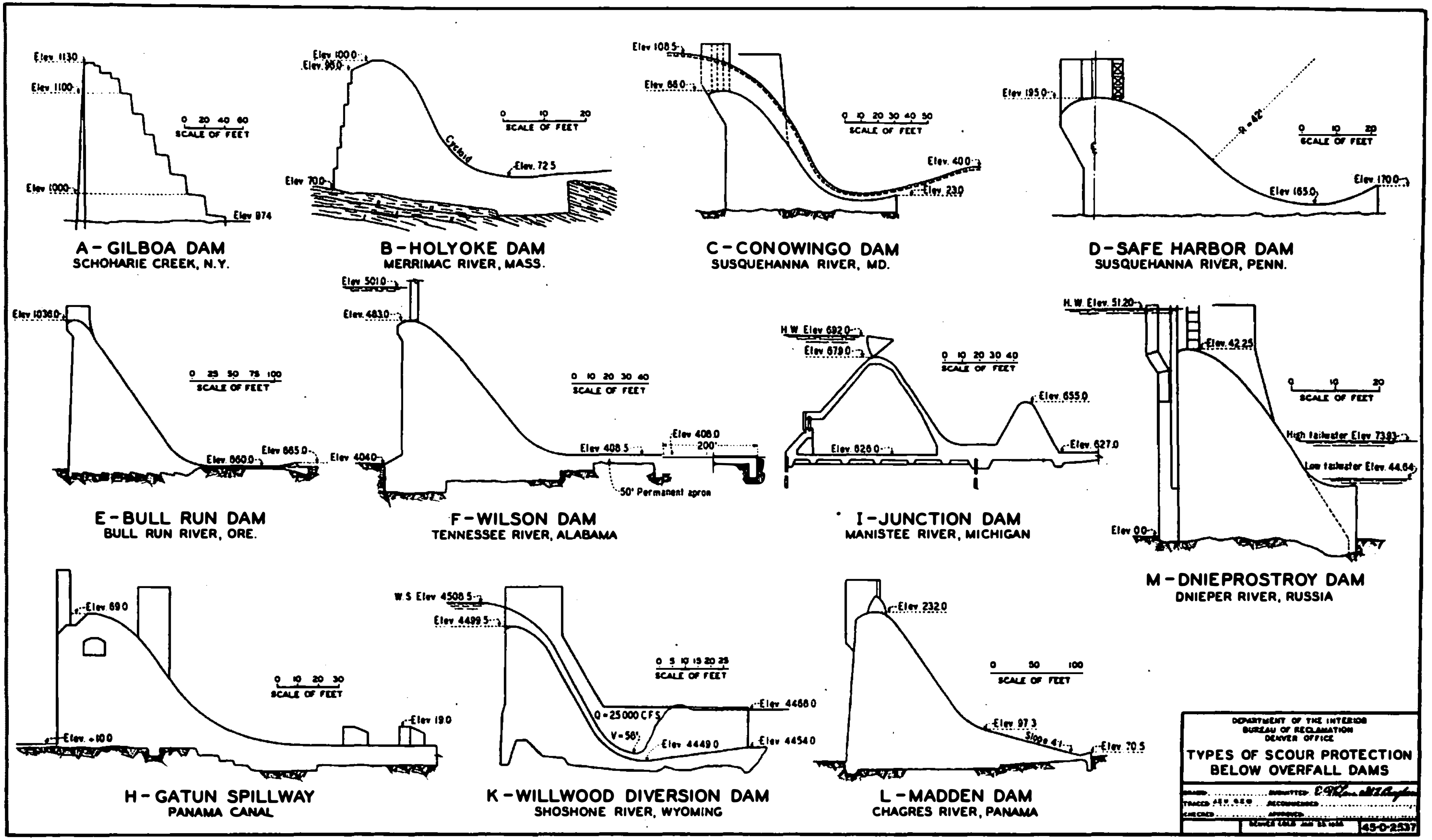
principles of other forms of protection are becoming better understood, this form seems to be less frequently used. The present tendency is to dissipate the energy in some form of stilling-pool or to divert the high velocity stream so that it does not come into contact with the bottom where damage will result.

## The Four Classes of Conditions

The most important factor in determining what form of protection should be used is the depth of water on the downstream side of the dam and its relation to the depth required to form the hydraulic jump.

In the most perfect form of hydraulic jump, the energy of the high velocity water is dissipated so thoroughly in internal impact that little energy remains to be used up in eddies and boils downstream. In the case of a dam with a well-formed jump at its toe,



A - GILBOA DAM
SCHOHARIE CREEK, N.Y.
B - HOLYOKE DAM
MERRIMAC RIVER, MASS.
C - CONOWINGO DAM
SUSQUEHANNA RIVER, MD.
D - SAFE HARBOR DAM
SUSQUEHANNA RIVER, PENN.
E - BULL RUN DAM
BULL RUN RIVER, ORE.
F - WILSON DAM
TENNESSEE RIVER, ALABAMA
I - JUNCTION DAM
MANISTEE RIVER, MICHIGAN
M - DNIEPROSTROY DAM
DNIEPER RIVER, RUSSIA
H - GATUN SPILLWAY
PANAMA CANAL
K - WILLWOOD DIVERSION DAM
SHOSHONE RIVER, WYOMING
L - MADDEN DAM
CHAGRES RIVER, PANAMA
SCALE OF FEET
Cycloid
50' Permanent apron
High tailwater Elev. 73.83
Low tailwater Elev. 44.64
Q = 25000 C.F.S.
Slope 4:1
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE
TYPES OF SCOUR PROTECTION
BELOW OVERFALL DAMS
45-D-2537

the velocity of the water is so quickly and uniformly reduced that little scour of bed and banks results. It is therefore desirable, whenever possible, to design the dam so that such a jump will occur, as the protection required for the bed and banks is thus reduced to a minimum.

The formula for the hydraulic jump in a horizontal channel of rectangular section is $D_2 = \frac{D_1}{2} + \sqrt{\frac{2 V_1^2 D_1}{g} + \frac{D_1^2}{4}}$ where $D_1$ and $D_2$ are the depths upstream and downstream from the jump respectively and $V_1$ and $V_2$ are the corresponding velocities. Consider an ogee dam, Figure 1, discharging 100 second-foot per foot of length with a drop H equal to 50 feet. The velocity $V_1$ (neglecting friction) would then be $\sqrt{2 g \times 50}$ = 56.8 feet per second and the depth $D_1$ would be $\frac{100}{\sqrt{2 g 50}}$ = 1.76 feet. Substituting in the above formula gives a value of $D_2$ 17.9 feet which is the height of tailwater required to form a perfect jump at the toe of the dam.

