STILLING BASINS

Report by
David Navon

for
CE 252

Design of Irrigation Structures

Colorado A & M College
Fort Collins, Colorado

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Chapter I

GENERAL CONSIDERATIONS

1. Introduction

One of the biggest problems that an Irrigation Engineer faces in the design of hydraulic structures, is the dissipation of the energy of high-velocity water. This problem is encountered whenever water flows in canals and gullies having a super-critical slope, when water flows over the spillway of a dam, or when it shoots out of an outlet in the dam or from under a gate. In all these cases there is super-critical, or high-velocity, flow having a high energy content. This energy, if not dissipated properly, will result in scouring the river or channel bed downstream, and may endanger nearby structures.

Energy dissipation of flowing water is most commonly and efficiently accomplished in a stilling basin, which is a structure so designed as to transform most of the kinetic energy of the flow into turbulence and eventually into heat. In this way the super-critical flow is transformed into sub-critical flow. Indeed, in some cases the energy is only slightly dissipated and the high-velocity jet is diverted farther downstream, where scour is permissible, without endangering the structure or any other features near the site.

The purpose of this report is to present a brief summary of the principles of energy dissipation, prevention of scour, other factors encountered in high-velocity flow, and the different types of stilling basins with examples from the engineering practice in the United States. The writer hopes this report will serve as a convenient summary, so much needed in this field, both for study and reference.

2. Principles of Energy Dissipation

Warnock (56) stated that the energy of super-critical flow is dissipated by one or a combination of the following means:

1. External resistance,
2. Impingement,
3. Internal resistance, or turbulence.

External resistance may be either between the water and the channel or between the water and the surrounding air--as in the case of a jet spread out by a deflector over a wide area, or a jet discharged from an outlet high above the tailwater, where the jet is disintegrated through air-entrainment resulting, at times, in a widespread rain instead of a solid jet. In most cases external resistance is used to dissipate only a small portion of the kinetic energy, although in other cases it dissipates a large portion of the energy.

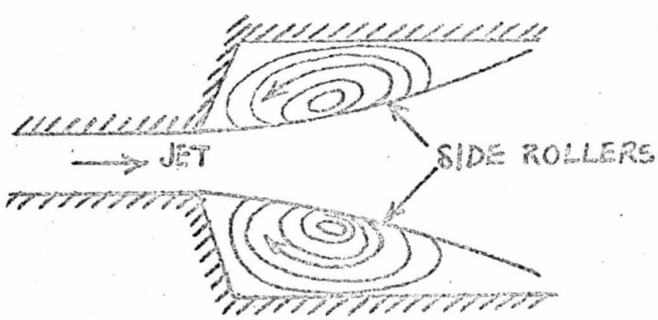
Impingement occurs whenever the jet strikes a pool of water or solid objects such as floor blocks. This method is not recommended, since the free impact of water against a structure causes an intense shock to the structure, damages the concrete face exposed to direct impact (e.g., the floor blocks), may result in cavitation, and the spray of the water results in other difficulties (Schoklitsch--39).

<u>Internal resistance or shear</u> is the most efficient and the most commonly used means of energy dissipation. Here the water is brought to a state of high turbulence, transforming the bulk of flow energy into heat. This is accomplished by either one of the following means:

(a) Counterflow.
(b) Rollers.
(c) Hydraulic jump.
(d) Small-grain turbulence.

<u>Counterflow</u> is achieved by jets discharged from outlets in the opposite walls of a channel or a basin. The jets, flowing in opposite directions, produce great turbulence when they meet in the middle of the basin.

<u>Rollers</u> cause high fluid resistance due to large differences of velocity throughout the fluid. There are three types of rollers--surface roller above the jet, ground roller under the jet, and side roller.



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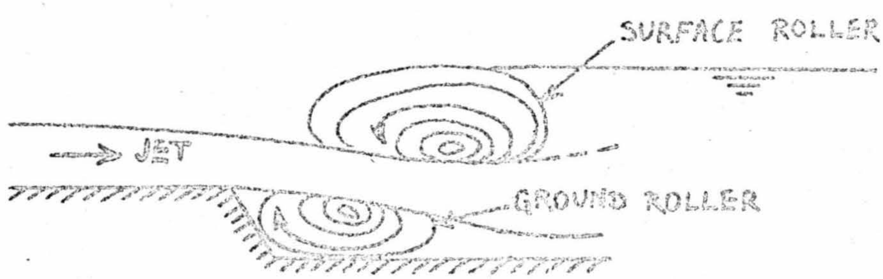


PROFILE

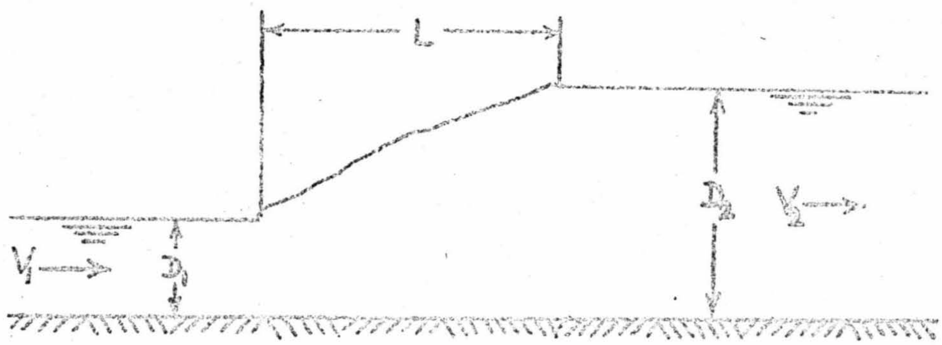
The efficiency of the roller depends on its size and shape, on the velocity-difference between the jet and the surrounding layers of water, and on the boundary proximity--the boundary being water, bed, or structures. The amount of energy dissipated increases with an increase of the velocity-difference and an increase of the volume of the roller. The dissipation decreases when the roller approaches a circular form. These properties were found in model studies on roller-type stilling basins.

The <u>hydraulic jump</u> is a particular case of surface roller, which occurs when flow conditions are favorable for its formation. The hydraulic jump is the classical type of energy dissipator, because of its efficiency and smooth action, and is therefore used very extensively. Because of its importance in the study of stilling basins, the hydraulic jump will be described in more detail in the next article of this report.

Small-grain turbulence is achieved when a jet, discharged from an outlet above tailwater, is protected by means of a close boundary, until submerged in the tailwater where it produces eddies on the bottom of the basin. This causes a state of small-grain turbulence in the water (rather than big rollers or waves), which is very efficient in dissipating the energy of the jet. The scale of the turbulence is a function of boundary proximity.

The use of small-grain turbulence in stilling basins was recently developed, in model studies of outlet works, by the U.S.B.R. (Peterka and Tabor, 31).

3. The Hydraulic Jump



When water flowing at a depth $D_1 < D_c$ (critical depth) impinges upon water of proper sequent depth downstream $D_2 > D_c$, a hydraulic jump is formed. The hydraulic jump is a breaking surge remaining fixed in position relative to the observer, if conditions of flow are suitable.

Considering the simple case of a rectangular cross-section and a level bottom of the channel, a very simple mathematical relationship can be developed for the jump characteristics. This relationship was found to agree quite precisely with observed data.

Let V_1, D_1, and Fr_1 be the velocity, depth of flow and Froude Number, respectively, upstream of the jump; and V_2 and D_2 the velocity and depth of flow downstream. Using the momentum equation, or in other words—equating the change of momentum with the change of static pressure within the boundaries of the jump, and neglecting the friction between the water and the channel, the necessary depth of tailwater D_2 may be obtained:

$$D_2 = -\frac{D_1}{2} + \sqrt{\frac{2V_1^2 D_1}{g} + \frac{D_1^2}{4}}$$

$$\text{or:}\quad D_2 = \frac{D_1}{2}\left(\sqrt{1+8Fr_1^2} - 1\right)$$

This equation determines the depth of tailwater necessary for the formation of a jump, when the upstream depth and velocity are known. The length of the jump L was found experimentally to be between 4 and 7 times its height, but the most acceptable values are:

$$L = 4(D_2 - D_1) \quad \text{or} \quad L = 5(D_2 - D_1)$$

The hydraulic jump is one of the most efficient energy dissipators, both with respect to distance and completeness of dissipation. Most of the dissipation occurs in the steep part of the jump, rather than further downstream, through unbalanced velocity distribution.

The energy loss can be determined mathematically as the loss of total head through the jump. The loss of total head, by Bernoulli's theorem is:

$$\Delta H = \frac{(V_1 - V_2)^3}{2g(V_1 + V_2)} = \frac{(D_2 - D_1)^3}{4D_2 D_1}$$

The loss of kinetic energy is the loss of velocity head:

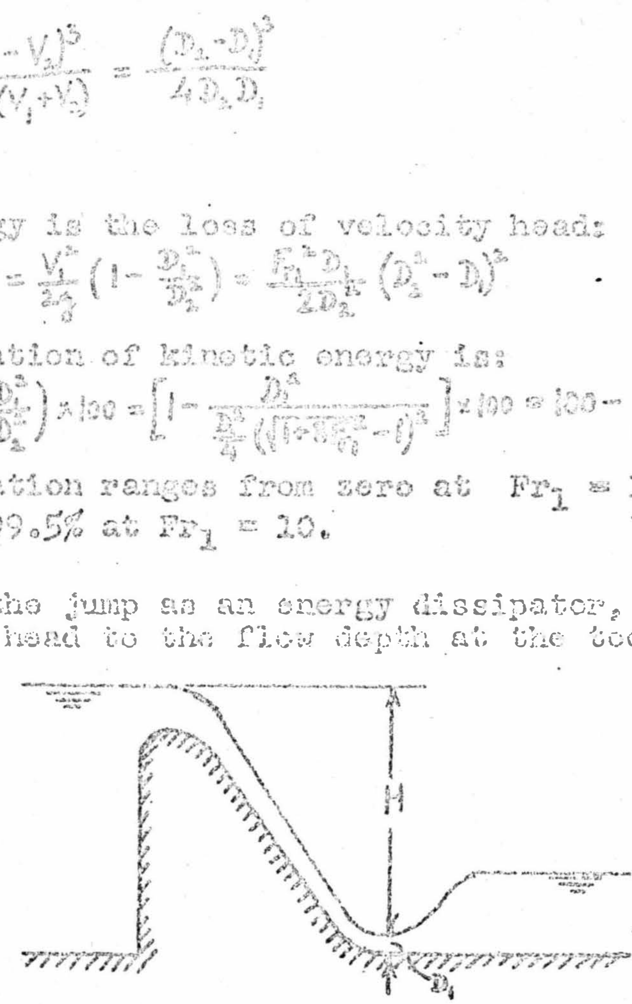
$$\Delta H_V = \frac{V_1^2}{2g} - \frac{V_2^2}{2g} = \frac{V_1^2}{2g}\left(1 - \frac{D_1^2}{D_2^2}\right) = \frac{F_{r_1}^2 D_1}{2D_2^2}\left(D_2^2 - D_1^2\right)$$

The percentage of dissipation of kinetic energy is:

$$\frac{\Delta H_V}{V_1^2/2g} \times 100 = \left(1 - \frac{D_1^2}{D_2^2}\right) \times 100 = \left[1 - \frac{D_1^2}{\frac{D_1^2}{4}\left(\sqrt{1+8F_{r_1}^2} - 1\right)^2}\right] \times 100 = 100 - \frac{400}{\left(\sqrt{1+8F_{r_1}^2} - 1\right)^2}$$

The percentage of dissipation ranges from zero at $Fr_1 = 1$ to 82.3% at $Fr_1 = 2$ and to 99.5% at $Fr_1 = 10$.

The efficiency of the jump as an energy dissipator, depends very much on the ratio of the head to the flow depth at the toe of the jump: $\frac{H}{D_1}$.



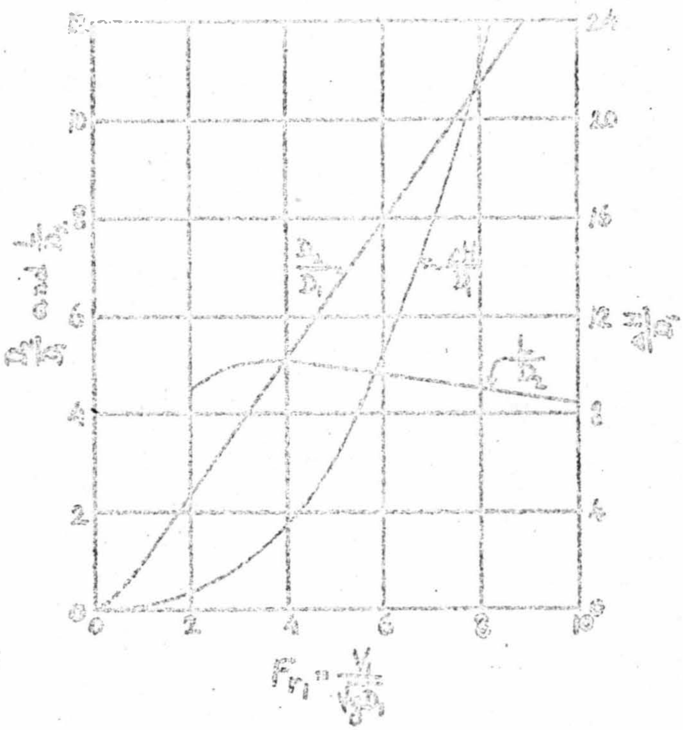
Stevens' formula, for the dissipation of energy in a hydraulic jump, is (41):

$$\text{The energy dissipated (\%)} = \frac{6.25\left(\sqrt{1+16\frac{H}{D_1}} - 3\right)^3}{\left(1+\frac{H}{D_1}\right)\left(\sqrt{1+16\frac{H}{D_1}} - 1\right)}$$

From this formula it can be found that for $H/D_1 = 1$, only 1.4% of the energy is lost; for $H/D_1 = 10$, 44%; and for $H/D_1 = 30$, 65% of the energy is lost through the jump. Thus, it is obvious that the jump is most effective for high values of H/D_1.

The curves, presenting jump characteristics as dimensionless functions of the Froude Number, show very close agreement between the mathematical relationships and the corresponding experimental values as found by Bakhmeteff and Matzke (Columbia) (3) and by Bliss and Chu (Iowa) (Rouse, 34, p. 146). (See figure on following page.)

For the jump on a sloping apron, there is no simple equation as for the horizontal apron, but for flat slopes up to 10 or 15%, the level-floor jump formula applies quite satisfactorily (64). For steeper slopes the jump becomes increasingly less perfect, and actually there is only a dip in the water surface, between the flow on the steep-slope



apron and the tailwater. The jet is drowned in the pool where it forms rollers. Viscosity and surface tension have an undetermined effect on this action (Posey and Woodward, 64).

