SPILLWAYS AND ENERGY DISSIPATORS

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Jacob E. Warnock, M. Am. Soc. C. E.,
Hydraulic Engineer, Bureau of Reclamation,
Denver, Colorado.

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INTRODUCTION

In considering the subject, "Spillways and Energy Dissipators," it is assumed that a discussion is desired on the practical problems that arise in the design of hydraulic structures. From that viewpoint, I have drawn from the pool of experience in the hydraulic laboratories of the Bureau of Reclamation during the past nine years. Some of the details to be discussed, which seem relatively simple now, might have been overlooked in the design except for their being brought to light by model studies. These possible errors, once incorporated in a field structure, would have been difficult to remedy.

The dissipation of energy contained in flowing water has provided many new problems in recent years with the widespread development of irrigation systems, particularly in the western United States. It is true that for years spillways have been constructed in connection with power developments or water supply storage to pass the surplus flood water, but in many of those cases the dam was not of excessive height or was built on a good foundation capable of withstanding scour by high-velocity water. Furthermore, the impounded water was not released except through the turbines for the generation of power in the case of a power development or through the conduits for distribution in the case of a water supply development. The energy in the water due to its impoundment was dissipated by the generation of power in the power plant and by maintaining a flow in the water supply distribution system. In a well-developed power or water supply system, the necessity for flow over a spillway is infrequent and usually of short duration.

In irrigation practice, the conditions of energy dissipation are complicated by the variation of flow throughout the irrigation season and by the necessity of locating structures in erodible material. The fluctuation of demand throughout the year makes it impracticable to coordinate power generation with irrigation demand. In early irrigation systems, a low diversion dam across the river with a feeder canal on a constant grade and laterals to the individual tracts of land were all that was required to supply the low flat lands along the river. With the demand for the irrigation of lands farther away from the river and at a higher elevation, it became necessary to introduce structures in the system to conduct the water to its destination. Yesterday the primary construction was the canal itself. Today, as it will be in the future, the main consideration is that of the structures in the canal.

In a typical irrigation system of today, we have a storage reservoir usually high in the mountains where the runoff from the winter snows is impounded in the earlier spring months to be held until it is needed during the growing season on the irrigated tracts in the valley below. If the runoff exceeds the storage capacity available, the excess is wasted over a spillway to flow down the natural course of the river. If the storage reservoir is comparatively near the irrigable land, the water may be released through a tunnel or conduit directly into the main canals of the system. Otherwise, it will be released into the river channel and diverted into the main canal at the upper end of the project. As the main canal winds around the mountainside with a grade sufficient to overcome channel friction, it may ultimately be several hundred feet above the land on which the water is to be used or above the river channel which descends on a much sharper grade. A farmer on the bottom lands may be supplied water from this main canal high up on the mountainside. A conduit to his tract would seem the logical method but there are intermediate tracts to be supplied, so a lateral canal or ditch is located with sections of channel connected by drops or chutes. As the water descends the chutes or drops, the energy it contains due to its elevation, must be dissipated.

In bringing an element of water from its original impoundment in the storage reservoir to its application to the land in the small ditch through the farmer's field, only a fraction of the energy has been dissipated in maintaining flow. This is true whether we are thinking of the stored water in Lake Mead and its release at Boulder Dam to flow down the Colorado River, to be diverted into the All-American Canal for irrigation purposes in the Imperial Valley in Southern California, or whether we are considering the stored water in the reservoir behind Tieton Reservoir as it is released into the Tieton River to be diverted a few miles below into the main canal of the Tieton Project twenty miles below.

Enroute the energy in this element of water due to elevation must be dissipated to prevent scour and consequent endangering of the security of structures. Enroute this element of water may have passed through a needle valve outlet, a sluice gate outlet, or over a spillway at the storage structure; it may flow into the headworks of the main canal or it may flow over the spillway of one or more diversion dams before it is finally diverted. Enroute through the main canal it may flow through several siphons across intersecting valleys or it may flow down a chute to finally be diverted into a lateral canal and out onto the land. At each of these structures the problem of energy absorption or dissipation is encountered.

TYPES OF ENERGY DISSIPATION

In the design of hydraulic structures, types of energy dissipation may be recognized as follows:

1. External friction, such as between the water and the channel or between the water and the air.

2. Impingement, such as a jet of water striking a pool of water or a solid object.

3. Internal friction or turbulence, such as occurs within a hydraulic jump or a roller.

EXTERNAL FRICTION

External friction can be only partially controlled and can be depended upon to absorb only a fraction of the energy, but in certain instances it may be important. In the case of needle valves or conduits discharging at a considerable height above the stilling pool, the friction between the jet of water and the air literally disintegrates the stream before it strikes the water surface below. At the Owyhee Dam (figure 1) the 48-inch needle valves discharge horizontally into the air 110 feet above the tailwater. The disintegration of the jets has to be seen to be realized. The "rain" beneath them equals in intensity the downpour during a cloudburst and it falls all along the area beneath the jets. The stream itself shows very little solid water but instead resembles a dense spray. As it strikes the water surface it appears to ricochet and strike a second time farther downstream rather than plunge into the pool. This ricochet is due to the lesser density of the air-water mixture as compared to the solid water in the pool. The absorption of air due to friction causes the jet to impinge over a wide area and with far less force than a jet of solid water.

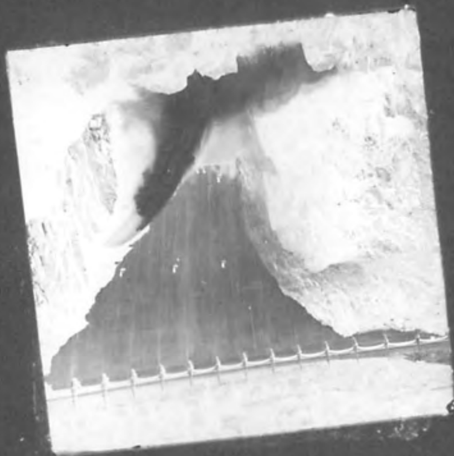

Figure 1 - Discharge from Owyhee Needle Striking the Tailwater Surface.