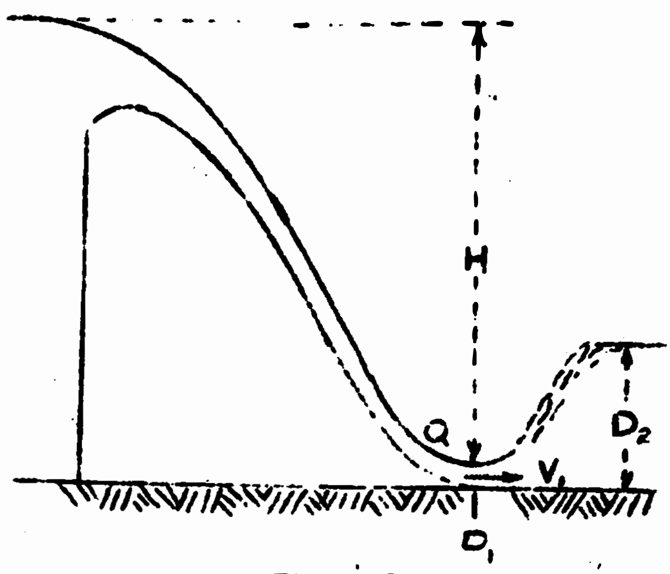


Figure 1.

The ideal condition would be to have a tailwater at such a height above the river bed for each discharge that it would form a perfect jump for the depth and velocity which would occur in the overfalling stream at the toe of the dam for that discharge. The height of the tailwater, however, is controlled by the conditions in the stream channel downstream from the dam and this ideal condition is never exactly attained. Frequently the stage-discharge or tailwater rating curve at the downstream side of the dam is as shown in Figure 2, but the curve which would be required to form the perfect jump on an apron at river bed level is as shown by the jump height curve A-B. This shows that for discharges less than 20,000 second-feet the tailwater height is greater than that required to form a perfect jump but at greater discharges the tailwater height is too low.

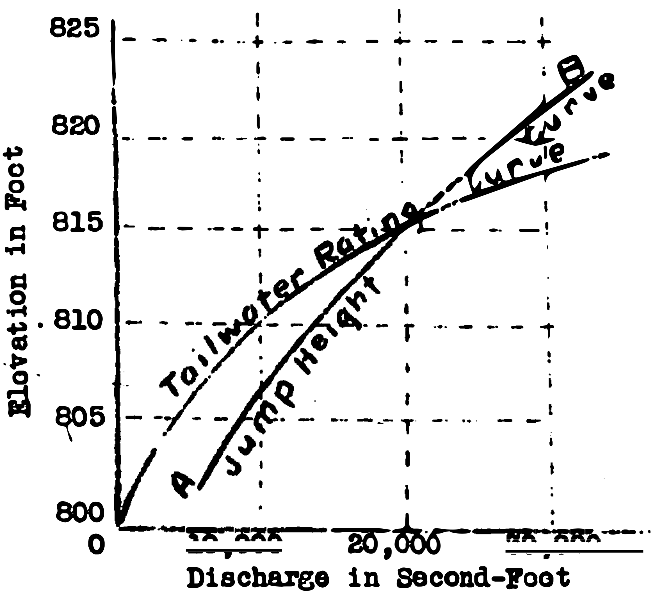



Figure 2.

The relations between the positions of these curves fall into the four following classes:

Case 1 - Jump height curve always above the tailwater rating curve.

Case 2 - Jump height curve always below the tailwater rating curve.

Case 3 - Jump height curve above the tailwater rating curve at low discharges and below at high discharges.

Case 4 - Jump height curve below the tailwater rating curve at low discharges and above at high discharges.

The best form of protection below a dam depends largely upon which of these conditions exist.

## <u>Class 1</u>

The first class frequently occurs when a dam is placed at the head of a rapids or sudden drop in the stream bed. Under these conditions the tailwater level is low and less than the height required to form the jump. Usually, too, the bed is of solid rock which will withstand considerable scour. Under these conditions an upward curving apron is frequently put on the dam which throws the stream of high velocity water passing over the dam upward so that it strikes the stream bed some distance away from the structure. Here the energy is dissipated by impact of the water on the river bottom and adjacent water, and although some scour takes place it is too small and too far from the dam to endanger it. One of the earliest examples of this type of dam is the famous Holyoke Dam on the Merrimac River in Massachusetts (Plate 1-B). Others are the

Conowingo* and Safe Harbor dams on the Susquehanna River (Plates

*Engineering News-Record, p. 127, Vol. 108, Year 1932.

1-C and D).

At the Bull Run Dam** for the Portland, Oregon, water sup-

**Trans., Am. Soc. C. E., p. 487, Vol. 95, 1929.

ply (Plate 1-E), a wide horizontal apron was used with upward sloping deflectors at the downstream edge which directed the deflected stream so that when it fell back to the river level it was spread over a large area, and consequently did not produce as severe scour as if the impact was localized.

The conditions on the Wilson Dam on the Tennessee River (Plate 1-F) are such that it probably falls in Case 1. A wide, level apron was constructed below this dam to protect the river bottom but the depth of the tailwater was insufficient to cause the jump to form and the high velocity water passed entirely across the 200-foot apron and eroded a large hole in the solid rock at its downstream edge.*

*Engineering News-Record, p. 190, Vol. 98, Year 1927.

Experiments with a model of this dam++ have shown that piers would

++Wilsonova Prohrada na Reco Tennessee, Alabama, U. S. A. - Antonin Smrcek.

not cause the jump to form on the apron with the tailwater depth available.