In trapezoidal channels the simple jump equation cannot be applied, and the relationship between the jump characteristics has to be found by trial and error computations or by plotting a set of curves based on the momentum equation:

$$A_1\bar{D}_1 + \frac{QV_1}{g} = A_2\bar{D}_2 + \frac{QV_2}{g}$$

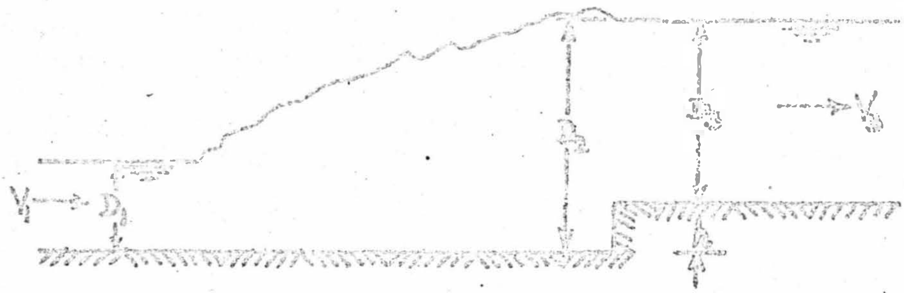
Where: A = area of channel cross-section
$\bar{D}$ = average depth of flow in cross-section
Q = discharge
V = velocity

Experimental work by Posey and Hsing verified the momentum equation in this case (Hsing, 20; Posey and Hsing, 32). The length of the jump increases rapidly with the flattening of the side-walls of the channel.

For the jump over a step in the level-floor channel (as in the case of end-sills), Forster and Skrinde (17) found the following equation:

$$D_3 = D_1\sqrt{1 + 2F_{r_1}^2\left(1 - \frac{D_1}{D_2}\right) + \frac{h}{D_1}\left(\frac{h}{D_1} - \sqrt{1 + 8F_{r_1}^2} + 1\right)}$$

This equation is solved by trial.



The minimum necessary tailwater depth D_3 is the critical depth: $D_3 \geqq D_c = D_1 F_{r1}^{2/3}$. Forster and Skrinde present also a jump equation for a sharp-crested weir in the level-floor channel, which can be applied to transverse sills in the basin. This equation is very complicated and also has to be solved by trial. However, the authors suggest using empirical results instead of their equation, since they don't agree too closely.

4. Prevention of Scour

One of the primary purposes for the construction of stilling basins, is to prevent dangerous scour of the channel bed below spillways, outlets, and drop-structures. When a high-velocity jet or sheet of water falls over the unprotected bed, downstream from the structure, it may erode the bed material and carry it either in suspension or as bed load further downstream. Serious scour immediately below the structure may endanger its foundation, but a certain amount of scour may be permissible further downstream if the bed material is not highly erodible, and there are no structures close enough to be endangered. Although the problems of scour fall in the field of erosion and sedimentation, which is out of the scope of this report, a short summary of the main characteristics of scour will be presented here.

Gilbert (19) found that the scouring capacity of a stream increases with the increase in flow discharge, with the decrease of size of bed material, and with the increase in flow velocity (the average increase being with $V^{3.2}$ when the velocity increase is due to a discharge increase). Rubey (35) found that the weight of the largest particles moved by a stream varied with V^6 for coarse sand and gravel, but smaller particles required higher velocities than indicated by this function. According to Gilbert, the significant velocity is that on the channel bed, rather than the mean velocity of the cross-section.

A quantitative study of scour was done by Schoklitsch (39), who found that the maximum depth of scour over the entire width of an unprotected river bed, after a prolonged impact of the free overfall over a weir, is:

$$T = \frac{0.30}{d_m^{0.32}} h^{0.2} q^{0.57}$$

Where: T is the maximum depth of water above the scoured bed (ft.),
q is the discharge over unit width of weir (cfs/ft.),
h is the head measured from head-water elevation to tailwater elevation (ft.),
d_m is the effective diameter of the bed material (that diameter of which 10% by weight of the sample is finer) (mm.).

Studying scour downstream from culvert-outlets, Blaisdell (7) found that the increase in depth and volume of scour with time is very rapid at first and then the depth slowly approaches the maximum. In his experiments, 65% of the total scour occurred in 12.5% of the total time of scouring, and 35% took the other 87.5% of the time.

Rouse (33) studied the scour effect of a vertical jet on a level sand bed, and found that:

(a) The depth of scour in uniform material depends upon the size and velocity of the jet, the mean fall-velocity of the material, and the duration of the scouring action (varying with the logarithm of time).

(b) The rate and magnitude of scour decreases with the decrease of the ratio of jet-velocity to mean fall-velocity of sediment, approaching zero as this ratio approaches unity.

(c) No equilibrium of scour can be expected in a relatively uniform material, the removal of sediment continuing as an exponential function of time. In a material having a wide range of sizes, selective sorting takes place, so that the mean fall-velocity of the sediment lining the hole rises steadily, with the tendency to approach a state of equilibrium.

(d) The amount of scour decreases with the increase of sediment load in the inflow, and the scour pattern will become stabilized if the sediment load is equal to that which the jet can scour.

(e) The resistance of the bed material to scour depends on its ability to resist shear.

Doddiah (14) studied the scour caused by hollow and solid jets vertically discharging downward and found that both jets have practically the same scouring capacity. He also found that scour increases with increase in the depth of tailwater, until the depth reaches a certain critical value, with further increase in depth decreasing the scour. The explanation of this strange effect is that the extent of turbulence increases with increasing depth of water and the boils cover a wider area, giving rise to a pronounced ring-vortex form of flow. Hence, with the same amount of energy, the horizontal component of velocity increases and the sediment is thrown farther from the center of the jet. When the depth of water is greater than the critical scouring depth, greater diffusion of the jet occurs in the pool and its scouring capacity decreases. The critical scouring depth depends on the characteristics of the jet, the pool, the flow, and the sediment. Doddiah also substantiated Rouse's conclusion that the depth of scour varies with the logarithm of time.

Lane (24) recommends limiting values of mean velocities safe against erosion in straight canals, after aging, with clear water. If the energy dissipator produces a mean velocity at or below the value given in the following table, the result should be satisfactory inasmuch as the bottom velocity won't exceed about 75% of the mean.

Bed material	Roughness (Manning's n)	Minimum mean velocity (fps.)
Sandy loam (noncolloidal)	.020	1.75
Silt loam (noncolloidal)	.020	2.00
Alluvial silts (noncolloidal)	.020	2.00
Ordinary firm loam	.020	2.50
Volcanic ash	.020	2.50
Stiff clay (very colloidal)	.025	3.75
Alluvial silts (colloidal)	.025	3.75
Shales and hard-pans	.025	6.00
Fine sand (noncolloidal)	.020	1.50
Medium sand (noncolloidal)	.020	1.65
Coarse sand (noncolloidal)	.020	2.00
Fine gravel	.020	2.50
Coarse gravel	.025	4.00
Cobbles and shingles	.035	5.00
Graded loam and cobbles (noncolloidal)	.030	3.75
Graded silt to cobbles (colloidal)	.030	4.00

The following corrections should be made in sinuous canals:

Degree of sinuousity	Corresponding value of mean velocity
Straight canals	100%
Slightly sinuous canals	95%
Moderately sinuous canals	87%
Very sinuous canals	78%

Blench (8a) extended Lacey's regime theory, previously used in India for channel design, and introduced the bed-factor b and the side-factor s, so that:

$$b = V^2/D$$
$$s = V^3/W$$

where V is the flow velocity, D is the depth, and W the width of the channel. Blench has shown that the minimum safe velocity in a channel is much lower (less than 1.0 fps.) at low flows than at high flows, for the same bed material.

5. Miscellaneous Factors

Besides prevention of scour, the purposes for the construction of stilling basins may be to overcome other difficulties resulting from high velocity flow, such as:

(a) Excessive waves.
(b) Splash and spray.
(c) Cavitation of structures due to extremely low pressures.
(d) Abrasion of structures due to impact of jet, sediment, and debris.

Chapter II

DESIGN CHARACTERISTICS

1. Design of Stilling Basins

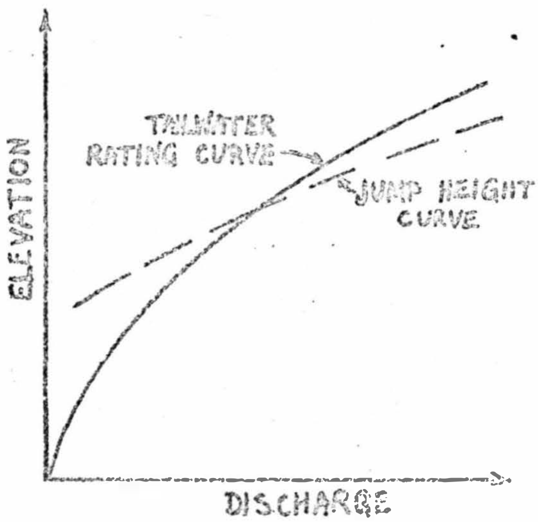
In the foregoing discussion it was pointed out that the hydraulic jump is the most efficient means of energy dissipation, but formation of a jump requires that the tailwater depth be equal to D_2 in the jump equation (Ch. I-3), which cannot be arrived at in most cases. Lane (25) classifies stilling basins in 4 classes, according to the relationship between the tailwater depth and the jump height D_2, as follows:

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

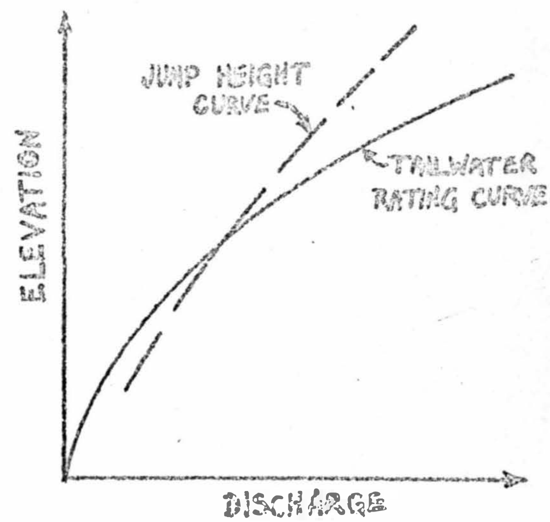
Class 2.--Jump-height curve always below the tailwater rating curve.

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

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



CLASS 3



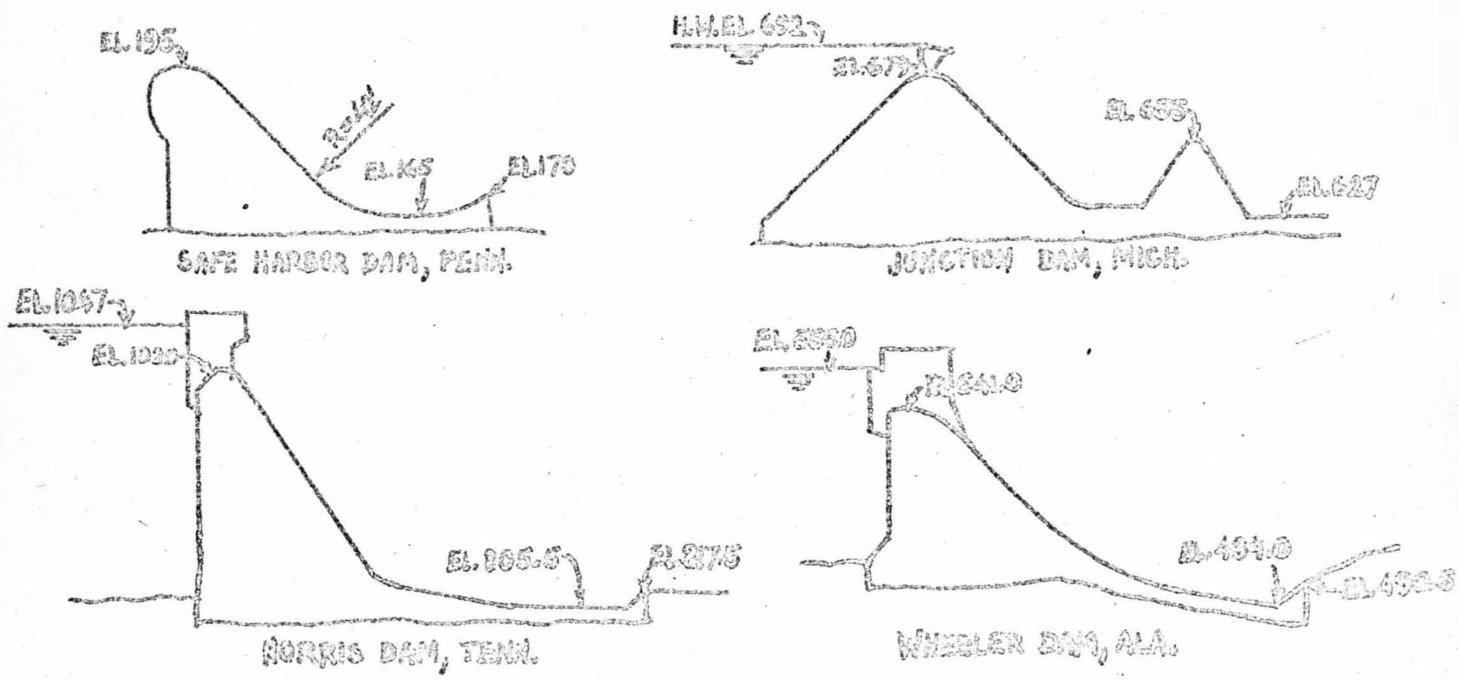
CLASS 4

Class 1. If the tailwater is always lower than necessary to form a jump, the flow will sweep across the apron (if a horizontal apron at stream-bed level is used) at high velocity and scour the bed downstream from the apron, as happened at Wilson Dam on the Tennessee River, Alabama. To prevent this dangerous condition, the flow can be thrown farther downstream, by upturned buckets or aprons with end-sills, so that the bed will be scoured far enough from the dam, without endangering the structure. This is possible on a solid-rock bed, as often occurs at the head of a rapid, where the tailwater is shallow. The bed will be scoured out until the tailwater depth will reach the necessary jump height depth, and no scour will take place any more. Upturned buckets in this case were used in Conowingo and Safe Harbor Dams on the Susquehanna River, Maryland and Pennsylvania, respectively.

Another method is to raise the tailwater depth by excavating a pool, or building a low secondary dam below the main dam, or by lowering the apron. A low secondary dam creates a pool with sufficient tailwater to form a jump, the same thing being achieved by excavation. This has been extensively used on earth foundations, as in Junction Dam on the Manistee River, Michigan, where a low dam 28 ft. high was built downstream from the 52 ft. high main dam.

When the apron is lowered below stream-bed level, a curved apron can be used, as in Wheeler Dam on the Tennessee River, Alabama, or a sloping section before the horizontal apron, as in Norris Dam on the Clinch River, Tennessee. A sloping apron changes gradually the tailwater depth, until it reaches the necessary jump height. The sloping apron for class 1 will be entirely below stream-bed level. Lane recommends a slope of 1:4 or flatter, although 1:3 slopes showed fairly good results, but slopes steeper than 1:3 show a rapid decrease in efficiency.

Floor blocks may also be used in this class, although they are not recommended, since the concrete is damaged from abrasion at high velocities.



Class 2. When the tailwater is always above the jump height, the water flowing down an ogee dam will dive under the tailwater and travel at high velocity a considerable distance along the bottom, this distance becoming shorter as the tailwater approaches the required jump height.