IMPINGEMENT

Direct impingement as a means of energy dissipation is not utilized except as a last resort when other methods fail. A recent study of a design for the Conchas Dam irrigation outlet works (figure 2) ended in impingement as a solution. Excessive tailwater caused by the bottom of the canal downstream from the outlets being located several feet above the conduits rendered a hydraulic jump or a roller design out of the question. Each of the two conduits will discharge 350 second-feet under a maximum head of 64 feet, or one outlet may discharge 700 second-feet under the maximum head. With these conditions the exit velocity will be

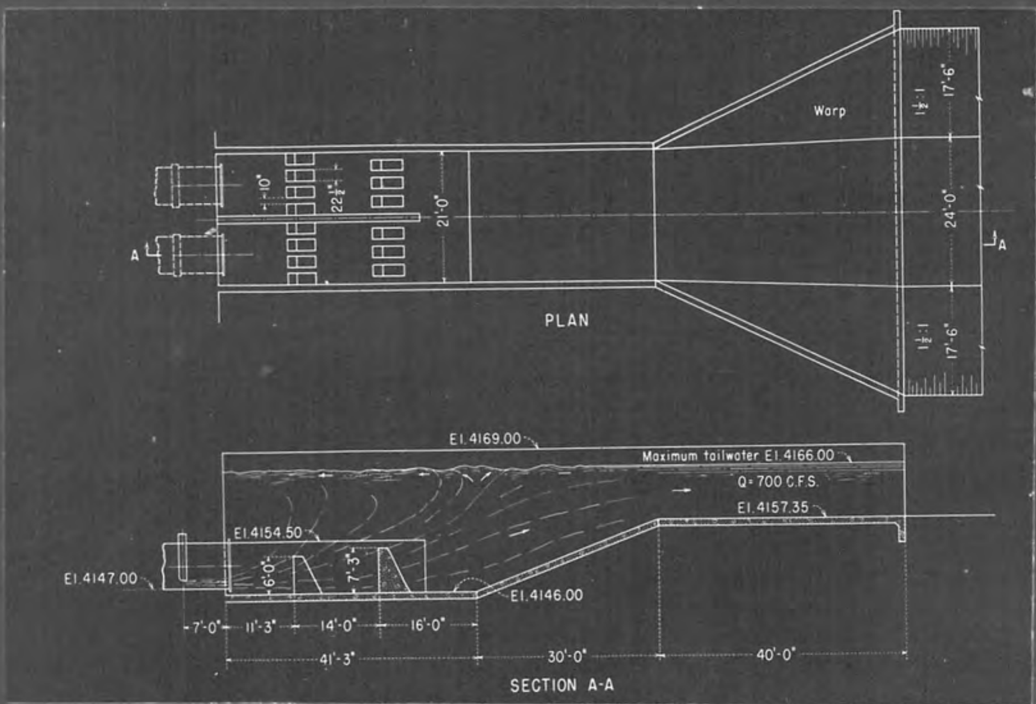

Figure 2 - Stilling Pool for Irrigation Outlet Works at Conchas Dam - Tucumcari Project.

very high, and since excessive tailwater prevents the formation of a hydraulic jump, baffle piers were located in the path of the high-velocity jet to disperse the velocity. The center training wall is necessary to prevent return eddies when one gate only is operating. The chief objection to impingement as an energy dissipator is the difficulty of inspection and maintenance of the impact surfaces. In this case, the pool can readily be unwatered during the winter months and necessary repairs made. The upstream face of the piers will be armored with standard 18-inch channels with the legs imbedded in the piers.

INTERNAL FRICTION

Two of the most convenient and practical methods of eliminating the energy of flowing water are the roller and the hydraulic jump. The application of the hydraulic jump is more general because the roller requires an excess of tailwater to form properly.

ROLLER STILLING POOL

An example of a roller as a means of energy dissipation is the stilling pool designed for Grand Coulee Dam (figure 3). With a designed capacity of one million cubic feet per second and a difference in head of 280 feet, the energy to be dissipated will be 31,800,000 horsepower, or 19,300 horsepower per foot of gross

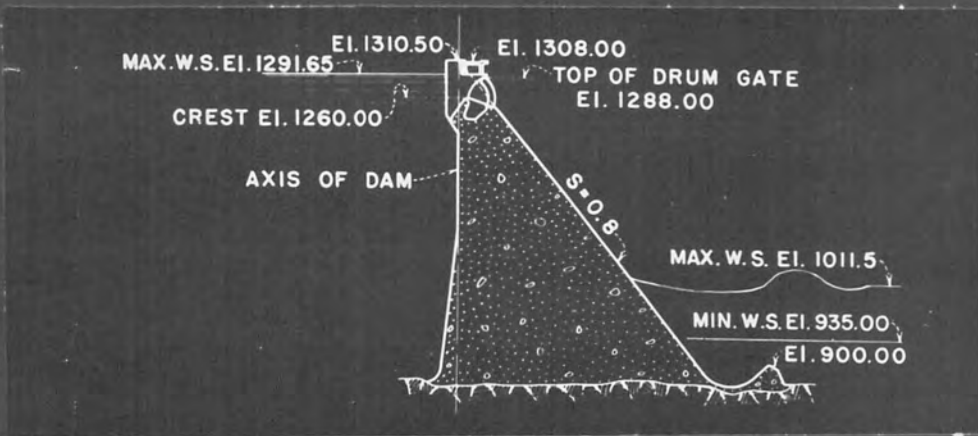

Figure 3 - Section Through Spillway of Grand Coulee Dam.

spillway crest length. An excess of tailwater depth made a hydraulic jump pool impracticable due to the enormous cost of a sloping apron whereby an agreement could be obtained between the tailwater curve and the jump-height curve. An apron curved in section (figure 4) and placed at a very low elevation was developed. In the final design of this curved apron, or bucket, the 50-foot radius was tangent to the 0.8 slope of the spillway face and to a 1:1 slope on the downstream portion.