When the tailwater depth is nearly sufficient to cause the jump, baffles or sills may be successfully used, but they often receive so much impact from drift or ice that they are expensive to build with a sufficiently strong anchorage. Baffles or piers do not dissipate as much energy as might be expected and they are therefore not as effective as the hydraulic jump.* Perhaps the best example

*Engineering News-Record, p. 800, Vol. 97, November 11, 1926.

of the use of baffle piers with a low tailwater level is in the Gatun spillway on the Panama Canal (Plate 1-H). This spillway is designed to discharge as much as 140,000 second-feet with a fall of nearly 75 feet. At the toe of the ogee section the water impinges directly against the flat faces of the baffle piers and much of it is thrown high into the air, reaching almost as high an elevation as the water above the dam.** When this water falls to the river level it has

**Trans., Am. Soc. C. E., p. 487, Vol. 93, 1929.
Engineering News-Record, p. 800, Vol. 97, Nov. 11, 1926.
Hydraulic Laboratory Practice, Freeman, p. 506.
International Engineering Congress, pp. 46-63, Vol. II, 1915.

nearly as great a velocity as it had at the toe of the dam. The principal effect of these baffle piers therefore is to distribute the impact over a large area rather than to dissipate the energy of the water.

If the tailwater level is not high enough to form a perfect jump it may be raised by building a low secondary dam below the main dam with a sufficient height to cause the jump to form at the foot of the main dam for all conditions of discharge. This method has been extensively used for dams on earth foundations. Plate 1-I shows a form developed by the Fargo Engineering Company. This method has also been used for a rock foundation at the Martin Dam of the Alabama Power Company. The dimensions required for such a pool are shown by the experiments recently published.*

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*Proceedings, Am. Soc. C. E., p. 1521, Nov., 1932.

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Another method which may be suitable in some cases is to excavate a pool just below the dam to provide a depth sufficient for the formation of the jump. In this case the tailwater level is not changed, but the depth required to form the jump is provided by the lowering of the channel bottom rather than by raising the water surface. This method was used in the Wilwood Dam** (Plate 1-K).

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**Engineering News-Record, p. 660, Vol. 99, October 27, 1927.

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On rock foundations where the tailwater level is nearly high enough to cause the jump to form, experience indicates that if no protection is added downstream from the bucket of the dam, the bottom will be scoured out until sufficient depth is provided to permit the jump to form and no further scour will take place.*

*Civil Engineering, p. 527, Vol. 1, 1931.

## Class 2

Dams in the second class are apt to occur where the foundation rock is at a considerable depth, and where the tailwater surface is therefore higher than necessary to form the jump. This case will be more frequently encountered in the future than in the past, due to the exhaustion of the supply of good damsites. Under these circumstances, if an ogee dam with conventional bucket is used, the water flowing down the face of the dam dives under the tailwater and travels at high velocity a long distance along the bottom, forming only a very imperfect jump. The more nearly the tailwater depth corresponds to the depth required to form the jump, the shorter the distance which the high velocities extend downstream. A perfect jump could be formed for any discharge by building a level apron below the dam to give just the correct depth for the formation of the jump, but this apron would probably not be at the correct elevation for other discharges. By making the apron sloping, the various depths required for the different flows can be provided. The high velocity stream flows down the sloping apron until the depth below tailwater level is reached at which the jump can be formed for that discharge, and at this point the jump occurs. The advantages of the sloping apron have been pointed out by G. Gale Dixon.*

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*Engineering News-Record, p. 696, Vol. 100, May 3, 1928.

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The formula given above does not exactly apply to jumps on a sloping floor since the water enters the jump in a direction somewhat different from that in which it flows away, while the formula assumes motion in the same direction. The formula for the formation of the jump on a sloping surface has been developed by R. W. Ellms.**

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**Trans., Am. Soc. M. E., Sept.-Dec., 1932, paper HYD-54-6.

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A jump formed under favorable conditions is an intimate, relatively uniform mixture of air and water with a white appearance, and the velocity is reduced within a comparatively short distance. As the jump becomes less perfect the mixture is not so intimate and the velocity is not so rapidly reduced. It is questionable if a perfect jump can be formed except on a level floor. As the slope of the floor is increased, the dissipation of the energy becomes less efficient. Experiments on the model of the Cle Elum Dam spillway with floor slopes of $1\frac{1}{2}$ horizontal to 1 vertical, 2:1, 3:1 and 4:1 showed progressively less scour as the slope was flattened.

Probably the outstanding example of the use of the sloping apron for dams in the second class is the Madden Dam (Plate 1-L) now being constructed to store water for the Panama Canal. This form was developed as the result of extensive model tests.*

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*Engineering News-Record, p. 42, Vol. 109, 1932.

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The height of the natural tailwater and the jump height curve for an apron at stream bed level are given on Figure 3, showing that this case falls into Class 2. The shape of the apron was determined by trial. By varying the tailwater level, a determination was made for each form of apron tested of the tailwater levels required for the various discharges, to cause the jump to form at the upstream edge of the apron. For each form a curve was plotted of tailwater level required against discharge over the dam. The agreement of this curve with the tailwater rating curve for the best apron developed is shown on Figure 4. The apron developed had a slope of 4 horizontal to 1 vertical with its upstream edge approximately 30 feet above the foundation level. At large discharges, however, the high velocities extended entirely across the apron and a small triangular sill or lip was placed on the downstream edge of the apron to deflect the swift water up off the stream bed. The sloping apron may require large volumes of concrete but in the case of the Madden Dam this was of assistance in resisting earthquake effects.

In the model experiments for the Madden Dam another form was investigated which requires less concrete and would prove advantageous in certain conditions. It may be called the high bucket