Building a horizontal apron, raised enough above bed-stream, would produce a jump for the design discharge, but not for other discharges. A sloping apron, on the other hand, will cause an efficient jump at all discharges. This time, however, the apron would be entirely above stream-bed level. A sloping apron in this class was used in Madden Dam on the Chagres River, Panama.

A low bucket is another type used in this class. The overfalling water follows around the bottom of the curved bucket and is thrown upward by the bucket-lip and forming a surface roller in the bucket and a ground roller downstream from it. The ground roller carries bed-material upstream towards the bucket-lip downstream face, which is usually an advantageous phenomenon, but if the bed material is of a nature such as may abrade the concrete, it should be removed by excavating the stream bed far enough downstream from the structure. Such a bucket was used in Grand Coulee Dam on the Columbia River, Washington. In shallow tailwater no surface roller will be formed, but the water will be swept downstream, rising considerably upward and falling back into the stream at the surface, causing no scour. This action occurred in Dnieprostroy Dam, on the Dnieper River, U.S.S.R.

MADDEN DAM, PANAMA   GRAND COULEE DAM, WASH.   DNIEPROSTROY DAM, U.S.S.R.

Class 3. In this case the tailwater depth should be increased in low discharges to form a jump. This may be done either by a low secondary dam or sill near the downstream end of a level apron, by depressing the apron, or by a sloping apron, with its upper end above bed level and lower end below bed level.

Floor blocks or dentated sills may be used successfully if the velocity is not too great. At low discharges the blocks break up the high-velocity flow and raise the tailwater, and at high discharges the blocks break up the nappe that dives in the high tailwater and flows along the bottom at high velocity.

Class 4. In this case the tailwater depth should be increased to form a jump at high discharges. This can be done by a low secondary dam or by an excavated pool, but only when it will not cause objectionable conditions at low discharges, where the tailwater will be too high. When this is objectionable, a sloping apron will provide a better solution.

Lane's classification is based on the hydraulic jump, by classifying the stilling basins with respect to the possibility of the formation of a hydraulic jump. Although not all stilling basins use a hydraulic jump for energy dissipation, studying the formation of a hydraulic jump is however important in most cases, since the preliminary design of a stilling basin is most usually of the hydraulic jump type, and then it is changed, if necessary, according to further investigation of the site and flow and according to results of model studies. When the jump-type stilling basin is not satisfactory, either because of hydraulic or economic reasons, other types of energy dissipation are used, such as roller action, imperfect jump, etc. (Ch. I-2).

The design of stilling basins is mostly based on model studies, and to a lesser extent on previous experience, but no general design formulas were as yet developed, although some designers developed formulas for special cases of stilling basins, mostly in relatively small structures. These will be presented in the following articles, dealing separately with each case.

The principal dimension of the basin is its length. The length of a jump-type basin should correspond to its jump length, which is about 5 times the tailwater depth, according to Bakhmeteff and Matzke (3). Brown (9) points out that the theoretical length can be shortened to 40% of its original value, by proper structural means.

Stilling basins can be classified, according to their structural properties, as follows:

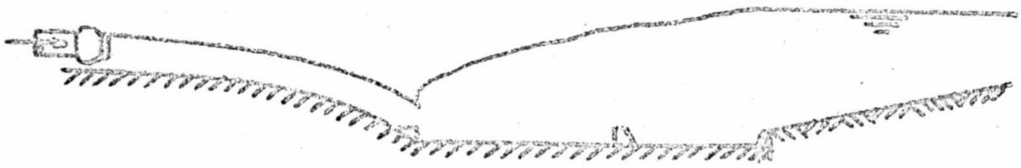
(1) Horizontal apron,
(2) Sloping apron,
(3) Chute basin,
(4) Upturned bucket,
(5) Upturned apron,
(6) Hump,
(7) Secondary low dam,
(8) Small-grain turbulence pool,
(9) Hollow bucket,
(10) Stilling pool,
(11) Miscellaneous types.

Many of these major types include certain features of other types and also some auxiliary devices, such as floor blocks, sills, deflectors, or dividing walls.

In designing basins for outlet works, four different cases will be encountered, according to the outlet elevation (Warnock, 56):

(a) When the outlet is above tailwater elevation and the channel bed is comparatively stable, a free jet may be discharged onto the tailwater which acts as a stilling pool. It may be left to the jet to scour out the necessary pool, as in Tieton Dam, or otherwise a pool may be excavated and riprapped if necessary, as in Grassy Lake and Deer Creek Dams.

(b) When outlet elevation varies between river-bed (center-line of outlet) and tailwater (invert of outlet) elevations and the channel is narrow and erodible, a chute basin is used, in which the jet flows down a trajectory-curved chute onto a level apron where a hydraulic jump is formed. Chute blocks, floor blocks and sill may be used if necessary.

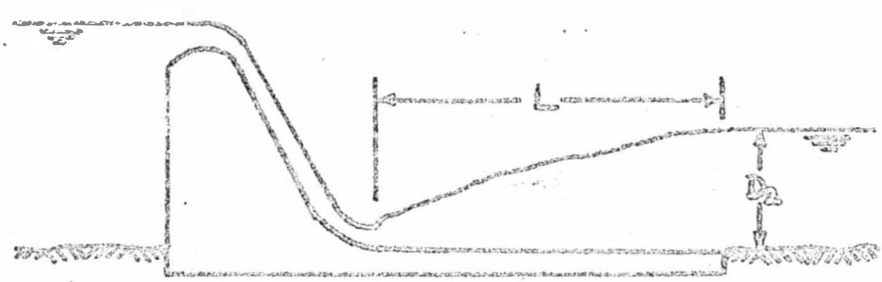


(c) When outlet (center-line) is lower than bed elevation, a hump is used. The hump has a simple curve on its upstream side and its downstream side is curved to fit the trajectory of maximum jet.

(d) When outlet is extremely low, an impact basin is used, with floor blocks and an end-sill, on which the jet impinges directly.

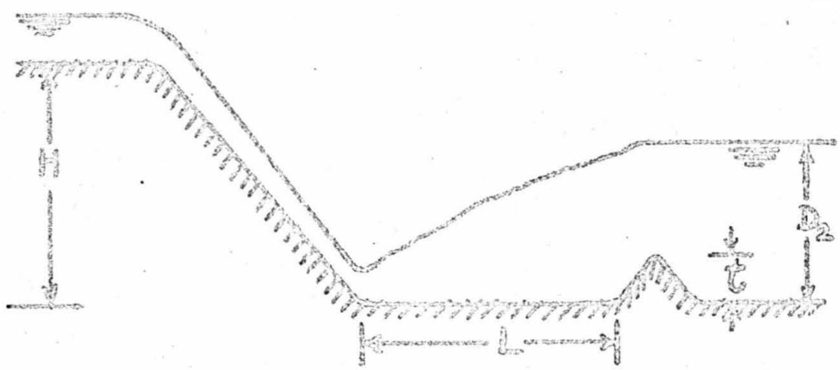
2. Horizontal Aprons, Blocks, and Sills

The conventional type of a stilling basin is a plain horizontal apron below an ogee spillway with a bucket between them. A hydraulic jump is formed on the apron and the apron should be long enough for the jump, e.i. $L = 5D_2$, according to Bakhmeteff and Matzke (3).



Such an apron was used in Wilson Dam on the Tennessee River, Alabama. It was soon realised that this apron is not the most satisfactory solution for energy dissipation. The apron should be long enough to confine the jump in its limits for all discharges, to prevent excessive scour downstream from the apron, thus requiring an uneconomical structure. To shorten the apron length certain devices were designed, such as floor blocks, chute blocks and sills. An end-sill, either solid or dentated, is most commonly used. It shortens the jump length and also throws the sheet of water upwards, so it falls back into the tailwater farther downstream, where it won't endanger the apron with the scouring capacity it still holds. Floor blocks are another means used to shorten the jump length, by increasing turbulence. They also dissipate some energy by direct impact of the flow against their upstream faces.

Schoklitsch (39) recommends the following dimensions of a basin with an end-sill, for a design discharge q:

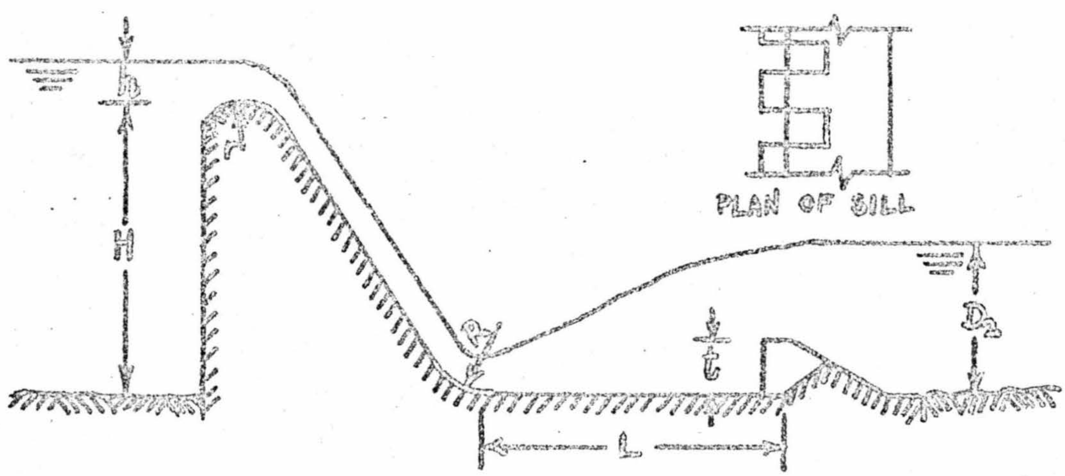


$$L = \frac{2}{3} H$$

$$t = 0.6\, q^{1/2} \left(\frac{H}{g}\right)^{1/4}$$

Forster and Skrinde (17) recommend a basin length of $L = 5D_2$ for an apron with a rectangular end-sill, which does not seem to be an improvement of the plain apron. The sill, however prevents scouring of the bed immediately below the apron.

One of the most successful end-sills is Rehbock's dentated sill. Rehbock (39) recommends the following dimensions of a basin with his sill:



$$t = 0.08\, h_{max}^{2/3} H^{1/3}$$

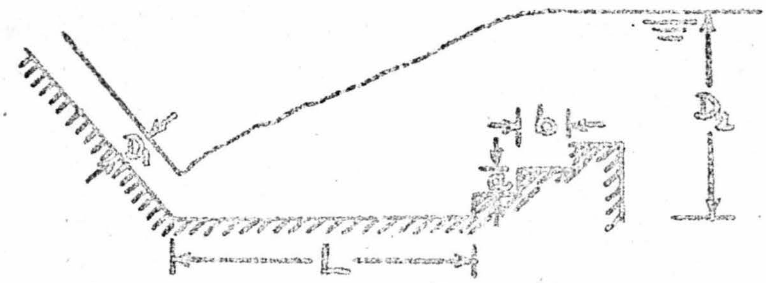
$$L = 2\, h_{max} + \frac{1}{8} H$$

$$r = 0.5\, h_{max}$$

$$R = 1.5\, h_{max}$$

Design limitations: $h_{max} < \frac{H}{2}$ and $D_2 > 1.2\, h_{max}$

Smetana (39) recommends the following design for a stepped end-sill:

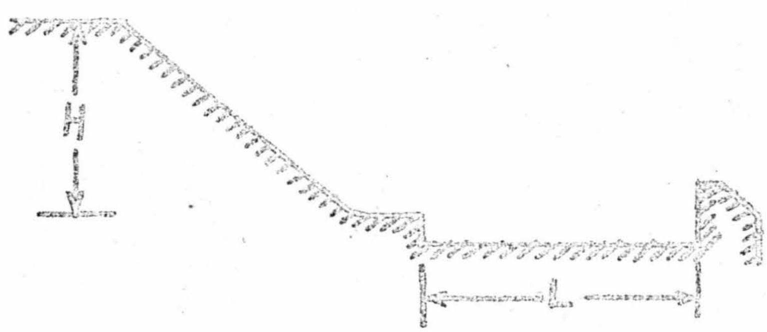


$$a = \frac{b}{2}$$
$$L = 2.72\, D_1\left(\sqrt{1+8F_{r_1}^{2}} - 3\right)$$
$$D_2 = 0.6\, D_1\left(\sqrt{1+8F_{r_1}^{2}} - 1\right)$$

Fr_1 is the Froude Number at the toe of the jump.

A stepped end-sill was used in Mississippi River Dam No. 8, where similar stepped floor-blocks were also used.

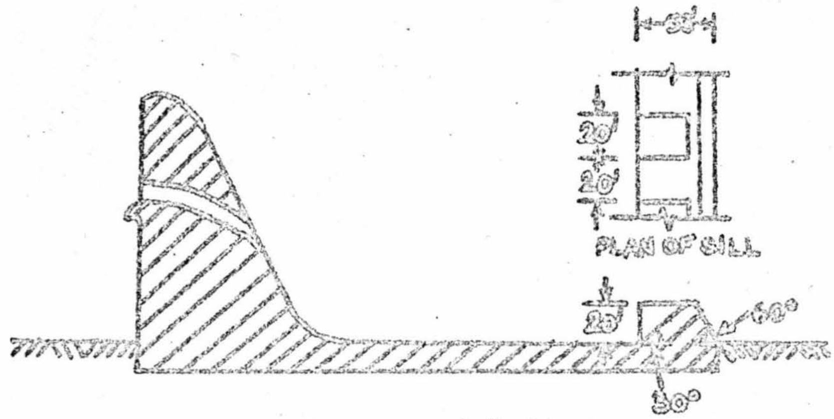
Another basin developed by Schocklitsch (39) is one with a step, serving as a chute block, and a vertical faced end-sill.



$L = \frac{H}{2}$ to H

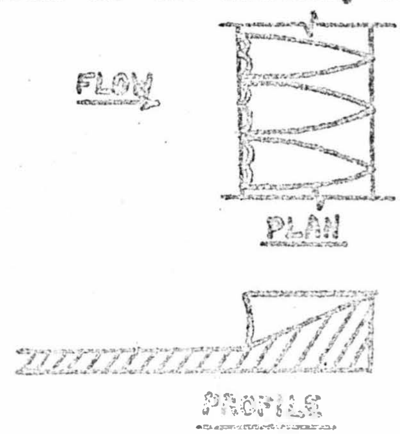
The dimensions of the basin, the step, and the sill depend on H and q in the design formulas, given by Schocklitsch.

A horizontal apron, with a dentated end-sill, was used in the service spillway of Hirakud Dam in India (10). Model studies for this stilling basin showed that the dentated end sill was more satisfactory than the solid end-sill in directing the flow into lower channel at a flatter angle, in spreading out the boil over the sill, and in reducing the violence of the ground roller. This spillway was designed to pass very high flows, and the energy dissipated in it may amount as much as 5,520,000 H.P.



HIRAKUD DAM, INDIA

Another type of dentated sill is the Hornsby sill.