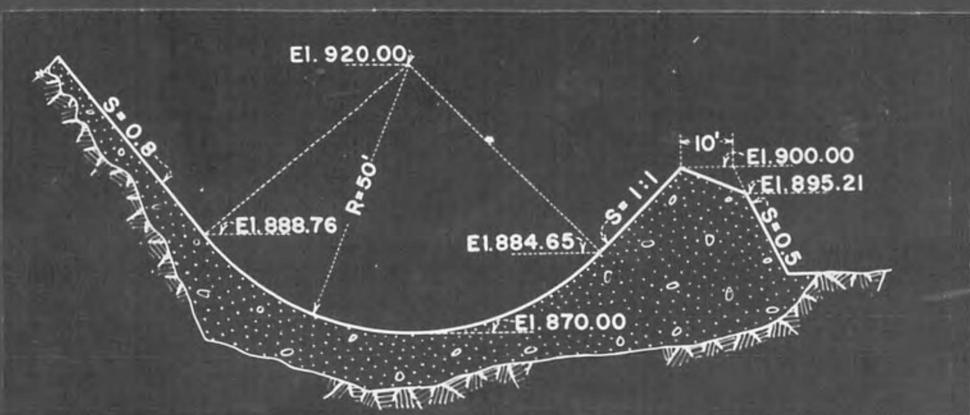

Figure 4 - Section Through Grand Coulee Stilling Pool.

The behavior of the internal mechanism of the roller was clarified by velocity traverses (figure 5) within the bucket. As the descending sheet of water plunged into the tailwater, a large reduction of velocity occurred and the sheet diverged in the same fashion as in the transition zone of a hydraulic jump. After the stream entered the bucket proper, it was diverted with a practically constant velocity to the top at elevation 900.0. The effect of the

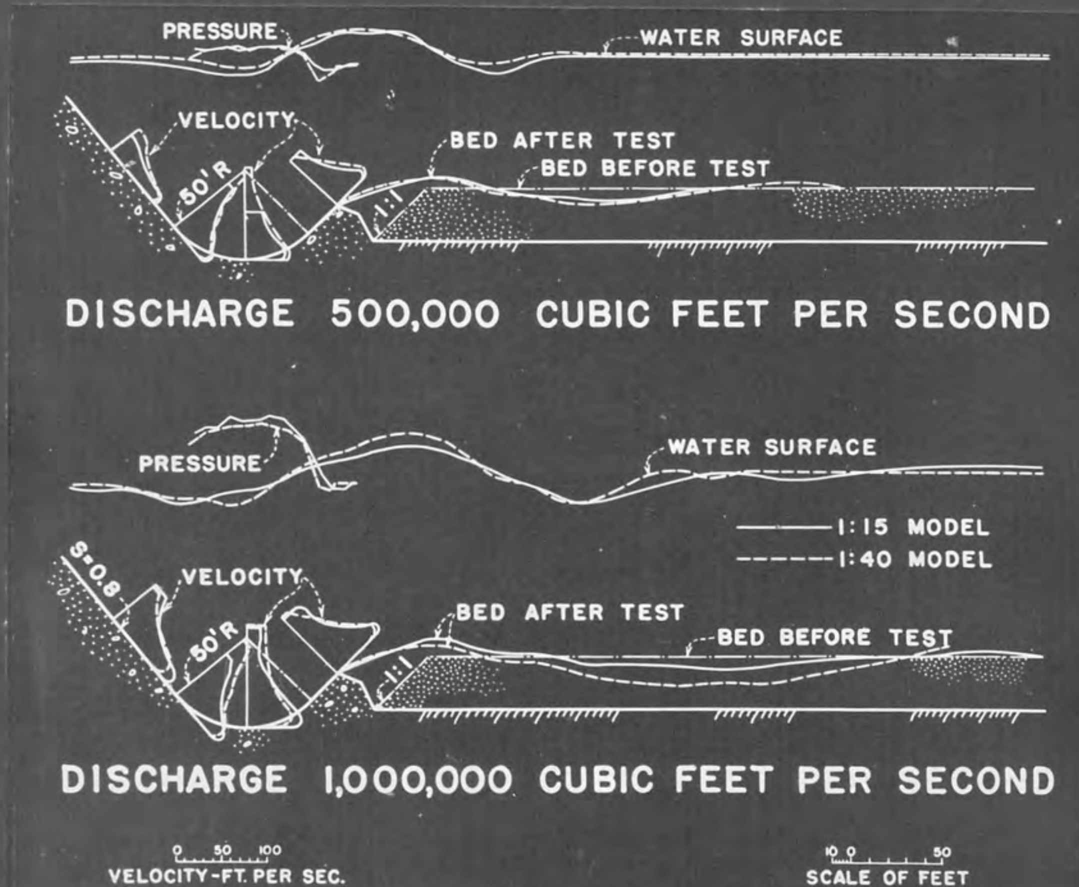

Figure 5 - Flow Characteristics in Grand Coulee Bucket.

lip is to divide the jet into two parts, one of which is deflected upward to form the elliptical surface with its major axis in a horizontal plane above the bucket; the other is turned downstream to form the ground roller.

The two rollers are inseparable. The surface roller is caused by the surface water flowing in to replace that carried away by the incoming sheet and the ground roller is caused by the inflow of water at the lip to replace that carried away by the jet as it passes the lip. The turbulence, entrained air and instability of flow makes it impossible to trace completely the dissipation of energy, but the major part is dissipated in the surface roller. However, the direction and intensity determine the amount of scour in the stream bed. A slope steeper than 1:1 on the upstream face of the lip produced a too nearly vertical deflection of the jet, with the result that the ground roller dipped sharply, scoured excessively and decreased the effectiveness of the surface roller. For slopes flatter than 1:1, the surface roller tended to sweep out of the bucket and the ground roller was obliterated by a downstream current which also produced excessive scour.