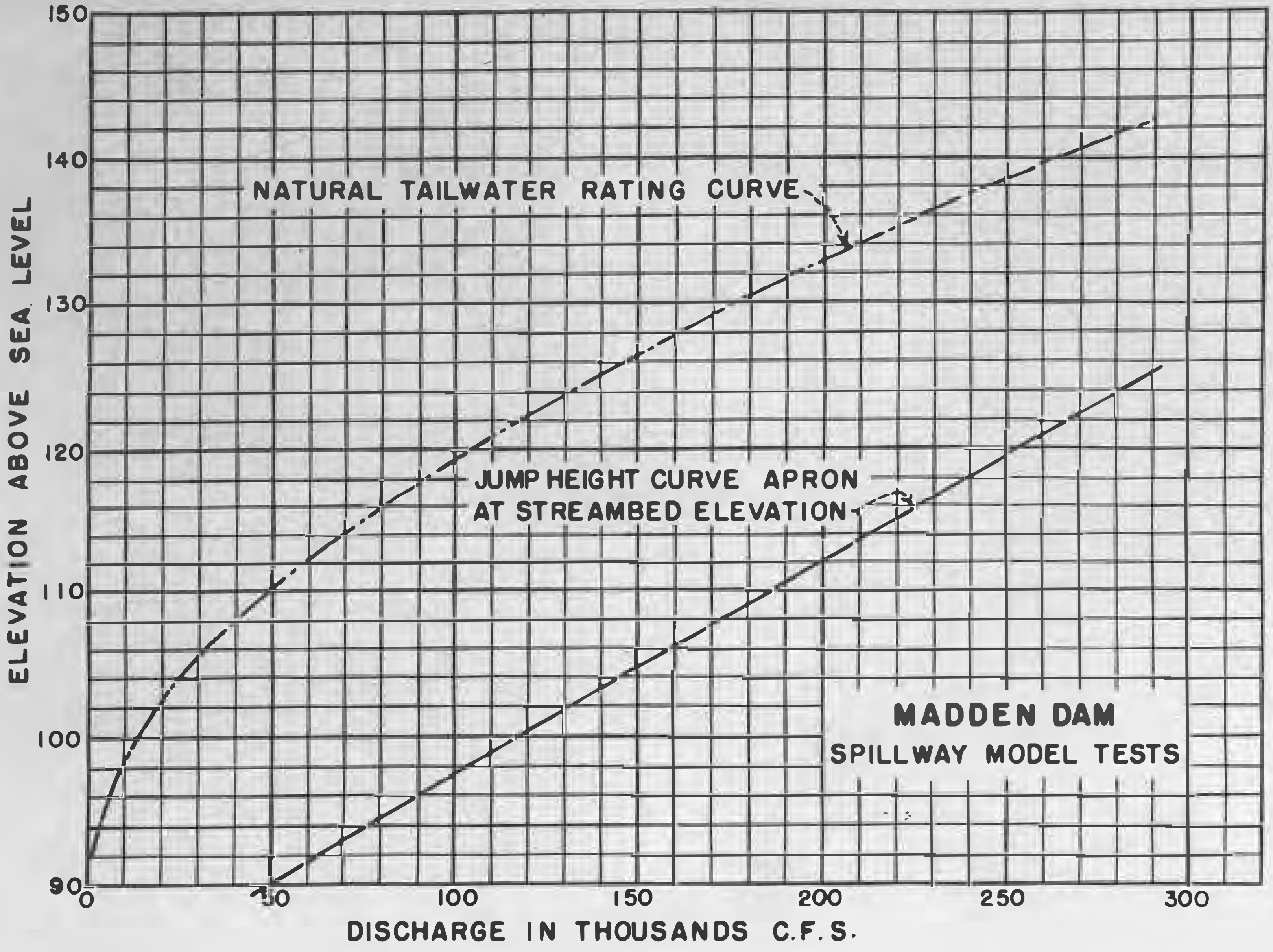
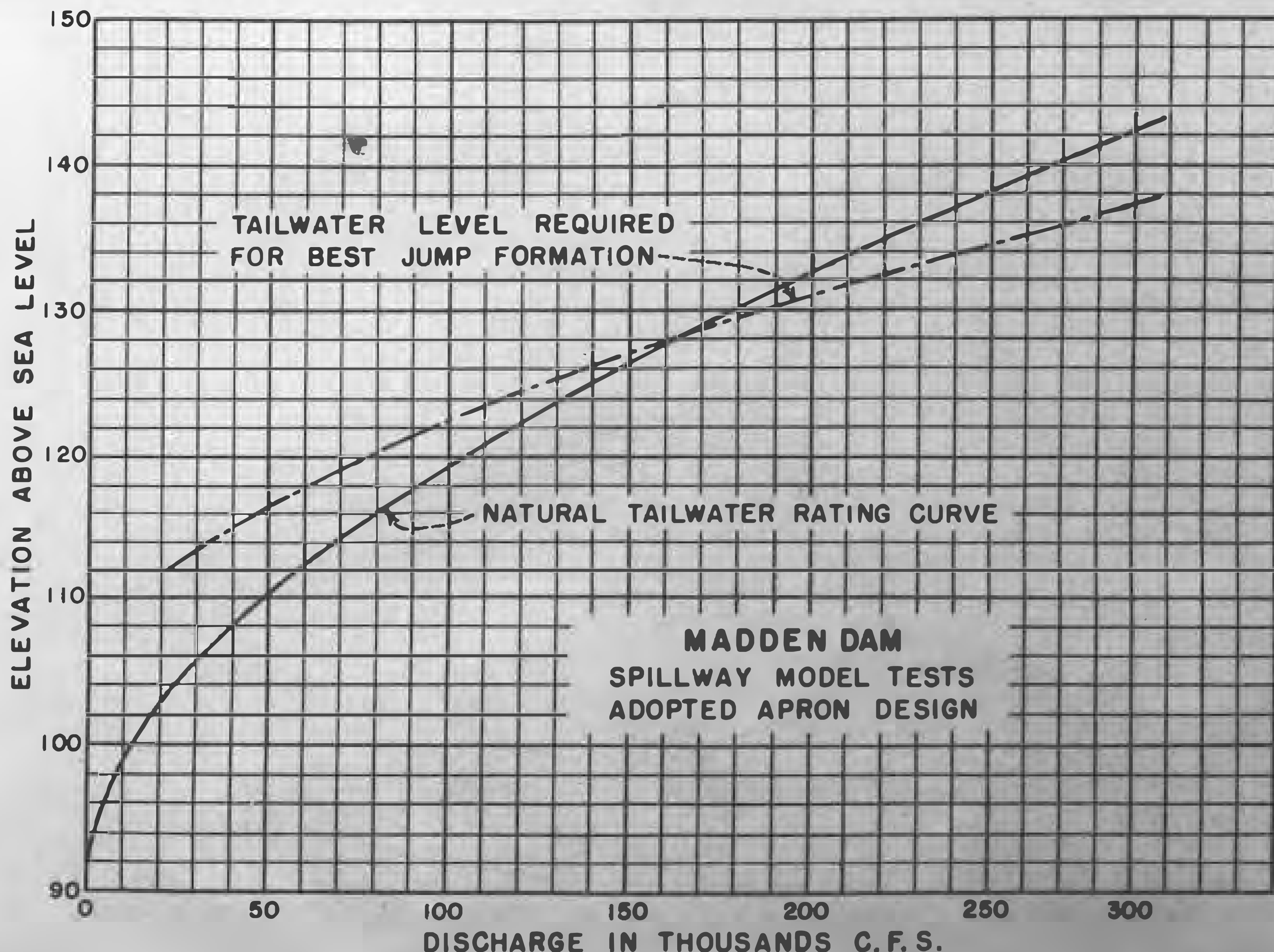