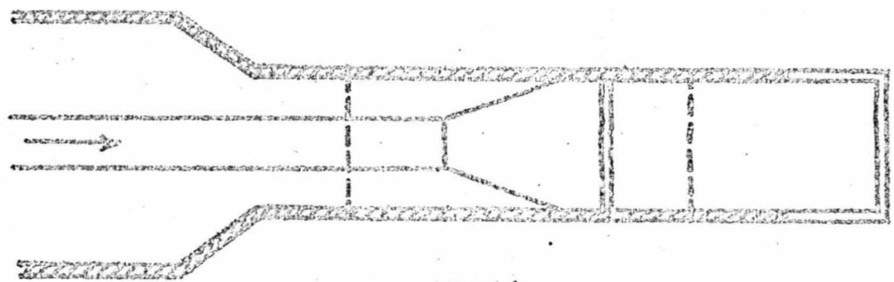
The common solid end-sill is that with a 1:1 slope on its upstream face and a vertical downstream face, as in Norris Dam.

When two transverse sills are used, Schocklitsch (39) recommends placing the first one at a distance, from the toe of the apron, equal to or greater than the head on the dam, and the second (end) sill at a distance, from the first sill, equal to or greater than 1.5 times the head on the dam. The first sill spreads the flow all over the apron and the end sill keeps the wave within the apron.

In general, the end-sill deflects the jet upwards and stabilizes the flow, while the dentated end-sill ensures the formation of a surface roller and increases the friction between the upper and the lower portions of the jet. Schocklitsch says that sills on aprons are much more efficient in prevention of scour than longer aprons would be, without sills.

Weide (63) found that the shape of an end-sill has no major effect on their required height, but the height of the sills affects the formation of a jump.

A U-shaped end-sill was developed, in St. Anthony Falls Hydraulic Laboratory, for a cantilevered ditch outlet (8). The stilling basin is horizontal and rectangular, at the end of a sloping trapezoidal ditch, and is above bed level.



PLAN

PROFILE

The first models had either one or two transverse sills (one being an end-sill), with the upstream face of the sill at 45°, 60° or 90°. With the 45° and 60° sills, the water was shooting upwards and scouring the bed far enough downstream, that the scour was not dangerous. Whirls were formed that caused much erosion, and splash was very high. The vertical sills reduced whirls and splash. A jump was formed over the end-sill, but the water was dropping over the end-sill too close to the structure. The best conditions were obtained with a cantilevered U-shaped end-sill and an additional transverse sill. Because of the splash, the rectangular section was covered, so that the wide-walls could be lowered. The overfalling water struck the bed vertically and scoured the bed deeply, but without endangering the structure. The strong whirls were eliminated and the erosion was tolerable.

Floor blocks, or baffle piers, are very much used on aprons, to increase turbulence, reduce velocity downstream, and shorten length of apron. Schoklitsch recommended a row of rectangular blocks, spaced at intervals equal to the length of block in the direction of flow (38). A stilling basin developed for the Wickiup outlet works, Oregon (53), using floor blocks, had a length of only 3 times the tailwater depth on the apron, while this depth was only 85% of the depth D_2 from the jump equation. The height of the blocks was 1/8 to 1/4 of the tailwater depth. Weide (63) made a special study of floor blocks and their effects in the control of a hydraulic jump. He used cubical blocks in his experiment and found that gravitational, inertial and viscous forces affect the flow conditions, and best conditions result when there is an optimum in the balance between these forces. A block coefficient C_B, depending on all these forces, is equal to:

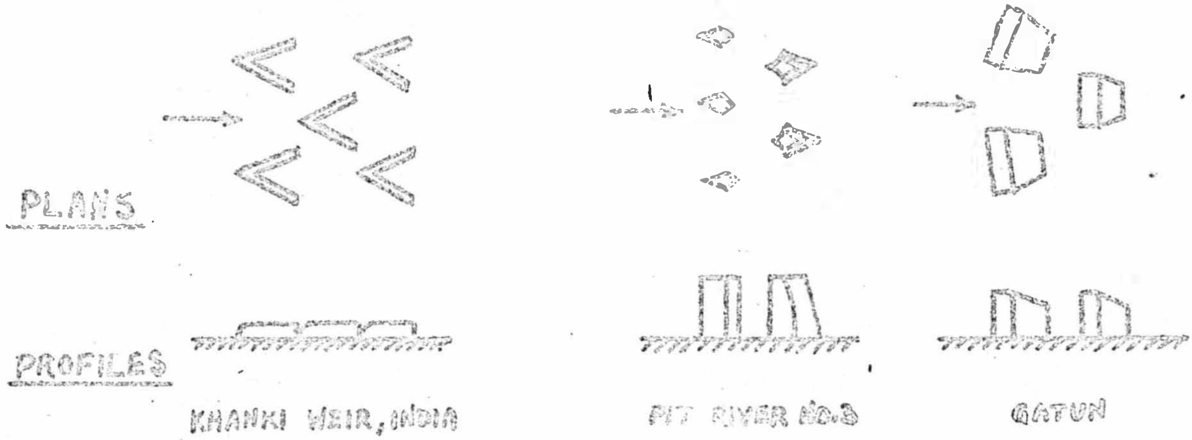
$$C_B = \left(\frac{D_2}{D_1}\right)^2 + \left(\frac{D_2}{D_1}\right) - 2F_{r_1}^2$$

Where D_1, D_2 and Fr_1 are the same as in the jump equations (Ch.I-3).

the height of the blocks was about equal to the depth D_1 at the toe of the jump. The blocks proved to be more effective when their height increased so much that they began to act as piers. The commonly used aggregate block width is 50% of the total basin width, but Weide recommends to use only 30-45%. He also found that D_2/D_1 increased with increasing L/D_2.

The common cross-sections of floor-blocks are rectangular, trapezoidal (either with vertical or sloping upstream-face), or stepped.

Other shapes of blocks (25):



When two, or more, rows of blocks are used, they are arranged so that the blocks in one row face the intervals between the blocks in the other row. The same applies to a row of floor blocks and a dentated sill.

Floor blocks are objectionable in some cases because they are subjected to cavitation and abrasion when velocity is too high or when heavy sediment, debris or ice is present in flow. They also divert flow to shoot upwards and spray into the air.

To prevent cavitation, the blocks should be placed far enough downstream, where sufficient submergence exists under the tailwater. The minimum submergence required to prevent cavitation of blocks and sills is (Creager, Justin and Hinds, 12):

$$S = j\frac{V^2}{2g} + p_v - p_a$$

Where: j = a coefficient dependent on the shape of the block or sill, determined by a model study. For an isolated block with a vertical upstream face j = 0.68 approximately.

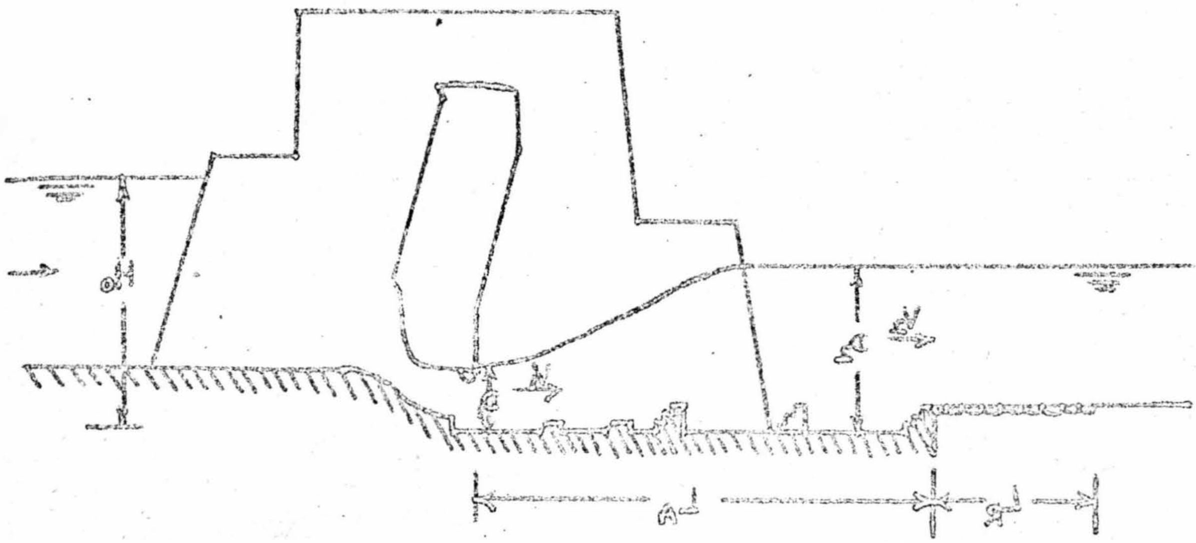
$\frac{V^2}{2g}$ = velocity head at the block.

p_v = vapor pressure of water.

p_a = atmospheric pressure.

Brown (9) presents two examples of abrasion of floor-blocks. In Conchas Dam, New Mexico, blocks adjacent to side-walls were damaged on downstream face, where eddy action caused gravel and debris in flow to grind away concrete to a depth of 12 in. In Norfolk Dam, Arkansas, the blocks were damaged on the upstream face by gravel in the flow and by impact of high velocity jets. In both cases, however, the damage was not dangerous to the structure and it was easy to repair.

For gate dams a depressed apron, with floor-blocks and an end-sill, was developed through model studies by the Corps of Engineers in the State University of Iowa (13). These dams usually operate under Class 3 conditions (see p. 11), and lowering the apron ensures a jump in most cases. The blocks and sill are helpful with either high or low tail-water.



The length of the apron, downstream from gate, is given by the following equations:

Apron without floor-blocks: $\frac{D_2}{H_0} = \frac{1}{\left(\frac{L_A}{V_2^2}\right)^{0.5}}$

Apron with floor-blocks: $\frac{D_2}{H_0} = \frac{1}{\left(\frac{L_A}{V_2^2}\right)^{0.7}}$

Length of riprap: $\frac{D_2}{H_0} = \frac{0.65}{\left(\frac{L_B}{V_2^2}\right)^{2/3}}$

where V is in fps and the lengths are in feet.

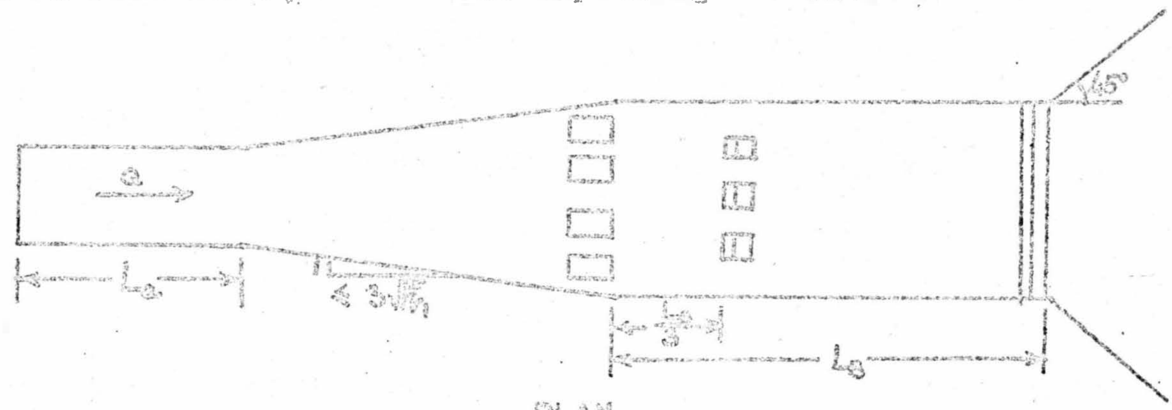
3. S. A. F. Stilling Basin

The Soil Conservation Service and Saint Anthony Falls Hydraulic Laboratory of the University of Minnesota (4, 5, 6) have designed an economical stilling basin for outlets of culverts and flumes and for turbine rooms--the Saint Anthony Falls Stilling Basin. This design has been developed by model studies, and within the range of application no further models are necessary to predict the performance of a basin of this type using the design dimensions as minimum values. The design is based on the hydraulic jump formula:

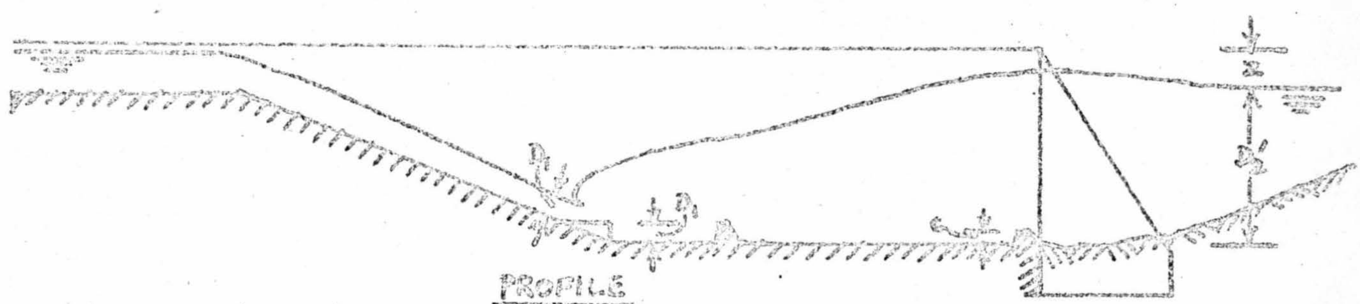
$$D_2 = \frac{D_1}{2}\left(\sqrt{1+8F_{r1}} - 1\right)$$

The range of variables in the tests was: Q = 0.04 to 21.0 cfs
F_{r_1} = 3 to 288
Re = 0.0127 to 2.1

The basin is either straight or diverging, with chute blocks, floor blocks, and an end sill on a level apron, preceded by an expanding transition. The transition causes a decrease in the flow depth, thus increasing the Froude Number and eventually reducing the basin length. Blaisdell (5) found that the saving in concrete was 61% and 41% in excavation, due to the expanding transition.



PLAN



PROFILE

Design characteristics and dimensions:

Length of open approach channel, preceding the transition:
$L_a = 5D_1$

Maximum permissible divergence of transition, to avoid excessive cross waves:

$$1:3\sqrt{Fr_1}$$

Length of stilling basin (for $Fr_1 = 3$ to 300): $L_B = \frac{4.5 D_2}{Fr_1^{0.38}}$

Height of chute blocks and floor blocks: D_1

Width and spacing of blocks: approximately $\frac{3}{4} D_1$

Distance from upstream end of basin to floor blocks: $\frac{1}{3} L_B$

Minimum distance from side walls to closest floor blocks: $\frac{3}{8} D_1$

Floor blocks should be placed downstream from openings between chute blocks.

Floor blocks should occupy 40 to 55% of the basin width.

Widths and spacings of floor blocks, for diverging basins, should be increased in proportion to the increase in basin width at the floor blocks location.