The extent of velocity reduction, and hence energy dissipation, can be appreciated by an examination of the velocity traverses made in the stilling bucket. With a spillway discharge of 500,000 second-feet, the maximum velocity measured in the model at the point where the descending sheet enters the pool and converted to prototype was 133.0 feet per second. This was a reasonable agreement with the calculated velocity of 141 feet per second. At the point of tangency, the maximum measured velocity was 55 feet per second.

The model studies showed conclusively that the efficiency of the roller depends upon its form. A circular roller such as was obtained by using a 30-foot radius bucket was not as effective as the elliptical roller formed by the 50-foot radius bucket. In the circular roller the peripheral velocity appeared to be approximately the same as the velocity of the boundary layer of the jet, while in the elliptical roller the peripheral velocity was somewhat less. The difference in velocity decreased the efficiency of the roller as a roller and increased its efficiency as an energy dissipator. The lesser velocity in the elliptical roller can probably best be accounted for by the greater internal absorption of energy caused by the greater number of eddies, whirls and vortices formed.

In a recent issue of the Saturday Evening Post, Garet Garrett in his article "Great Works" described the Grand Coulee spillway as follows: "After it (meaning the Columbia River) has been obliged by its own hand to split off one seventh of itself and give it to economic slavery, what is left to go over the dam will represent 32,000,000 horsepower. It may be thinking that that much of itself will be free to take revenge. It does not know. These 32,000,000 horsepower will be caught in a silly concrete trap called a surge bucket at the base of the dam, and all thought of revenge will be churned out."*

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HYDRAULIC JUMP STILLING POOL

The most widely used method of dissipation is the hydraulic jump stilling pool. Excellent examples of its application in recent years are the Norris Dam, constructed by the Tennessee Valley Authority; the Madden Dam, built by the Panama Canal Zone; and the Shasta Dam (figure 6) now under construction of the Bureau of Reclamation.

The all-important criterion in the design of a hydraulic jump pool is to obtain, insofar as feasible, an agreement between the tailwater for a given discharge with the height of water necessary to form a perfect hydraulic jump. Nature seldom is so kind as to provide conditions whereby this agreement can be exactly attained. As an alternate, the floor of the pool may be located so as to produce an approximate coincidence between the tailwater curve and the jump-height curve.

At Shasta Dam, if a horizontal floor were placed at sufficient depth to form a perfect jump at maximum discharge, there would be an excess of depth at lesser discharges. In studying the profile of the stilling pool, three methods of analysis were used:

(1) Visual observation of the behavior as seen through the glass side of a sectional model.

(2) Pitot tube velocity measurements in the model at the downstream end of the pool to determine the depth for a given discharge at which maximum reduction of velocity occurred.

(3) Computation of the slope of the floor by assuming a ratio between length of jump and depth at the downstream end of pool and finding the locus of the downstream depth.

In the original model, the floor was horizontal with a 2.5:1 slope at the upstream end.

In the first method of analysis, the tailwater was varied until the hydraulic jump appeared most efficient, while in the second, velocity measurements were made with a pitot tube placed near the end of the apron.

In the second method, a point of minimum velocity occurred when the jump was most efficient. A decrease in tailwater caused the jump to move downstream, thus increasing the velocity, while an increase in tailwater elevation drowned the jump, causing the jet to dive under and retain more of its initial velocity. The elevations determined for various discharges by both methods were plotted and the results found to be the same for all practical purposes. The jump-height curve obtained in this manner was considerably above the normal tailwater curve for the high discharges and below for the low discharges. This fact indicated that the original apron was placed too high for most efficient operation at the high discharges. Only at intermediate flows did an efficient jump form. Moreover, the pool surface was rough at all discharges.

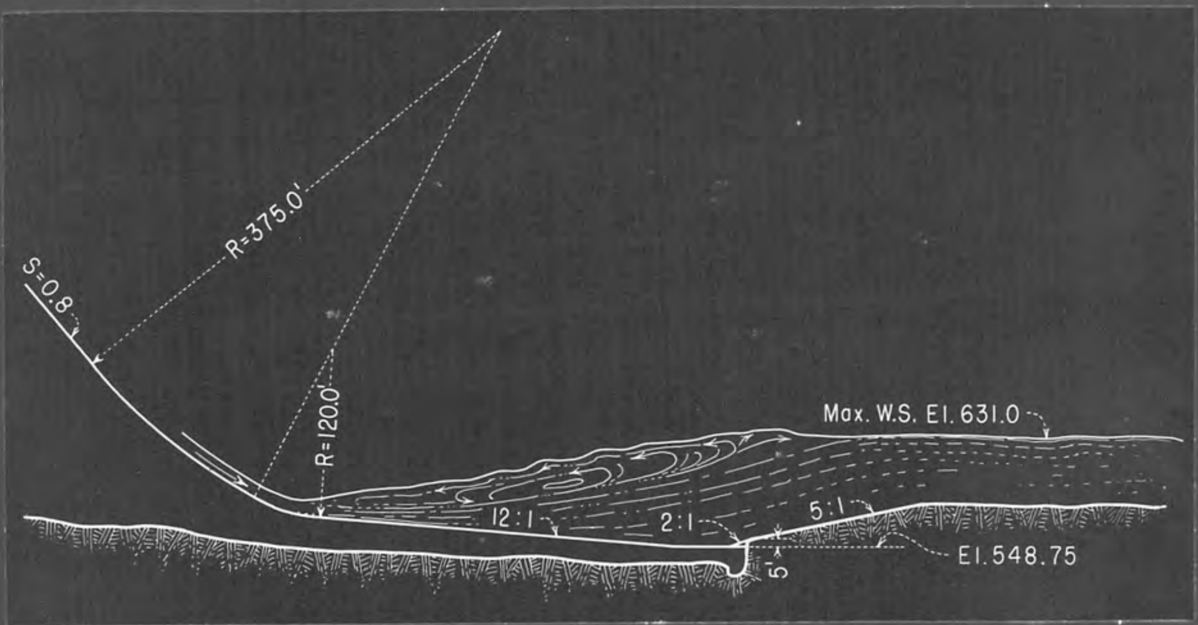

Figure 6 - Final Design of Stilling Pool for Shasta Dam.