NATURAL TAILWATER RATING CURVE
JUMP HEIGHT CURVE APRON AT STREAMBED ELEVATION
MADDEN DAM
SPILLWAY MODEL TESTS
ELEVATION ABOVE SEA LEVEL
DISCHARGE IN THOUSANDS C.F.S.
150
140
130
120
110
100
90
0
50
100
150
200
250
300

FIG. 3

X-D-1186



150
140
130
120
110
100
90
0
50
100
150
200
250
300
ELEVATION ABOVE SEA LEVEL
DISCHARGE IN THOUSANDS C.F.S.
TAILWATER LEVEL REQUIRED FOR BEST JUMP FORMATION
NATURAL TAILWATER RATING CURVE
MADDEN DAM
SPILLWAY MODEL TESTS
ADOPTED APRON DESIGN


FIG. 4

X-D-1187

type. It consists of the ordinary curved bucket of the ogee weir located some distance above the river bottom, as shown on Figure 5-A.

When the tailwater level is somewhat higher above the lip of the bucket than the upper surface of the high velocity sheet at the end of the bucket, any water which might be on top of this sheet is swept away by friction with this sheet and the upper surfaces of the sheet is below the tailwater level. A pressure is then exerted on the under side of this sheet beyond the lip of the bucket as shown by the arrows in Figure 5-A, due to the pressure exerted upstream beneath the sheet from the higher tailwater level. The high velocity jet is deflected upward by these forces in a great sweep which may carry it even higher than the tailwater level, to which it falls back after the upward motion is overcome by gravity. As this water falls on top of the deep tailwater it does no damage to the river bottom and the only scour on the bottom is that due to the upstream flowing water of the eddy which forms beneath the high velocity sheet. This condition of flow occurs at flood times on the dam (Plate 1-M) of the recently completed Dnieprostroy power plant on the Dnieper River in Russia.* For somewhat higher tail-

*Engineering News-Record, p. 877, Vol. 108, June 23, 1932;
p. 90, Vol. 109, July 21, 1932.

water levels the path of the high velocity sheet tends to become nearly vertical and part of it falls back into the "valley" along

HIGH BUCKET TYPE

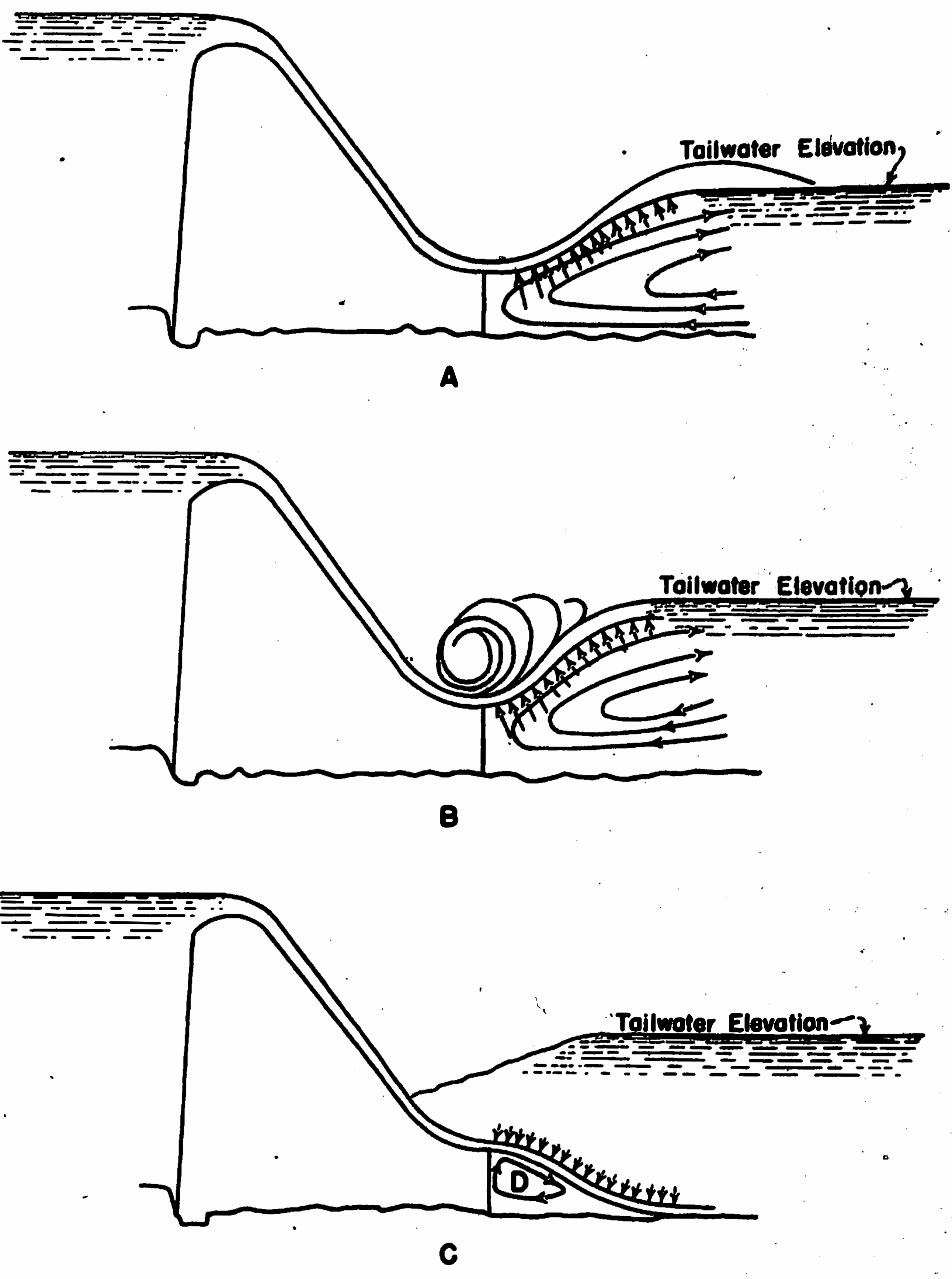



FIG. 5

X-D-1188

the face of the dam, as shown on Figure 5-B. This acts as a brake on the high velocity sheet and it does not sweep so high. The energy in this case is very thoroughly dissipated without attacking the stream bed. On the Madden Dam model the height of tailwater which would give these conditions for a bucket lip at elevation 90 was determined for the entire range of discharge. These tailwater heights were plotted against discharges and were found to be approximately parallel to the tailwater rating curve of the river as shown on Figure 6; that for condition A was roughly 3 feet below the tailwater curve of the river and that for condition B approximately 5 feet above. By lowering the bucket 5 feet the A curve would approximately coincide with the tailwater curve of the river, and a rise of 3 feet in the bucket would bring the B curve into agreement. Thus with a bucket at elevation 85 the A condition would occur at all discharges while at elevation 93 the B condition would occur at all discharges.