Height of end sill: $c = 0.07\ D_2$

Tailwater depth above apron --

for $Fr_1 = 3$ to 30: $D_2' = \left(1.1 - \frac{Fr_1}{120}\right) D_2$

for $Fr_1 = 30$ to 120: $D_2' = 0.85\ D_2$

for $Fr_1 = 120$ to 300: $D_2' = \left(1.0 - \frac{Fr_1}{800}\right) D_2$

maximum depth: $D_2' = 1.4\ D_1\ Fr_1^{0.45}$

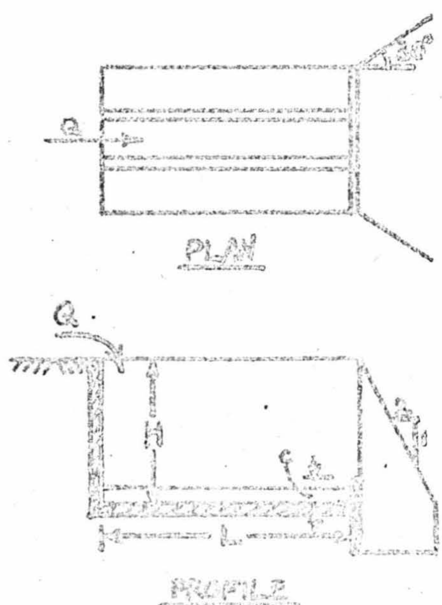
Height of side walls above maximum tailwater depth: $z = \frac{D_2}{3}$

Wing walls should have a flare of 45° and a top slope of 1:1.
Side walls may be parallel or diverge as an extension of the transition.
A cutoff wall of nominal depth should be used at the end of the basin.
Entrained air is neglected in the design of the basin.
The S. A. F. basin is most economical at the design flow, but it also performs well at any smaller flows.

4. Small Drop Structures

Drop structures are placed at intervals along a gully with a natural continuous steep gradient, in order to change the profile into a series of gently sloping reaches separated by artificial drops. A drop structure is a combined spillway (vertical drop) and stilling basin.

Morris and Johnson (28) designed a drop spillway having a horizontal apron with two longitudinal sills and an end-sill, and an excavated pool downstream.



The model tests that resulted in the final design were performed on a range of drops having a ratio of fall height H to critical depth of flow D_c from 1.0 to 15.0. As a result of their studies, Morris and Johnson presented the following design formulas:

Length of basin (for $\frac{H}{D_c} > 1.0$): $L = \left[2.5 + 1.1\frac{D_c}{H} + 0.7\left(\frac{D_c}{H}\right)^3\right]\sqrt{HD_c}$

This formula agrees with Etcheverry's (16) rational formula:

$$L = C_L\sqrt{HD_c}$$

where C_L is the coefficient of apron length. Etcheverry's coefficient was between 3.1 and 4.5.

Height of end-sill: $c = \frac{1}{2}D_c$

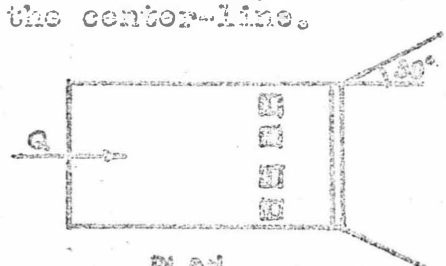
Height of longitudinal sills: $\frac{3}{4}c$

Width of longitudinal sills, for ordinary concrete construction: 6 in.

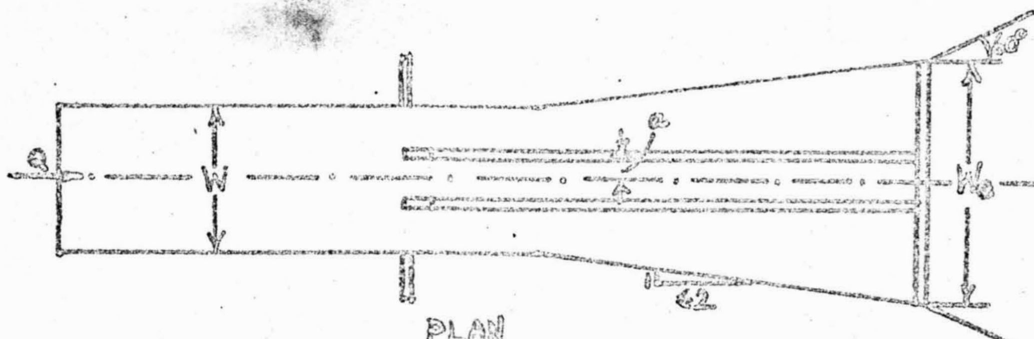
The excavated pool increased tailwater depth and provided the necessary space for silt deposit.

A similar drop spillway, the Wisconsin notch spillway, was presented by Kessler from the University of Wisconsin (22). It has 2 transverse sills instead of the end-sill, but the scour in this design was excessive.

Based on the two previous designs, the Soil Conservation Service and St. Anthony Falls Hydraulic Laboratory (8) designed the Whiting straight drop spillway outlet, for use in Whiting Field Naval Air Station. This structure has only an end sill and one row of floor blocks, and proved to be quite satisfactory in regard to scour, although some scour still occurred on the center-line.



The Box-inlet drop spillway outlet has a diverging stilling basin, with two or four longitudinal sills and an end-sill on a horizontal apron. This spillway was also designed by the Soil Conservation Service and St. Anthony Falls Laboratory (15). This is a general design of an outlet for any size of drop spillway in any size and type of gully.



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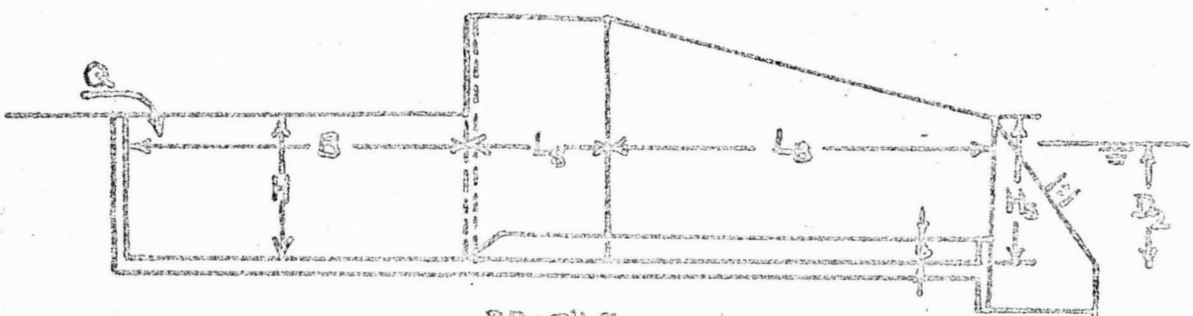


PROFILE

Design formulas:

Minimum length of straight section: $L_s = D_c\left(0.2\frac{W}{B} + 1\right)$

Where D_c is the critical depth in straight section.

Maximum flare of side walls in diverging section: 1:2

Minimum length of diverging section: $L_d = \frac{LW}{2B}$

Where L is crest length: $L = 2B + W$

The required tailwater depth -- for $\frac{W_e}{D_{ce}} < 11.5$: $D_2 = 1.6\,D_{ce}$

for $\frac{W_e}{D_{ce}} > 11.5$: $D_2 = D_{ce} + 0.052\,W_e$

Where D_{ce} is critical depth at end-sill. It is not recommended to design basins with $\frac{W_e}{D_{ce}} > 11.5$.

Height of end-sill and longitudinal sills: $s = \frac{1}{6} D_2$
Maximum width of all sills: s
When $W_e < 2.5W$, only two longitudinal sills are necessary. Distance from center-line to sills: $a = \frac{1}{6}W$ to $\frac{1}{4}W$

When $W_e > 2.5W$, four longitudinal sills are necessary. Each additional sill should be placed half the distance from center of first sill to side wall at the end of the basin.

Minimum height of side walls: $H_s = \frac{4}{3} D_2$

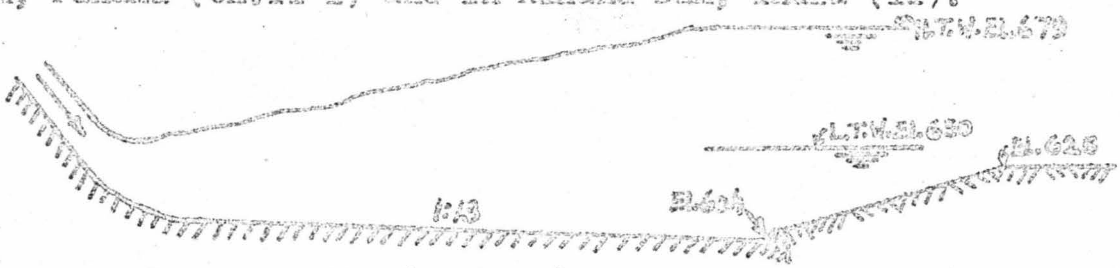
Wing walls are triangular in shape, have a top slope of 1:1, and a flare of 60°.

The tests on this structure were performed within the range of flows of 0.24 to 0.96 cfs, and fall heights H of 0.166 ft to 0.667 ft. The maximum observed scour below floor of basin was 0.1 ft.

It should be mentioned here that although trapezoidal drop structures (with inclined side-walls) are extensively used, they are not recommended, since the high-velocity flow down the sloping walls is concentrated in the center of the basin, and its energy is not dissipated (until farther downstream) by resistance in the channel (Warnock, 56).

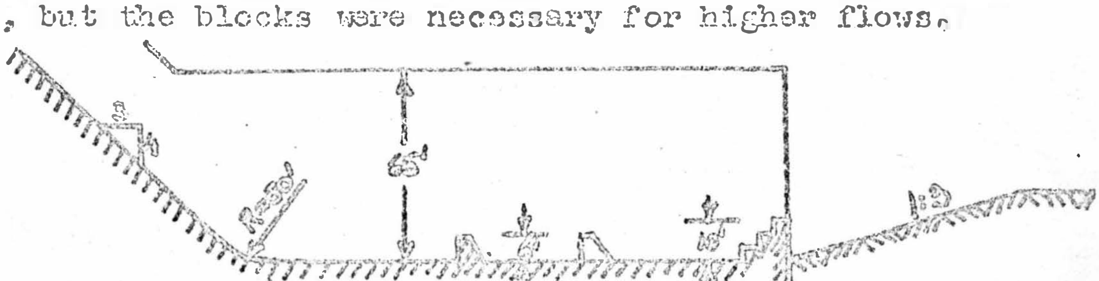
5. Sloping Aprons and Chute Basins

A sloping apron is used when the apron has to be either depressed or raised in order to change the tailwater depth. The profile may be either curved, as in Wheeler Dam, Alabama (Ch.II-1), or straight, as in Madden Dam, Panama (Ch.II-1) and in Rihand Dam, India (11).



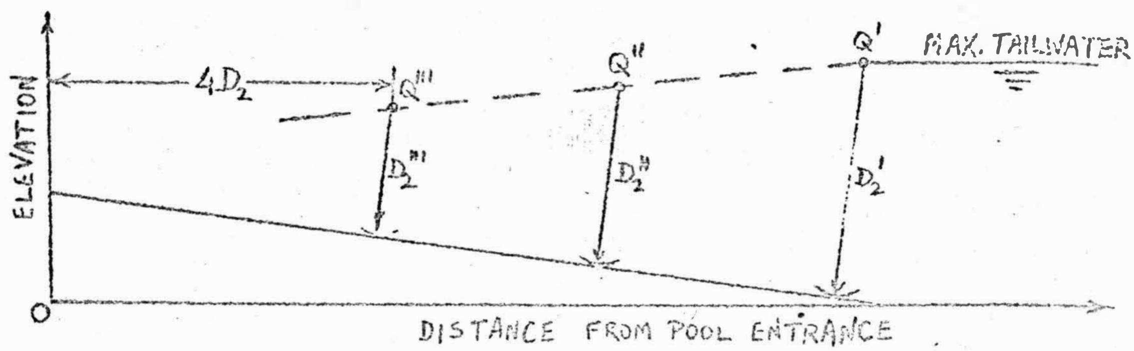
RIHAND DAM, INDIA

Sometimes a sloping section is used preceding a horizontal apron, as in Norris Dam, Tennessee (Ch.II-1). It may happen, however, that although (in the preliminary design) a sloping apron seems to fit a certain case, the model studies show later that a horizontal apron is more satisfactory, as in Detroit Dam, Oregon (60). The preliminary design of Detroit Dam had a depressed sloping apron 250 ft long with a slope of 1:10. Since the jump extended into the exit channel for the maximum flow (157,600 cfs.), the apron was lowered by steepening the slope to 1:7. The effect of lowering the apron was offset by the effect of the steeper slope, and thus there was no improvement in flow conditions. When the apron was flattened again (1:10) but made 30 ft longer, and floor-blocks and an end-sill were added, there was considerable improvement in flow conditions. The jump still extended into the exit channel, this time for low tailwater. In order to reduce the amount of excavation, a horizontal 100 ft section was tried, following a 180 ft sloping section (1:10), but the flow was still unsatisfactory. Finally a horizontal apron, 250 ft long, was tried. This design was more satisfactory, the jump being mostly confined to the apron limits. The final design had a horizontal apron 225 ft long, with two rows of floor-block 6 ft high and a stepped end-sill 10 ft high. This basin was satisfactory even without floor-blocks for flows below 100,000 cfs., but the blocks were necessary for higher flows.



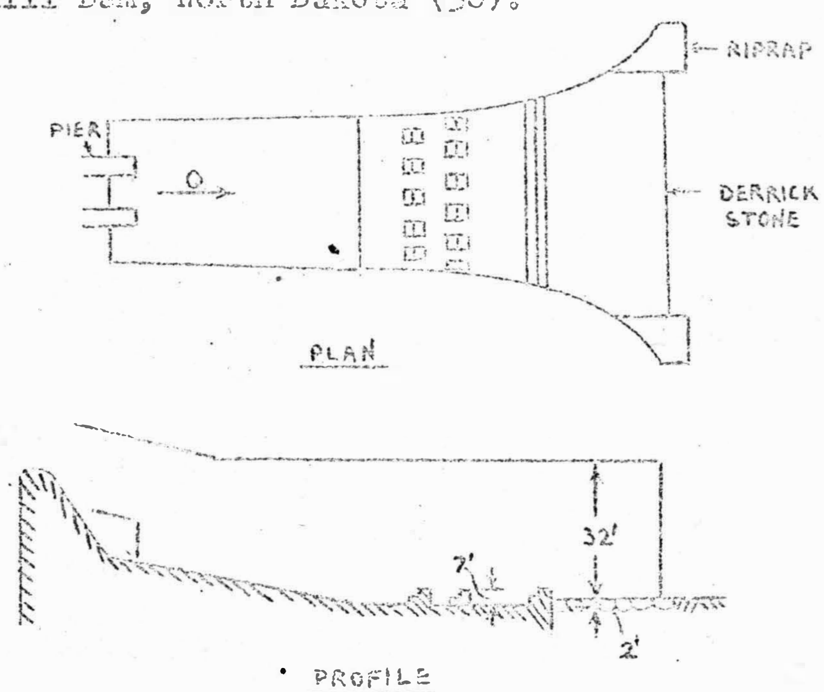
DETROIT DAM, ORE.

In Shasta Dam, California, a 1:12 sloping apron was used. This slope was found graphically, as follows (Warnock, 56):



With the pool entrance as origin, elevation as ordinate, and length of pool as abscissa, points Q', Q'',....for each discharge were plotted at a distance $4D_2$ and an elevation of the respective natural tailwater. From these points as centers, arcs were drawn with the respective depths D_2 (computed from the jump equation for a horizontal apron) as radii. The tangent to these arcs gave the sloping apron.