An 8:1 sloping apron improved the action so that the jump could not be drowned on the model and the tailwater at which minimum velocity occurred could not be reached. Even so, the velocity at the end of the apron was equal to or less than the minimum for the original design. The jump-height curve obtained visually coincided with the natural tailwater curve for all flows except those near the maximum of 250,000 second-feet where five feet additional depth was found necessary to give satisfactory conditions.

A 12:1 apron, which deepened the pool at the upstream end, corrected this condition. The jump had an efficient appearance for all discharges and it was noted that any appreciable increase in tailwater depth for the maximum discharge gave a rougher pool surface. It was noted also that considerable reduction in tailwater depth gave rough but satisfactory conditions, thus indicating a factor of safety.

In the computation (figure 7) of the slope of the apron, the length of jump was conservatively assumed to be four times the depth downstream from the jump. Using the momentum formula for the

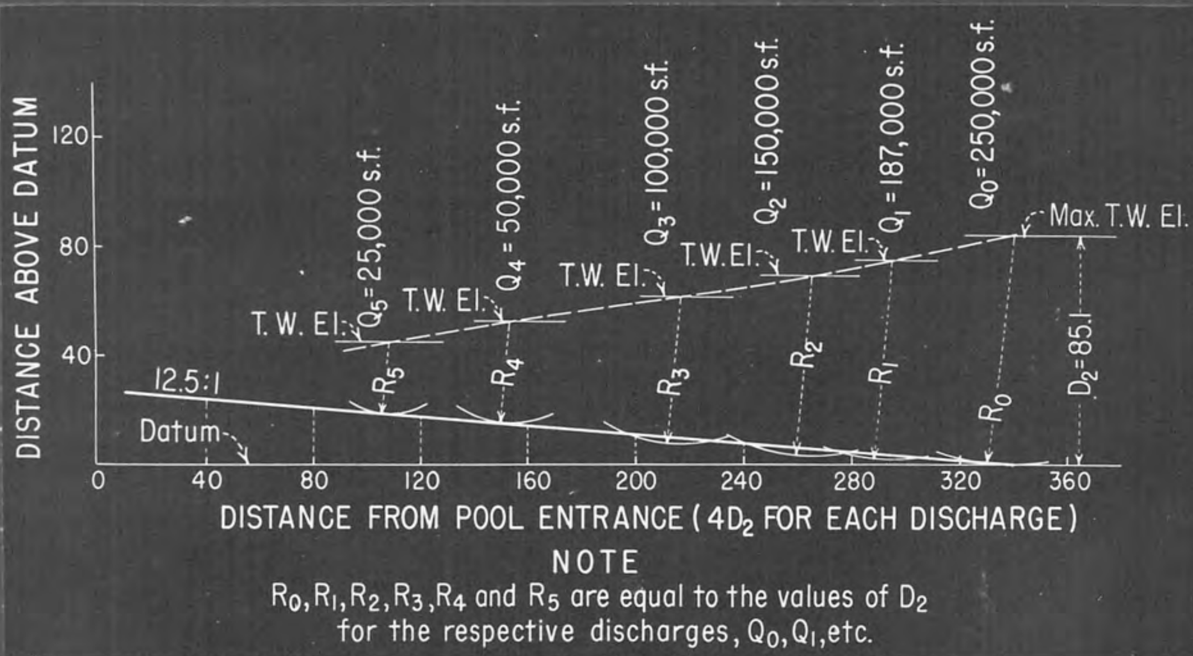

Figure 7 - Design of Sloping Apron for Shasta Dam.

hydraulic jump

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

the values of D_2 for different discharges were computed. With the pool entrance as an origin, length of pool as abscissa, and elevation as ordinate, the tailwater elevation for each discharge was plotted a distance of 4 D_2 from the origin. With these points as centers and radii equal to the respective D_2, arcs were drawn below. A line tangent to these arcs is the profile of the stilling apron to be used. The slope of the Shasta apron computed by this method was 12.5:1. A slope of 12:1 had been previously determined by the model studies.

DISTRIBUTION OF FLOW

Experience has taught us that to secure a stable and uniform jump, the water entering a hydraulic jump stilling pool must be uniformly distributed with no return flow along the sides of the pool.

This requirement in the case of small structures, particularly in canals, has led to a certain amount of conflict between the structural and the hydraulic engineer. It is cheaper to pave the sides of the canal to form a so-called "trapezoidal pool" than it is to construct vertical retaining walls.