For the conditions at the Madden Dam, where there was on the river bed material which was sufficiently fine to be moved by the back eddy, these relations were found to be unstable, as the back eddy carried the fine material up toward the bucket and tended to build up a bar there, which cut off the back pressure from the tailwater and caused the high velocity sheet to flow more nearly horizontal. In this position the high velocity sheet tended to

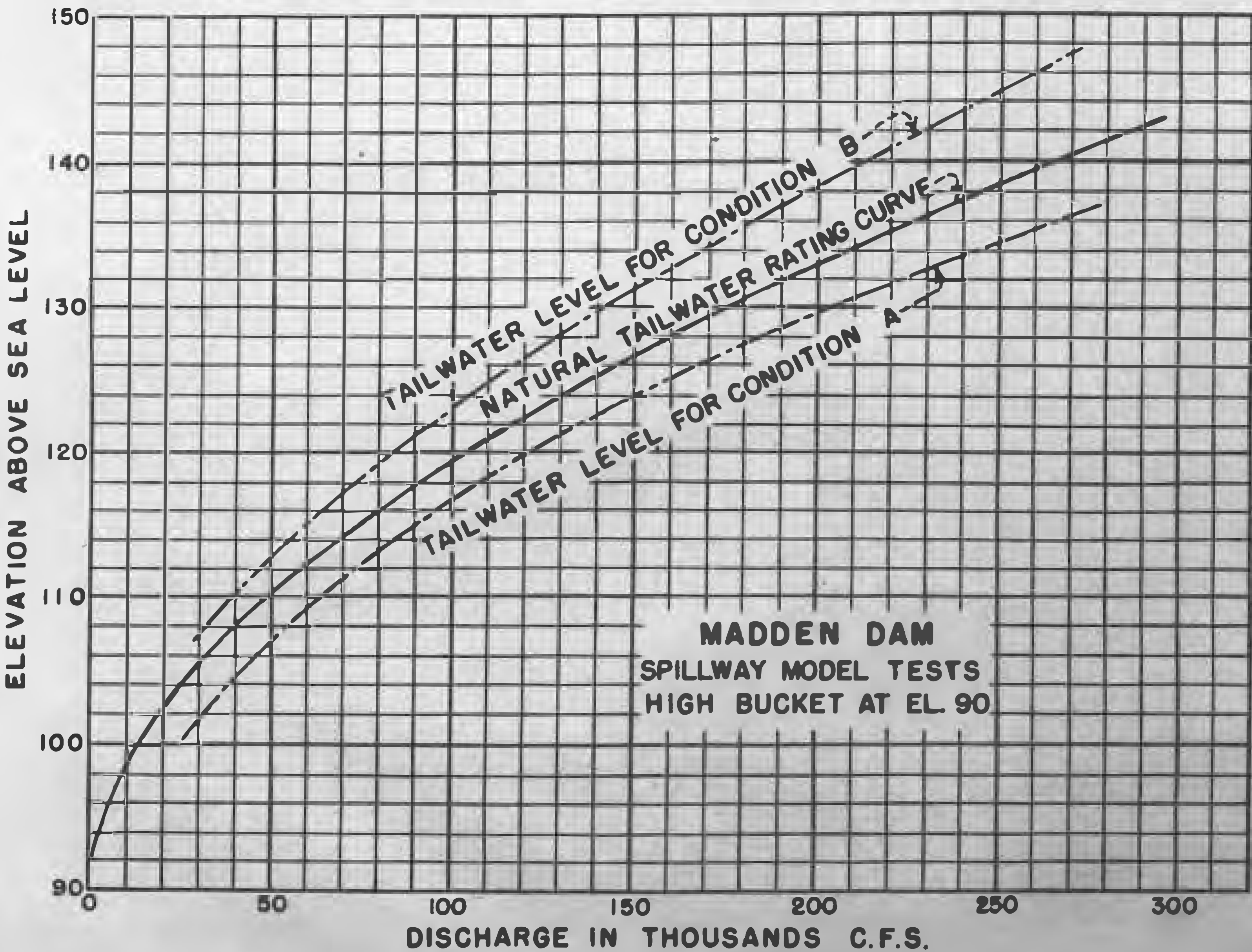

MADDEN DAM
SPILLWAY MODEL TESTS
HIGH BUCKET AT EL. 90
TAILWATER LEVEL FOR CONDITION B
NATURAL TAILWATER RATING CURVE
TAILWATER LEVEL FOR CONDITION A
ELEVATION ABOVE SEA LEVEL
DISCHARGE IN THOUSANDS C.F.S.
90
100
110
120
130
140
150
0
50
100
150
200
250
300

FIG. 6

X-D-1189

carry along with it all the water in the eddy below it and built up a low pressure area at D of Figure 5, C in the place of the eddy. The force of the tailwater pressure then acted on the top side of the sheet, and deflected it downward against the bottom, which it struck with a considerable impact. With a bottom composed of solid rock or material of too large size to be moved by the back eddy, this condition would not occur.

It is believed that these flow conditions offer a method of scour protection below dams worthy of serious consideration for the condition of deep tailwater below overfall dams. Their appearance is not as satisfactory as the hydraulic jump on the sloping apron but they require much less concrete than the sloping apron and if the condition C can be avoided they bring even less scour on the bottom. In the Madden model the lip of the bucket was horizontal. If it was curved up somewhat as on the Dnieprostroy Dam it is probable that the C condition would be less likely to occur. It is not necessary to have only one of the two satisfactory conditions throughout all discharges, as one might be used for low flows and the other for high ones. This will often permit the matching of the river tailwater curve when it cannot be done with either one alone.