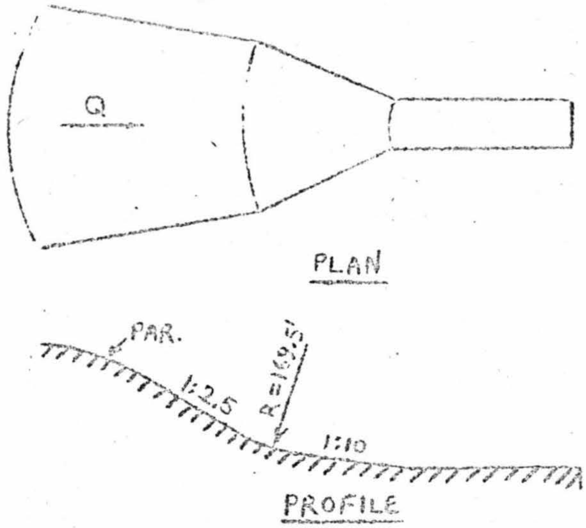
Another use of sloping aprons is in transitions, where the sloping apron serves as a chute preceding a horizontal apron. Such a chute was used in Baldhill Dam, North Dakota (36).



BALDHILL DAM, N.D.

The pier-tails were extended into the chute to reduce turbulence over the chute. The side-walls were flared to produce uniform flow in the basin. Comparison of model and prototype has shown that the jump was almost the same in both cases and the results were satisfactory.

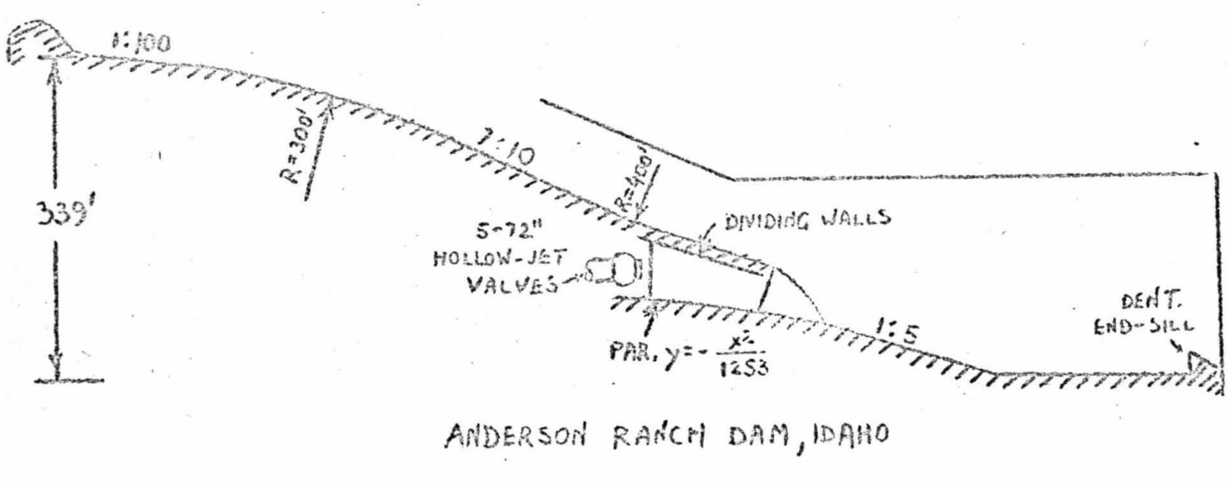
Denison Dam on the Red River, Texas (59), has a chute which is circular in plan and a combination of a parabola and two slopes in profile. This spillway handles a maximum flow of 750,000 cfs.



DENISON DAM, TEX.

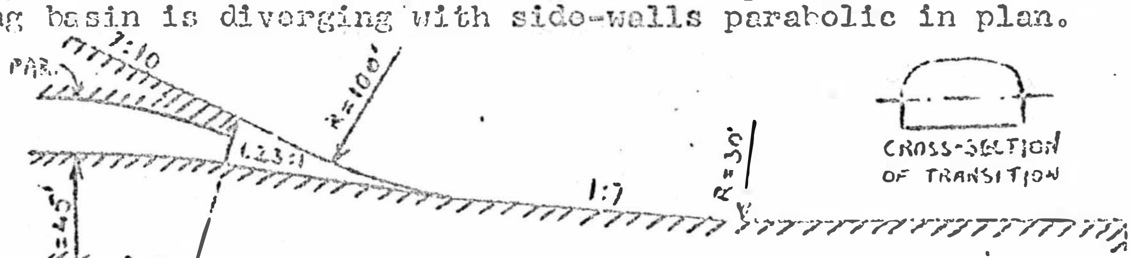
Chutes, combined of circular and sloping sections, were used in Medicine Creek Dam, Nebraska (49), Anderson Ranch Dam, Idaho (43), and Friant Dam, California (46).

MEDICINE CREEK DAM, NEB.



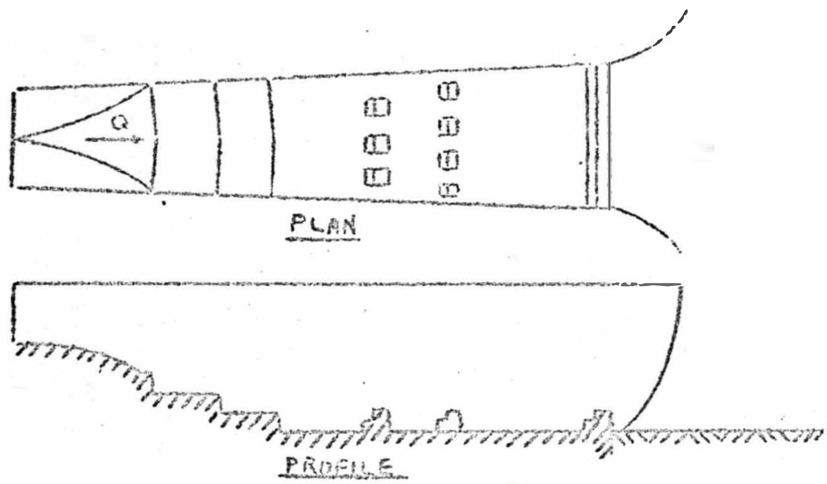
ANDERSON RANCH DAM, IDAHO

In Friant Dam there are 4-102 in. circular outlets, and the jets discharged out of them have to be spread quickly to prevent excessive velocity and severe eddies in the basin and scour below basin. To spread the jets, a transition was designed with a cross-section being elliptical above the center-line and rectangular below it. The transition profile is circular on the bottom and parabolic on the top. The stilling basin is diverging with side-walls parabolic in plan.



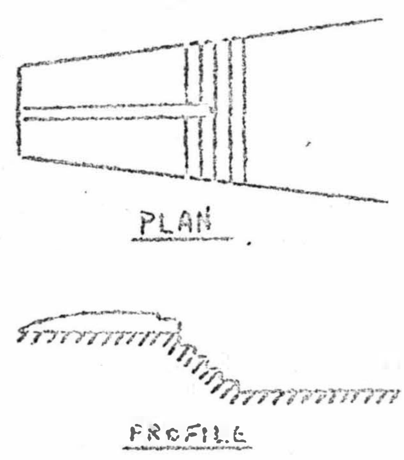
FRIANT DAM, CALIF.

In Arkabutla Dam on the Coldwater River, Mississippi (57), a trajectory chute was used, followed by steps perpendicular to diverging side-walls, leading into a horizontal apron with 2 rows of floor-blocks and an end-sill. The curved chute spreads the jet since the velocity over the chute is twice the velocity of the jet. The steps increase the resistance to flow and diminish the energy, which is eventually dissipated by a jump on the apron. The steps are perpendicular to side-walls in order to distribute the flow evenly across the basin.



ARKABUTLA DAM, MISS.

In Tappan Dam, Ohio (62), parallel steps were used, preceded by a curved hump on the upper apron, designed to prevent the drowning of the jump.

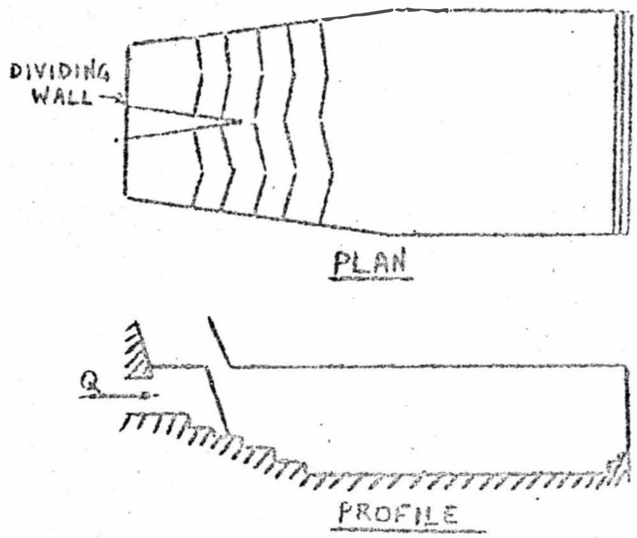


TAPPAN DAM, OHIO

In Mohawk Dam, Ohio (62), steps were made parallel to diverging side-walls and to the dividing wall between two sluiceways. (See following page.)

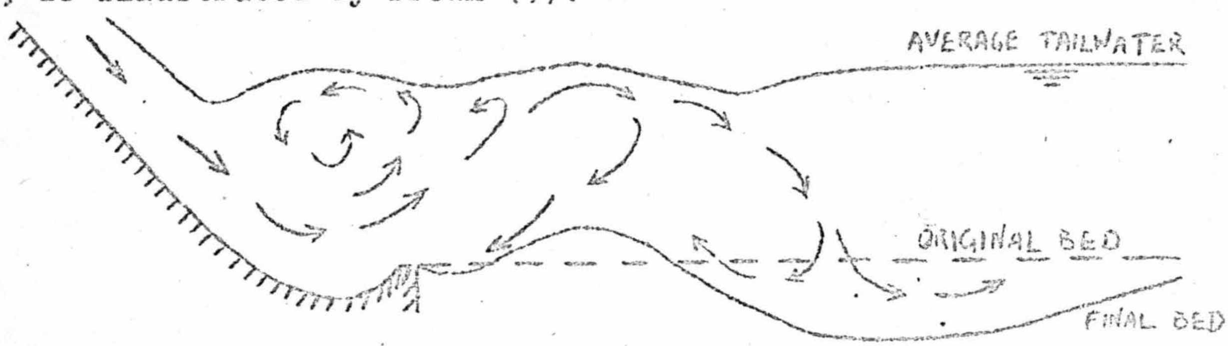
6. Upturned Buckets

The upturned bucket is extensively used where tailwater elevation is above the required jump height (D_2), in which case the bucket is a roller type dissipater. The sheet or jet of water plunges into the tailwater on the bucket, is deflected upward by the bucket-lip, and forms two elliptical rollers--a surface roller which dissipates the



MOHAWK DAM, OHIO

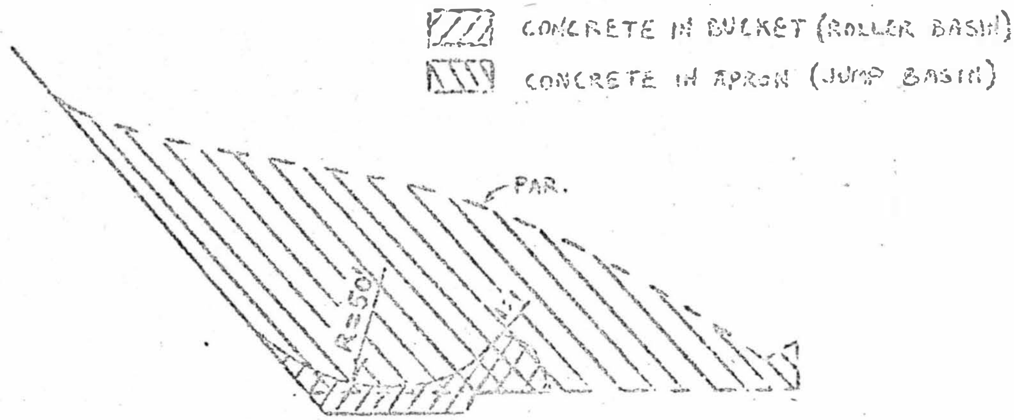
kinetic energy and a ground roller which prevents scour immediately below the bucket-lip (Ch.I-2). The typical action in an upturned bucket, is illustrated by Brown (9):



According to Brown, the most important variables in the bucket design, are: (a) the elevation of the exit channel with respect to the bucket lip; (b) the slope of the bucket lip; and (c) the height of the bucket lip. For a relatively high dam, a 50 ft radius of the bucket and a 1:1 slope of the bucket lip will ensure that the jet will be deflected away from the stream-bed. A flatter slope is likely to cause submergence of the jet in the overlying tailwater and high velocities over the exit area, resulting in excessive scour. A steeper slope will deflect the jet too steeply, resulting in a sharp dip of the ground roller into the exit bed, again producing excessive scour. For gate-dams or outlets, low dividing walls in bucket (between outlets) are recommended to prevent excessive side rollers, when the outlets do not operate symmetrically. Side rollers, or eddies, damage the concrete by abrasion from debris or rocks. See also Keener's paper on Grand Coulee Dam (21).

It was mentioned in Ch.I-2 that the amount of energy dissipated depends on the volume of the roller, or the capacity of the bucket (in this case). This is the most commonly accepted theory, but Brown disagrees with it, though he admits that he has no better theory to present.

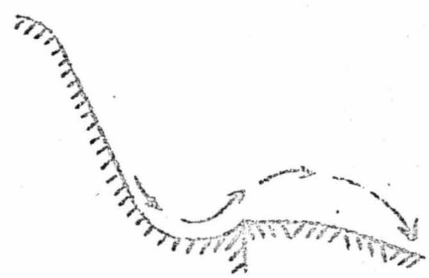
The roller type bucket was used in Grand Coulee Dam, Washington (2). A hydraulic jump basin was first designed, but it required a parabolic apron of enormous thickness. Grand Coulee Dam handles a maximum flow of 1,000,000 cfs. under a maximum head of 280 ft. A dentated bucket-lip improved flow conditions and reduced scour, but it was eliminated from the design because of cavitation and abrasion from ice and solid matter in the flow.



GRAND COULEE DAM, WASH.

This comparative figure shows clearly how much more economical the upturned bucket is than the jump basin, when the apron has to be raised considerably to enable the formation of a jump.

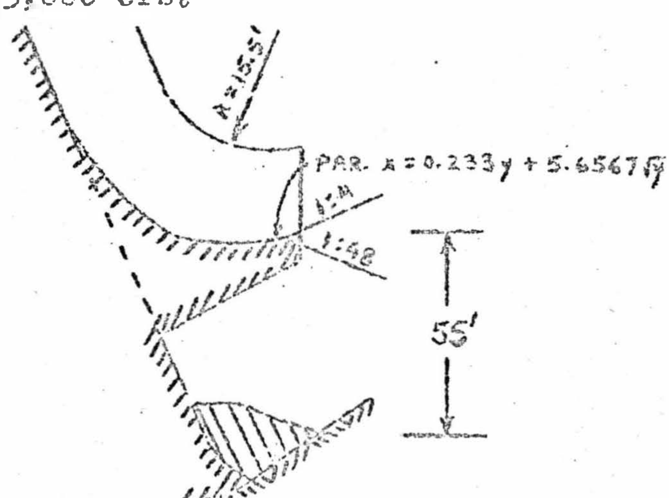
Where scour downstream from dam is permissible, a trajectory bucket is used, as in the auxiliary spillway of Hirakud Dam, India (10).