In 1937, the problem of the spillway for the Dos Bocas Dam being constructed by the Rural Electrification Division of the Puerto Rico Reconstruction Administration was undertaken by the Bureau of Reclamation upon the recommendation of the consulting board. Operation with the original abutment entrance design was very unsatisfactory. The square entrances caused the sheet of water to spring free from the spillway sides, resulting in a much reduced flow at the side walls and a concentrated flow in the center of the stilling pool. This unbalanced condition caused a severe whirl on each side of the pool. These whirls caused disruption of the hydraulic jump formation. The introduction of a curved entrance at each abutment (figure 8) and the streamlining of the upstream nose of the bridge piers eliminated this undesirable condition. As the studies progressed, a position of the stilling floor was developed which produced

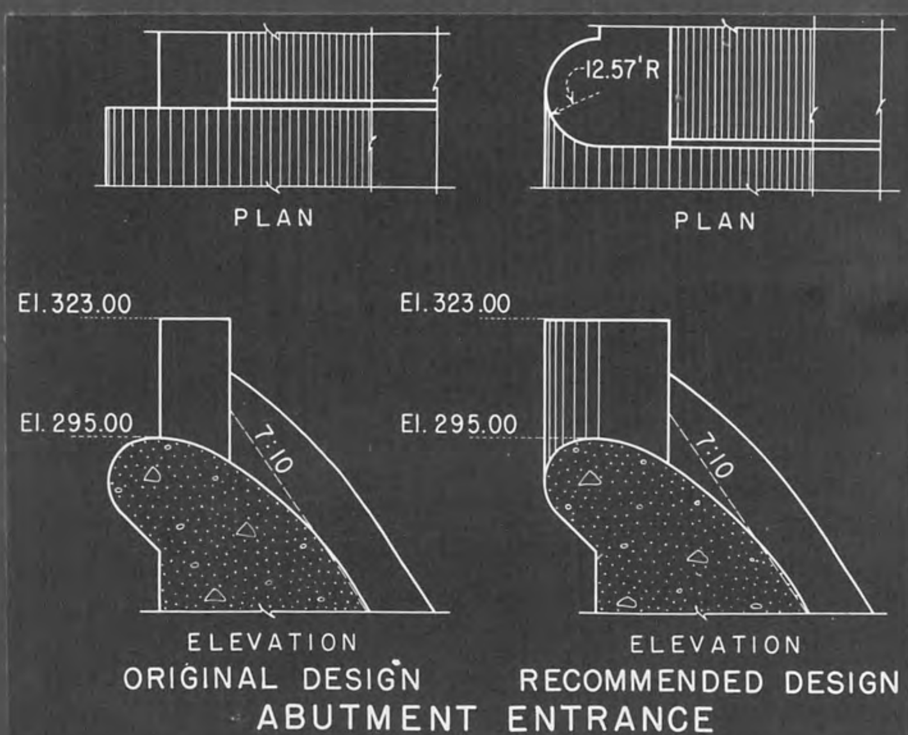

Figure 8 - Change of Entrance Condition on Dos Bocas Dam.

entirely satisfactory flow conditions for the maximum design discharge of 200,000 second-feet and for all other discharges except in the region of 50,000 second-feet. A severe whirl on the left side of the apron was formed by two distinct conditions acting together, the elimination of either or both being difficult. The whirl was considered troublesome inasmuch as any loose rock or gravel immediately downstream from the apron would be transported onto the apron to erode the concrete surface.

The first cause of this whirl was that even with the change of design on the bridge piers and abutment entrance there was still some lack of uniform distribution of flow over the extreme ends of the spillway. Secondly, at flows of less than 75,000 second-feet, the topography downstream on the left side acted as a control forcing the stream over into the main channel. As a result, a "dead" area was formed from which water flowed back into the jump area, disrupting the jump and forming the whirl. Sills, dentates and variations of the apron elevation did little toward elimination of the whirl. The final solution (figure 9 and figure 10) consisted of shifting the entire spillway crest fifteen feet to the right and converging the spillway and stilling pool wall toward the downstream end of the apron. The step by step analysis of this change is as follows: By rotating the left wall about its intersection with the crest line so that the downstream end

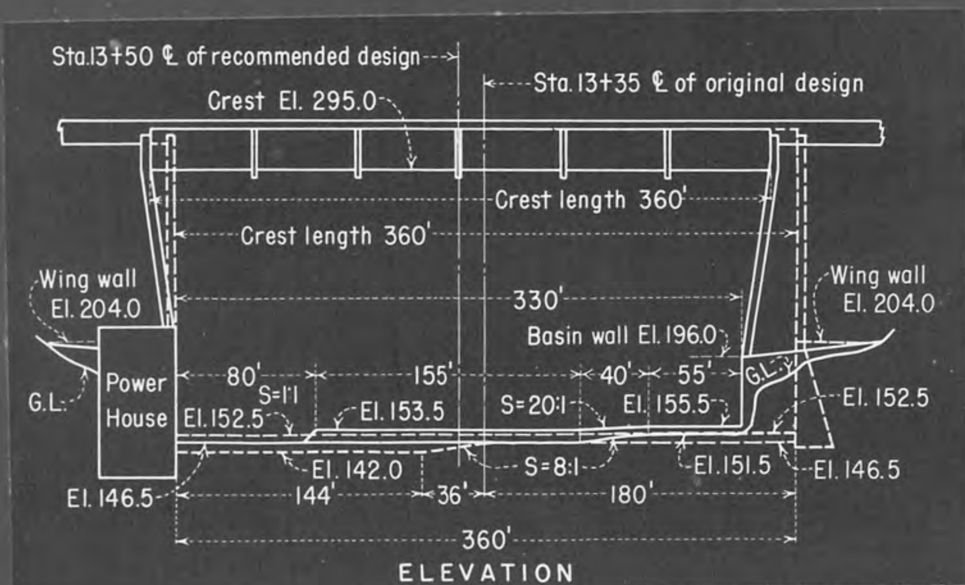

Figure 9 - Downstream Elevation of Dos Bocas Spillway Showing Original and Recommended Designs.

was fifteen feet to the right of the original position, it was found that the deficiency of water was eliminated and the stream so directed toward the original channel that the "dead" area in