The conventional bucket with relatively short horizontal apron may be satisfactory for Class 2 conditions if the jump height curve (based on the apron level) is not too far below the tailwater rating curve, since the proper depth to form the jump may be reached on the slightly sloping portion of the bucket, and a relatively perfect jump be formed. If the difference between the two curves is great, the depth causing the formation of the jump will be reached on a steeply sloping surface in the bucket or on the steep face of the dam and the jump be very imperfect, with high velocity currents extending far downstream, necessitating a long apron. The length of the apron may be considerably shortened by the use of some forms of sill at its downstream end.

## Class 3

The third caso occurs where the tailwater depth at low discharges is insufficient to cause a jump to form on a lovel apron at rivor bottom level, but is moro than sufficient at high discharges. This is a common caso and tho solution consists in artificially creating enough wator depth to make the jump form on tho apron at low discharges. This may be dono by a low secondary dam or sill across noar tho downstroam end of a levol apron. This secondary dam must bo high enough to causo the hydraulic jump to form upstream from it for all flows for which the natural tailwater depth is insufficient. It may nood a secondary apron downstream from it for it is in face just a second dam. Tho dopth required may also be secured by depressing the apron, proforably by sloping it downward in a downstream direction. This method was used on the repair of the Hamilton Dam.*

*Engineering News, p. 130, Vol. 108, January 28, 1932.

This third case is usually favorable to the use of baffle piers or some form of dentated sill near the end of the apron, as these tend to break up the high velocity flow at low discharges and also to raise the tailwater, both of which actions promote the formation of the jump. They would also be advantageous at high flows since then the depth of tailwater is greater than required to form the jump and the nappe over the crest tends to dive down to the bottom of the river and flow along the apron at high velocity as previously described. When this condition occurs, the sills or piers are useful in breaking up this destructive current. For low intake dams it is not uncommon to have the height of tailwater at high flows sufficient to submerge the entire dam. In such cases the drop at the dam is slight and protection against these high flows is no problem.

## Class 4

The fourth case, where the tailwater depth is sufficient at low flows but too small at high flows, can be solved by increasing the depth of tailwater sufficiently to cause the jump to form for the maximum discharge contemplated, either by a secondary dam or an excavated pool. With these of the magnitude required for the maximum flow, the tailwater depth would be more than sufficient to cause the jump to form on the apron at the lower flows but this condition would probably not be objectionable for low flows.

## Effect of Changes in Dam Crest Length

To make the foregoing analysis as readily understandable as possible, it has been assumed that the stream flow was uniformly distributed over a fixed length of the dam and therefore for a given discharge there would be a fixed condition of overflow and a fixed tailwater elevation. By the proper selection of crest length, however, it may be possible to secure a closer agreement between the tailwater rating curve and the jump height curve than is secured by an arbitrarily chosen length. If the first assumed length produces a Case 1 condition, the agreement will be improved by increasing the crest length, which will cut down the discharge per foot length and consequently lower the jump height curve. Similarly, a Class 2 condition can be improved by decreasing the crest length, producing a greater discharge per lineal foot of crest and thus raise the jump height curve. Such changes may result in increases in the cost of crest gates or other features of the dam but this might be much more than offset by the reduced cost of bottom protection, and the possibilities of such a saving justify a study of this phase where the circumstances permit a choice of crest lengths.

## Effect of Crest Gates

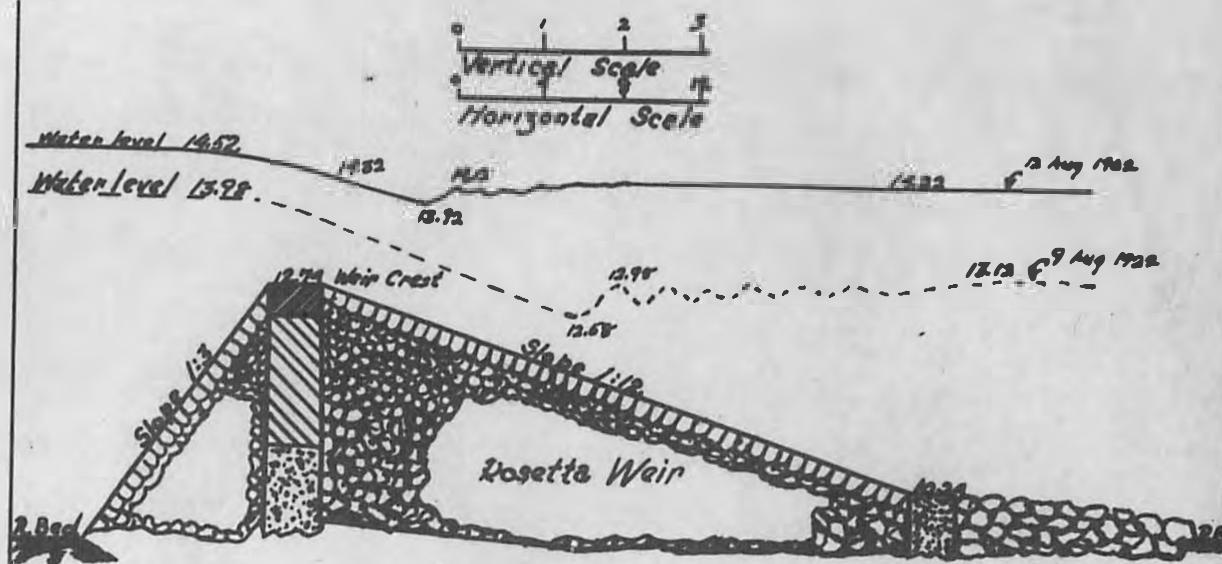
The foregoing analysis has also been simplified by assuming that the flow of the stream was uniformly distributed over the entire crest length of the dam. With control gates on the crest, however, the flow can be concentrated and at certain portions of the dam the flow will be greater than at others. To be perfectly safe, the protection should be designed so that it will be sufficient with any possible condition of gate openings and flow. As a practical matter, however, it may be assumed that reasonable judgment will be used in operating the gates, and sufficient protection provided so that no severe damage would result with the undesirable conditions acting for a limited time, such as might occur; for example, in the case of the failure of a gate to close when desired. In most cases it will be found best to have the crest gates designed to be capable of operating partially open, in order to distribute the water uniformly over the crest instead of permitting only an entirely open or entirely closed condition.