HIRAKUD DAM, INDIA

This bucket does not dissipate the energy of the jet, but throws it far enough from the dam, where it will scour out a proper stilling pool.

An elevated trajectory bucket was designed for Anchor Dam, Wyoming (42). This is a new design, departing from conventional practice in the U. S. (according to the U.S.B.R.'s report). Maximum flow on spillway is 13,000 cfs.



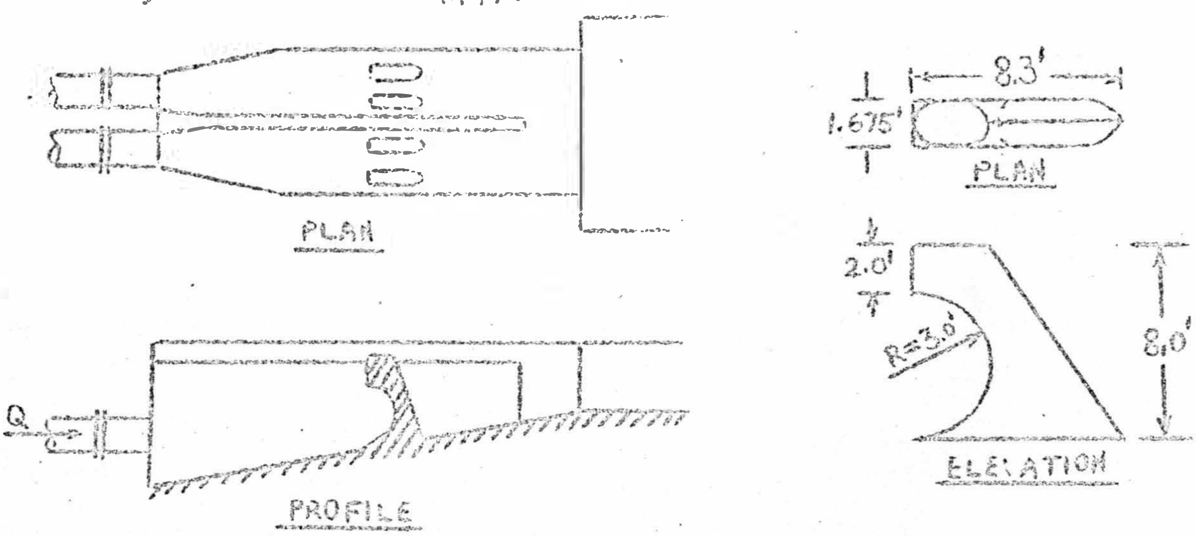
ANCHOR DAM. WYO.

The bucket was first designed circular, but changed to parabolic to provide a smoother curve without increasing the size of the structure. Drainage from the bucket is provided by the dentated lip, having 40 in. wide slots in it. The dentated lip provides a good directional control and doubles the length of jet impact-area on stream-bed, thus reducing the depth of scour. The scour area starts at 20 ft downstream from the dam, and its center is at 165 ft from the dam. Converging side-walls were used on the spillway, terminating with a 70 ft wide bucket. These walls improve the flow conditions and make a more economical structure.

7. Upturned Aprons, Humps and Secondary Dams

When outlets are lower than bed elevation, either an upturned apron or a hump, or a combination of both, is used to raise the high-velocity flow to the proper elevation, so that a jump may be formed on another apron downstream.

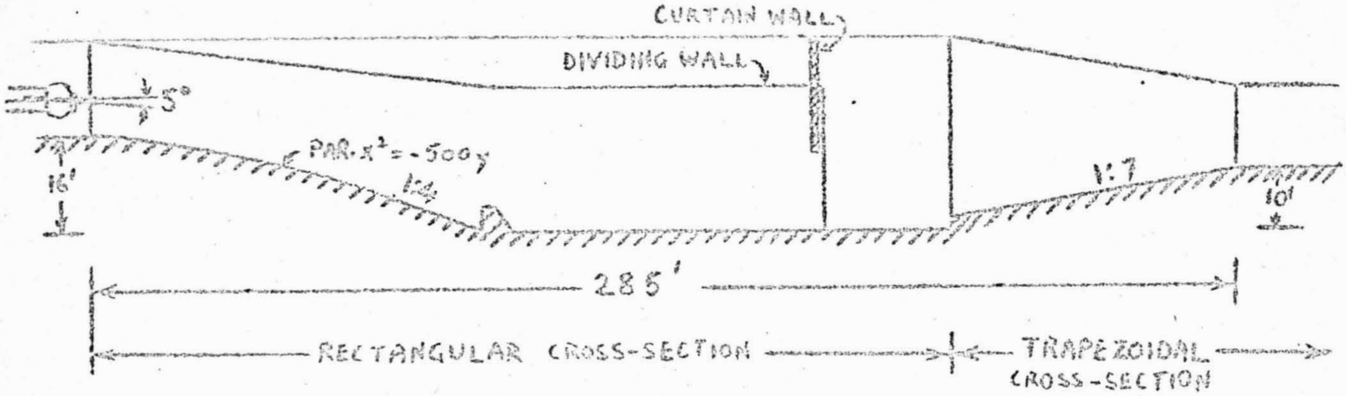
An upturned apron, with floor blocks, was used in Angostura Canal Outlet Works, South Dakota (44).



ANGOSTURA CANAL, S.D.

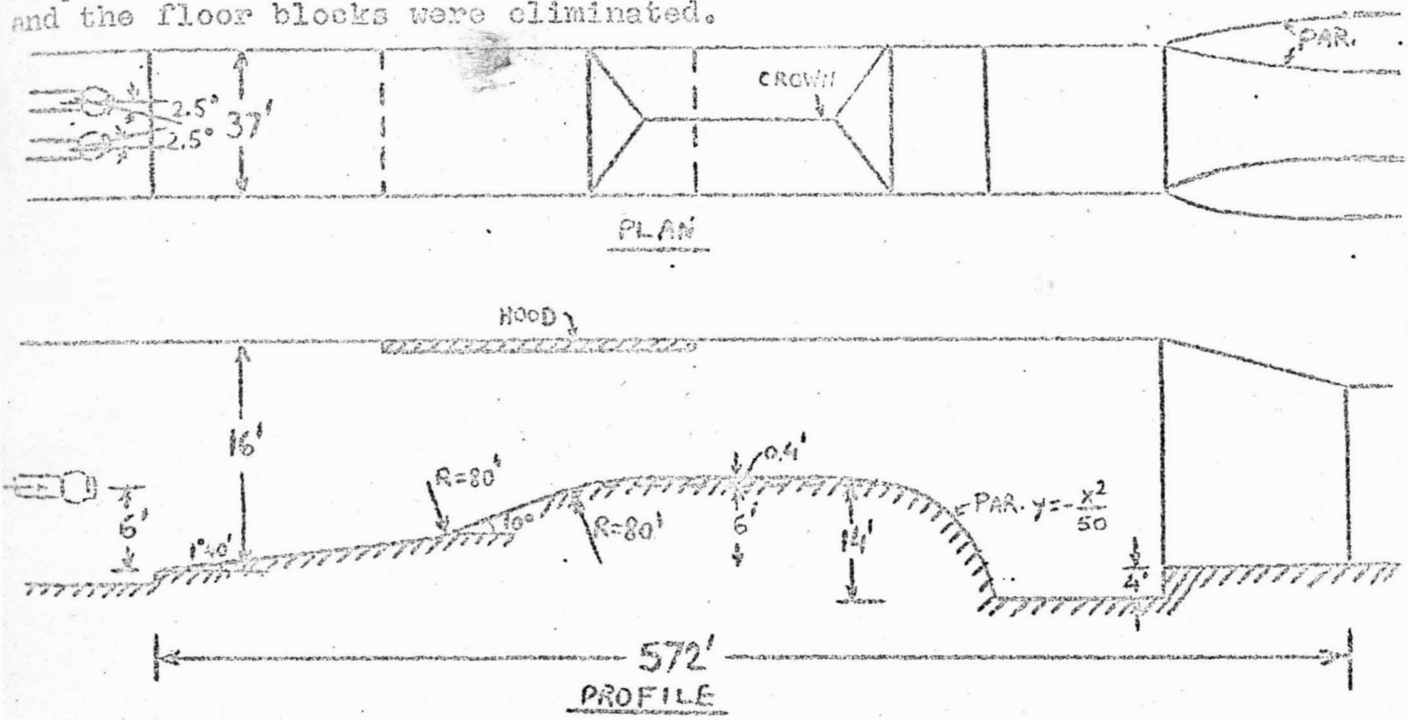
The floor blocks on this apron were of a special shape. They were projected above maximum tailwater, since lower blocks deflected jets upwards. The blocks were undercut in the middle to improve flow conditions at low tailwater. The square edges of the blocks were rounded to reduce the tendency for cavitation. Both the blocks and the dividing wall reduced excessive waves in basin.

A chute basin was first designed for the Outlet Works of Horsetooth Dam, Colorado (47), with a depressed horizontal apron, terminating in an upturned apron. A long dividing wall was necessary, between the two outlets, to prevent side-eddies in case of unsymmetrical operation, and curtain walls were necessary between side-walls and dividing wall, to reduce the height of waves. Floor blocks reduced further the height of waves, when only one valve was discharging. Better flow distribution was obtained when the valves were tilted 5° down from the horizontal.



HORSETOOTH DAM, COLO. (PRELIMINARY DESIGN)

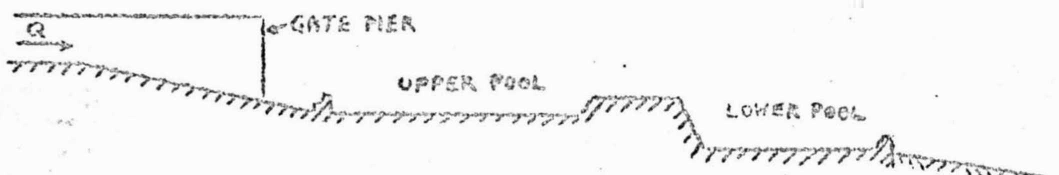
This design was satisfactory in performance but uneconomical, so a hump basin was designed instead, where the dividing wall, curtain wall, and the floor blocks were eliminated.



HORSETOOTH DAM, COLO.

The sill below the valves, the flat portion of the upturned apron and the hood were designed to reduce the excessive splash from the jets discharged on the apron. A crown was placed on the hump to improve the distribution of the flow (towards the side-walls), and thus to stabilize the hydraulic jump. The steep parabolic chute and the depressed horizontal apron, both tend to stabilize the jump.

A secondary dam is used when the tailwater is much lower than the required jump height (D_2). A stilling basin with a secondary dam was designed for Dam No. 1 on St. Lucie Canal, Florida (61).



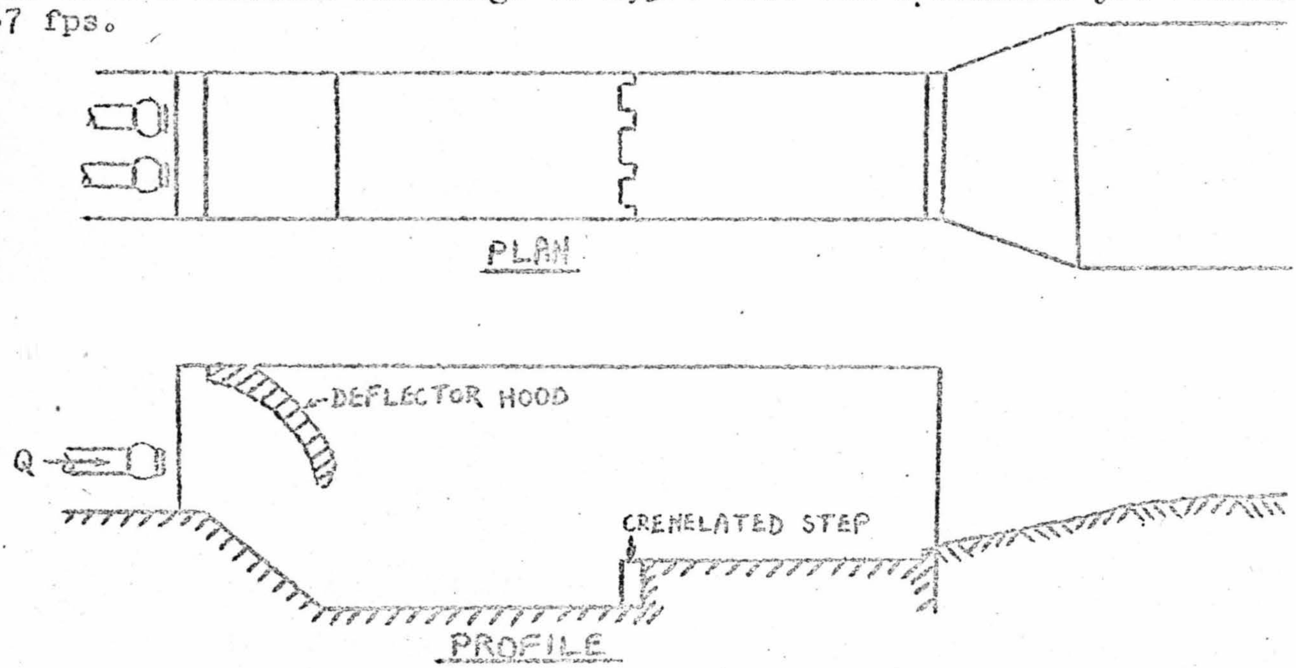
ST. LUCIE CANAL DAM NO. 1, FLA.

At high flows, tailwater depth was sufficient to form a jump in upper pool, while at low flows, the secondary low dam served as a control, holding enough water in upper pool to form one jump there, and a second jump was formed in lower pool. The gate piers were extended on the sloping apron to improve the jump in case of unsymmetrical gate operation.

8. Small-grain Turbulence Pools

Stilling basins using small-grain turbulence were recently designed by the U.S.B.R. for outlet works in Enders Dam, Nebraska, Boysen Dam, Wyoming, and Soldier Canyon Dam, Colorado (31). These basins are chute basins in which the jets are protected by a close boundary until submerged in a depressed pool, where the jets produce eddies on the bottom of the pool. This design was found to be more economical than a hydraulic jump basin which required long chute transitions and dividing walls between outlets.