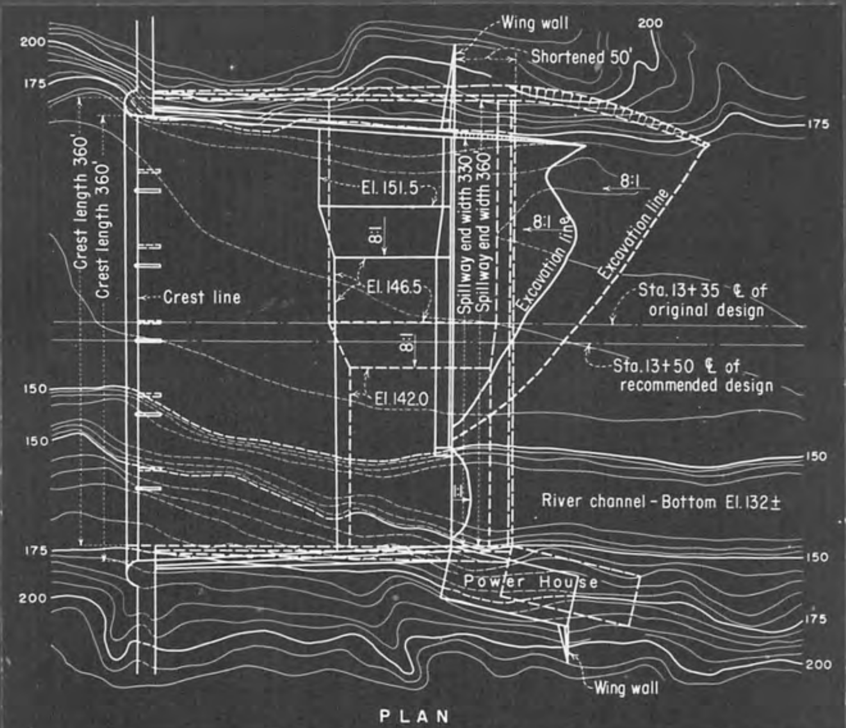

Figure 10 - Plan of Dos Bocas Spillway Showing Original and Recommended Designs.

which the whirl formed was eliminated. To establish symmetry of design again the right wall was rotated a similar amount to the left. No adverse effect was noted in the flow conditions; in fact, there was some improvement despite the fact that the downstream end of the apron had been constricted a total of thirty feet. However, the rotation of the right wall moved it fifteen feet out into the river channel. To bring it back to its original position, which was more desirable from a structural standpoint, the entire crest was shifted to the right a distance of fifteen feet, which in effect moved the downstream end of the left training wall thirty feet to the right. This change of position not only appreciably improved the hydraulic conditions in the stilling pool but produced a decided economy by reducing the deep rock excavation on the left side. The depth of excavation at the downstream end of the left training wall was reduced from 45 to 20 feet.

MASONRY DAM OUTLET WORKS

An important problem in recent developments is the provision of discharge capacity to release water in excess of the power demand. At Grand Coulee Dam (figure 11) three tiers of twenty 8-foot 6-inch outlets are being provided to release 225,000 second-feet, and at Shasta Dam, a total of seventeen outlets of the same size will be provided in three tiers to release a flow of 63,000 second-feet. The two installations have been studied practically as one problem so that the tentative design for Shasta Dam is identical to that of the upper tier at Grand Coulee Dam. Certain last minute changes could not be included at Grand Coulee because of the construction progress.

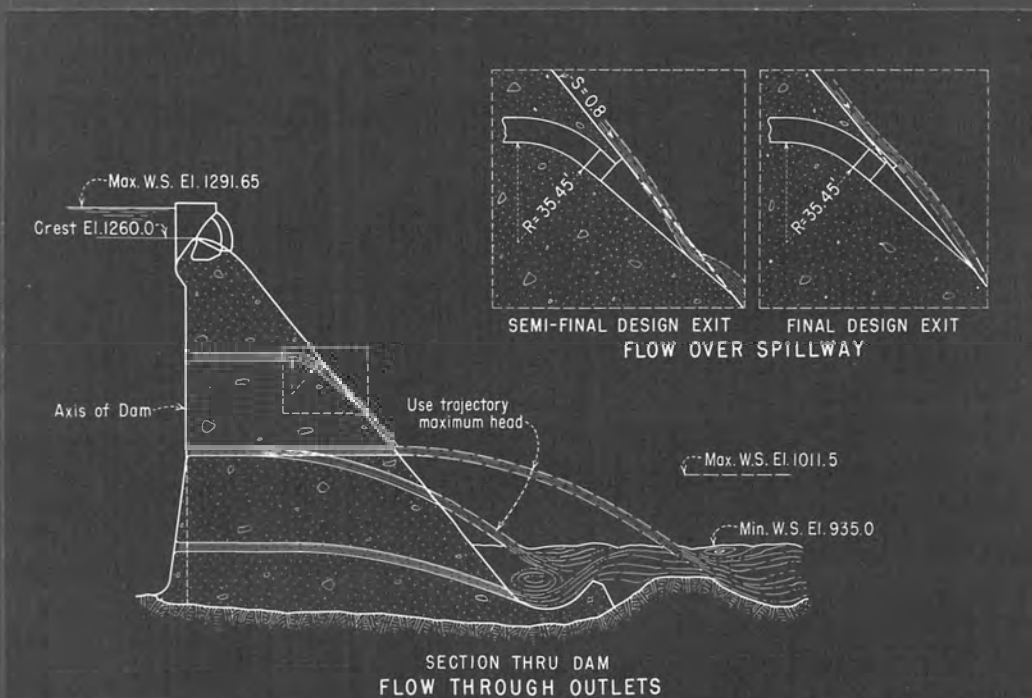

Figure 11 - Design of River Outlets for Grand Coulee Dam.

In the original design of these conduits, they were placed horizontally so that the jet would plunge into the tailwater downstream from the stilling pool. The model showed extreme scour conditions, particularly along the riprapped banks of the powerhouse tailraces. In the next design, the conduits were placed on a parabola through the dam so that the jet would plunge into the spillway stilling pool. The dissipation of energy in the flow from the upper tier was satisfactory, but with a high tailwater the jet from the middle tier was diverted over the bucket lip and into the erodible river bed. However, an error in assumptions was made which was brought to light in a subsequent model. If the control is at the downstream end of the conduit, the pressure gradient is above the conduit and the pressures in the conduit are all positive. Actually, the control was at the upstream end and the pressure gradient between the inlet and outlet dropped below the conduit a maximum of 46 feet or the difference in elevation between the inlet and outlet. Obviously, that condition is an impossibility in the prototype since absolute pressure is approximately 33 feet below atmospheric pressure. To prevent cavitation this negative head should not exceed the head at which cavitation occurs.