## Protection for Weirs of Indian Type

The foregoing classification applies principally to protection below masonry overfall dams of the types commonly used in this country. For the broad weirs with slightly sloping aprons frequently used in India and Egypt, a somewhat different analysis is necessary. The oldest form of this type of weir consists of a

# Rosetta Weir
## Profile of Standing Wave

All Dimensions in Meters



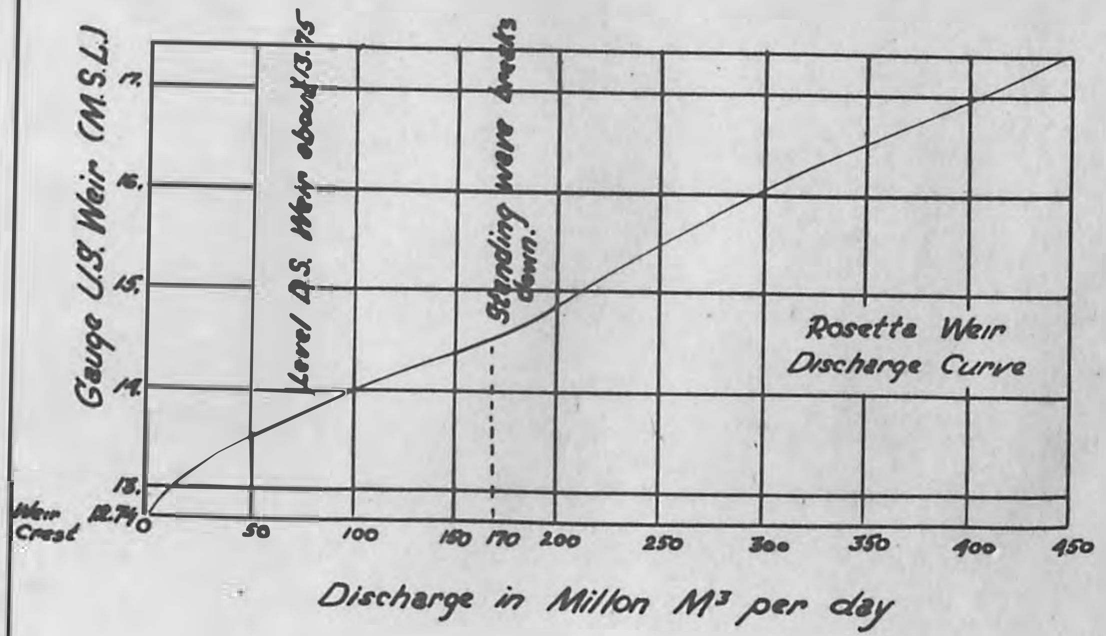




Fig. 7

X-D-1190

pile of loose rock, with or without masonry dividing walls, having a sloping upper surface paved with hand-laid rock or masonry, as shown on Figure 7. After passing over the crest of a weir of this type the water flows down the sloping apron until it encounters the tail-water, and at this point a hydraulic jump is usually formed. To prevent scour, this jump should form far enough from the downstream end of the apron that the turbulence of the jump does not reach to the unprotected river bed. To secure this result may necessitate the extension of the apron below the natural level of the river bed. It is necessary, or at least desirable, to have the point where the jump occurs move up the apron as the discharge increases, since this provides a longer length of apron to take care of the greater turbulence of the jump with the larger flows. It is also an advantage if the weir is entirely drowned out for very high flows, since this eliminates the combination of high discharge and fall which would produce so much destructive energy. These conditions exist at the Rosetta weir,* ( Figure 7) which is at the head of the delta of the

*Standing Wave Weirs, by A. D. Butcher.

Nile. The surface profiles show that the position of the jump moves upstream as the discharge increases and for a flow of 170,000,000 cubic meters per day the jump is entirely drowned out. Although for many dams of this type the maintenance cost has been high, when conditions are favorable it may be very low, as indicated by the maintenance charge of 0.4% for a 15-year period at the Laguna Dam on the Colorado River,* all of which was due to a cutoff of the river just downstream from the weir, which lowered the tailwater level 7 feet and necessitated an extension of the apron.

*New Reclamation Era, p. 189, December, 1924.

With this type of weir the possibilities of increasing the tailwater depth are rather limited. It is therefore unsuited to conditions having a low tailwater level and two complete failures are believed to have been due to tailwater levels so low that the hydraulic jump occurred too near to or below the downstream edge of the masonry, causing an undermining of the structure which progressed upstream and caused a breach. An adjustment of the dam to fit the tailwater conditions may sometimes be made by a proper choice of the dam length, as previously discussed.

Most of the recent dams of this type in India and Egypt have been constructed with broad, sloping masonry aprons surmounted with piers and Stoney gates. The foregoing discussion applies to this kind also, although it is complicated somewhat by the necessity of having sufficient tailwater depth to cause the jump to form on the apron with the gates in a partially open condition. To secure this it may be necessary to carry the apron below natural bed level.

If the river carries much bed load, allowance should be made in computing the position of the jump for a retrogression of the downstream river level.

## Tailwater Rating Curves

Enough has been given in the foregoing to show the importance of the tailwater rating curve in the solution of this problem. One of the first steps in attacking the problem at any site is to determine the tailwater rating curve, either by observing the actual levels for a wide range of discharges or by computation. The former method should be used if possible, but fairly satisfactory results may be secured by determining the tailwater elevation for various discharges by means of backwater curves for these flows. Since the water levels at downstream points for these discharges are probably also unknown, the curves may be started at assumed elevations far enough downstream from the dam that the error in the assumed elevation will have disappeared before the curve has been computed as far upstream as the dam. That this is the case may be determined by assuming somewhat different elevations for the same discharge at the lower end of the stretch and if both assumptions give practically the same elevation at the damsite, the error introduced by an incorrect assumption is negligible.

## Laboratory Experiments Necessary

It is not the intention of this article to give the impression that hydraulic laboratory tests are unnecessary in working out the best form of scour protection below dams. Such tests should be made on all important structures and will usually pay for themselves in the improvements which they bring about in the minor features of the design, entirely aside from the major improvements which they make possible. The intention of this analysis is only to point out the lines along which the best solution probably lies, in order that effort may not be wasted in unnecessary investigation.

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