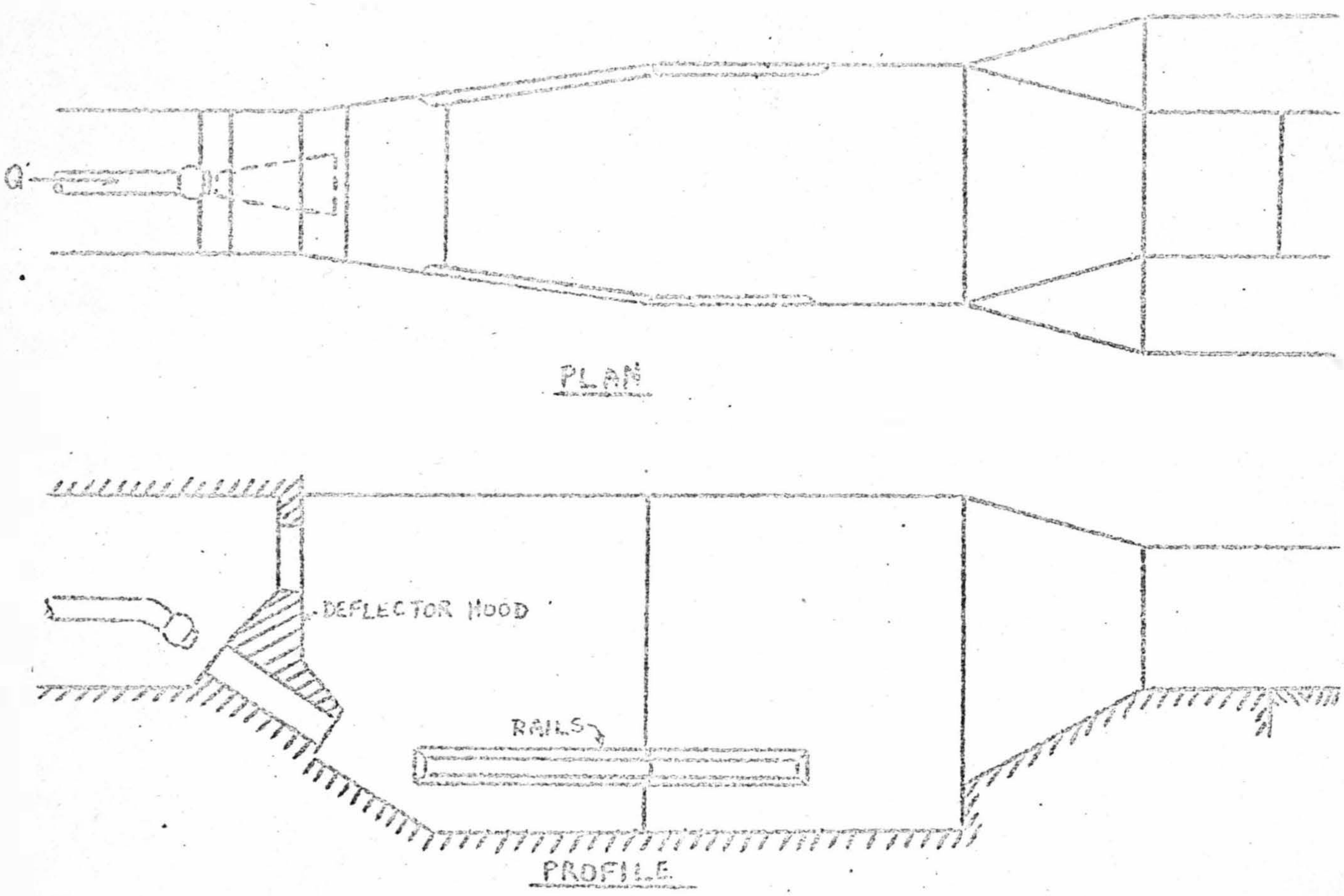
In Enders Dam Outlet Works (31) there are two 60 in. hollow-jet valves with a maximum discharge of 1,360 cfs. and a maximum jet velocity of 67 fps.



ENDERS DAM, NEB.

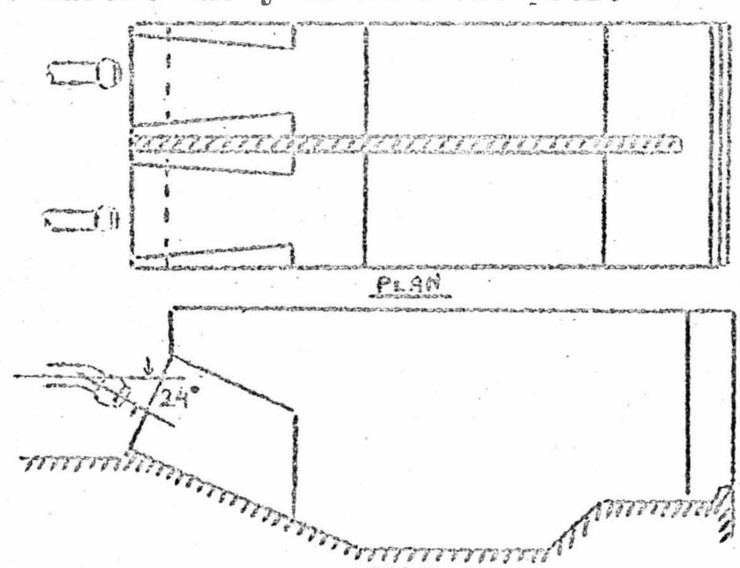
The deflector hood was curved gradually to direct the jets downwards and to prevent backing up of the flow which might submerge the valves. A steeper hood might have caused the flow to back up, but the lower lip of the hood has to be steep for adequate distribution of the flow, when only one valve is discharging. The length of this basin is only 75 ft as compared to a 175 ft long jump basin.

A similar pool was designed for Soldier Canyon Dam Outlet Works in Horsetooth Reservoir (31), having one 18 in. pivot valve with a maximum discharge of 99 cfs. and a maximum velocity of 115 fps. There the deflector hood is a circular diverging nozzle, detached from the valve to prevent high air velocities causing negative pressures on the hood. Rails were installed on the side-walls to reduce the height of waves.



SOLDIER CANYON DAM, COLO.

In Boysen Dam Outlet Works (31 and 45) a deflector hood was not satisfactory, even though the valves were tilted 15° downwards, since surface velocities were too high and the waves were excessive. Converging walls, between side-walls and dividing wall, proved to be most satisfactory, producing small-grain turbulence in the pool. The flow was very smooth and the scour slight. The valves were tilted 24° downwards to direct the jets into the pool.

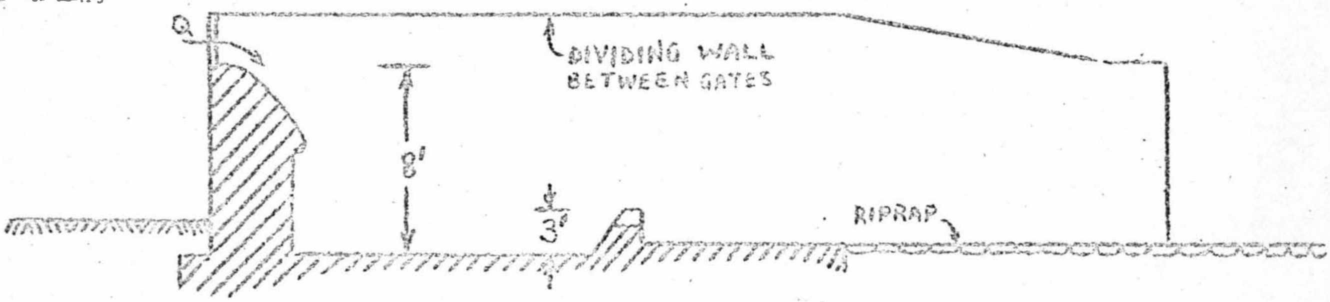


BOYSEN DAM, WYO.

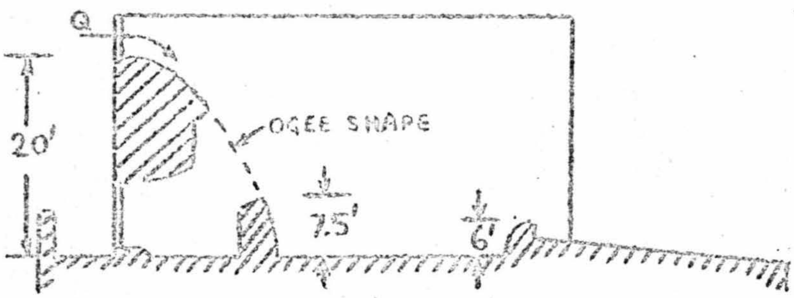
The long dividing wall was not necessary here from the hydraulic point of view, but it was necessary as a structural support for the power house on top of the basin.

9. Hollow Buckets

The hollow bucket was designed by Meyer (27) as a modification of the conventional horizontal apron below an ogee spillway. The hollow-bucket provides a cushion of water into which the overfalling sheet of water falls, and a hydraulic jump is then formed. A back-eddy, or backlash, is formed in the bucket and dissipates part of the kinetic energy of the sheet, before the jump dissipates the rest of the energy. Hollow buckets were designed by Meyer for the gate spillway of Chippewa River Dam and for the gate spillway and sluiceway of Minnesota River Dam.



CHIPPEWA RIVER DAM

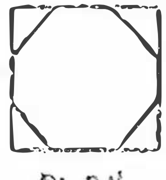


MINNESOTA RIVER DAM

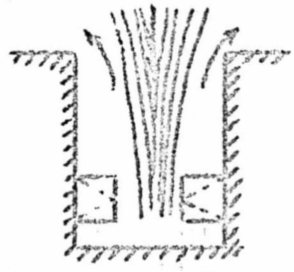
10. Stilling Pools and Miscellaneous Energy Dissipators

If scour below a dam is permissible, the jet may be left to excavate its own stilling pool. After sufficient depth is reached and the bed of the pool becomes paved with large material as a result of the sorting action, the scour will practically cease. However, if scour is not permissible, an artificial stilling pool may be provided.

The kinetic energy of a vertical jet may be dissipated by impingment of the jet with the water in the pool or with solid objects such as corner blocks.



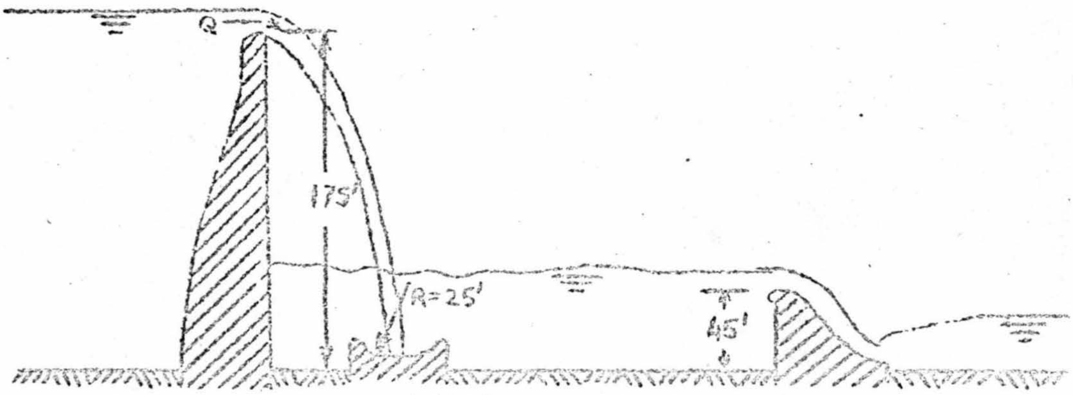
PLAN



PROFILE

When the jet falls from sufficient height, it is disintegrated in the air and falls into the pool as a widespread rain, most of its kinetic energy being thus dissipated by air resistance, rather than by impingment with the pool. This was the case in Owyhee Dam (56), where jets were discharged from 48 in. needle valves 110 ft above pool elevation.

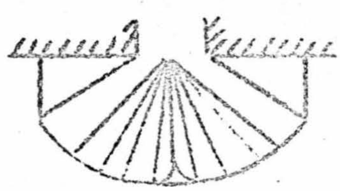
An artificial stilling pool may be created either by excavation or by the construction of a secondary dam, as in Calderwood Arch Dam (12).



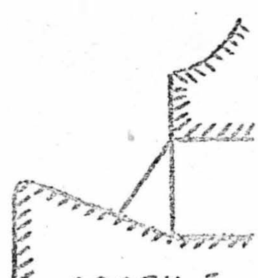
CALDERWOOD ARCH DAM

A curved concrete bucket was placed in line with the trajectory of the jet, to receive its impact and turn it up into the overlying water. For complete stilling of a vertical jet a deep cushion of water is required, and model tests of Calderwood Dam indicated that the jet lost only little of its energy prior to striking the bucket, but the performance of the whole pool was successful, both in model and prototype.

When a jet is discharged horizontally into a stilling pool, its impact on the water in the pool and its scour effect on the bottom of the pool, will be much reduced if the jet is flared out laterally, through a jet deflector, before striking the pool. Such a deflector was designed for the Bluestone Dam (12).

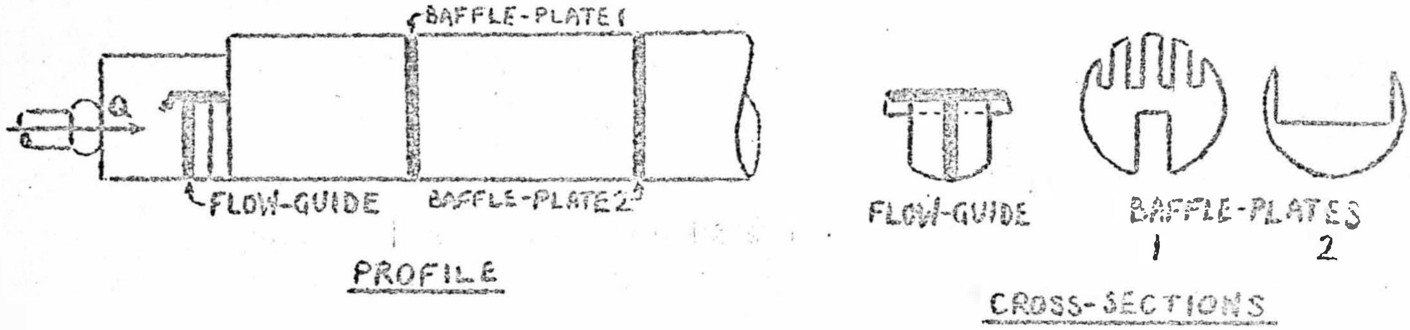


PLAN



PROFILE

A high-velocity flow in a conduit loses much of its kinetic energy by external resistance of the walls, but when more complete dissipation is necessary, artificial obstructions are placed in the conduit. In the design of Tecolote Tunnel, California (52), a flow-guide and two baffle-plates were placed below the regulating valve. The flow-guide breaks up the jet and improves flow distribution. The first baffle-plate (downstream from flow-guide) breaks up the jet further and reduces the velocity. The second baffle-plate acts as a weir to provide backwater which submerges the jet. This is necessary for low flows under high heads, when the tailwater is too low.



TECOLOTE TUNNEL, CALIF.

A rectangular stilling basin, with floor-blocks, was first designed for Tecolote Tunnel, but it required additional excavation, and thus was comparatively uneconomical.

11. Summary

This paper has presented in brief the problems of energy dissipation, as well as practical examples of stilling basin designs, using the most typical methods for dissipating the kinetic energy of high-velocity flow. The designer should remember that each structure is an individual case, and the proper stilling basin should be applied only after careful model tests of the various possibilities. Only in relatively small structures, can previous experience and design formulas be used, without any model studies. In many cases, the most satisfactory design from the hydraulic point of view may be uneconomical, and other models should be tested further to find a more economical design, which will still be hydraulically satisfactory.

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