With these limitations, a new design of outlet was evolved in which an elbow was placed as near the face of the dam as structurally feasible and the water was discharged down the face of the dam into the spillway stilling pool bucket. Structural limitations had dictated a maximum head of 250 feet on the outlet control gates. A minimum head of 50 feet was assumed. A cone at the end of the elbow reduced the diameter from 8 feet 6 inches to 7 feet 9 inches. This reduction of diameter made the exit the control for all heads in excess of 50 feet and completely eliminated the negative pressures. The stilling pool bucket with the outlets discharging will function as an energy dissipator in a manner similar to that during the spillway discharges.

Later studies of the outlets installed in a model of the spillway showed a rather severe splash and spray condition where the sheet of water impinged in the opening of the outlet. This condition existed at flows of less than 500,000 second-feet distributed uniformly over the entire crest at Grand Coulee. A deflector was detailed to divert this flow over the opening. This additional structure permitted moving the conduit elbow three feet downstream thereby shortening the opening on the face of the dam a distance of 13 feet.

LOW HEAD DIVERSION DAMS

An interesting and rather startling incident, impossible to observe on the prototype because of the turbid condition of the flood water, was witnessed in a model (figure 12) of the Power Canal Diversion Dam on the Salt River Project in Arizona.

The dam, originally built in 1903 to divert water for power development in connection with the construction of Roosevelt Dam, was partly demolished by a flood in 1916. In 1935, plans

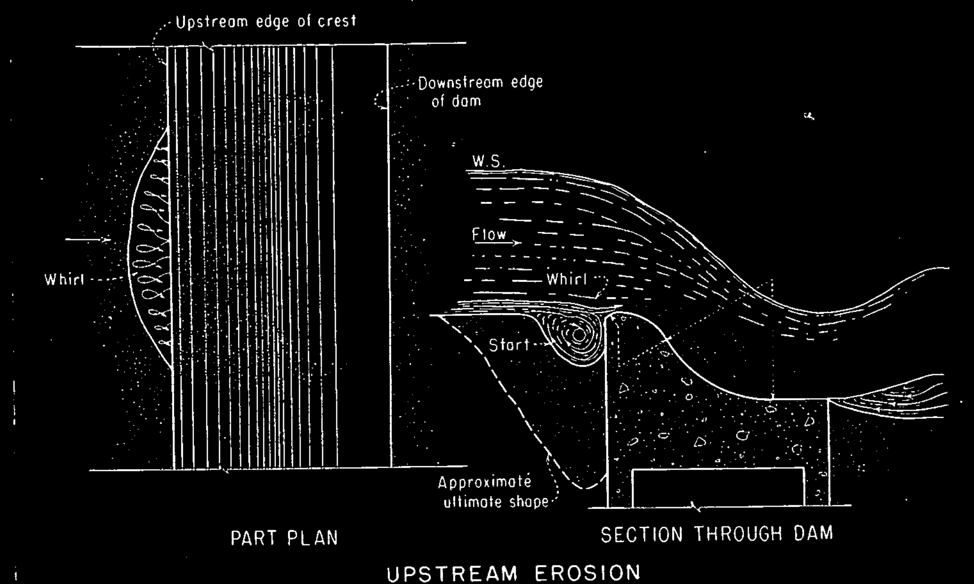

Figure 12 - Erosion Upstream From Crest of Salt River Power Canal Diversion Dam.

were formulated to rebuild the dam using the same general cross-section as before so that the portions of the old dam could be utilized.

The river above the dam carries a heavy bed load and during a flood bars form across the dam completely covering it for short intervals. In the clear water of the model, it could be seen that holes were scoured, often to a depth of 12 feet, along the upstream face of the spillway crest. The velocity of approach was high due to the shallow channel and as the water passed over the crest an eddy was formed below the upstream edge. This eddy picked up bed material near the upstream face of the dam and carried it downstream. The pocket increased in size until the intensity of the eddy was decreased and it could no longer pick up material. The hole was gradually filled again from the material being moved along by the stream, but while a particular hole was filling another would be forming elsewhere (figure 13). As a hole became filled, the cycle would be repeated. Examination of the portions of the original dam remaining in place disclosed scour to a depth sufficient to confirm the observations in the model.

Based on these facts, there is reason to believe that one of the major factors in the failure of the dam was piping under the dam due to the reduction of percolation length by the formation of the holes upstream. Only one of the original eleven sections of the spillway was moved any distance from its original position in the dam. Assuming that the major cause was piping, the one section was undermined and literally skidded downstream where it came to rest tilted upstream.

Figure 13 - Holes Scoured Upstream From Crest of Salt River Power Canal Diversion Dam.

In the redesign, the river bed was heavily riprapped upstream from the dam to stabilize it against a recurrence of the failure. Within a few days after the completion of the reconstruction in 1937, a flood, equal in magnitude to the one which had caused the previous failure, passed the dam. Subsequent examination of the riprapping showed no disturbance had occurred.

The section of the original dam (figure 14) moved downstream during the failure, was uncovered by the 1937 flood. Previously, it had been reported as "lost." Soundings below the reconstructed dam after this flood showed deep scour between the dam and the "lost section." Model studies of this combination checked the field measurements and showed that continued flood flow would move the "lost" section not downstream, but upstream. Undermining at the upstream side would disturb its equilibrium and cause it to roll toward the dam. In other words, an object too heavy to be moved downstream might roll upstream by undermining. To prevent such an incident at the prototype and its consequent endangerment of the reconstructed dam, the old section was removed by blasting.

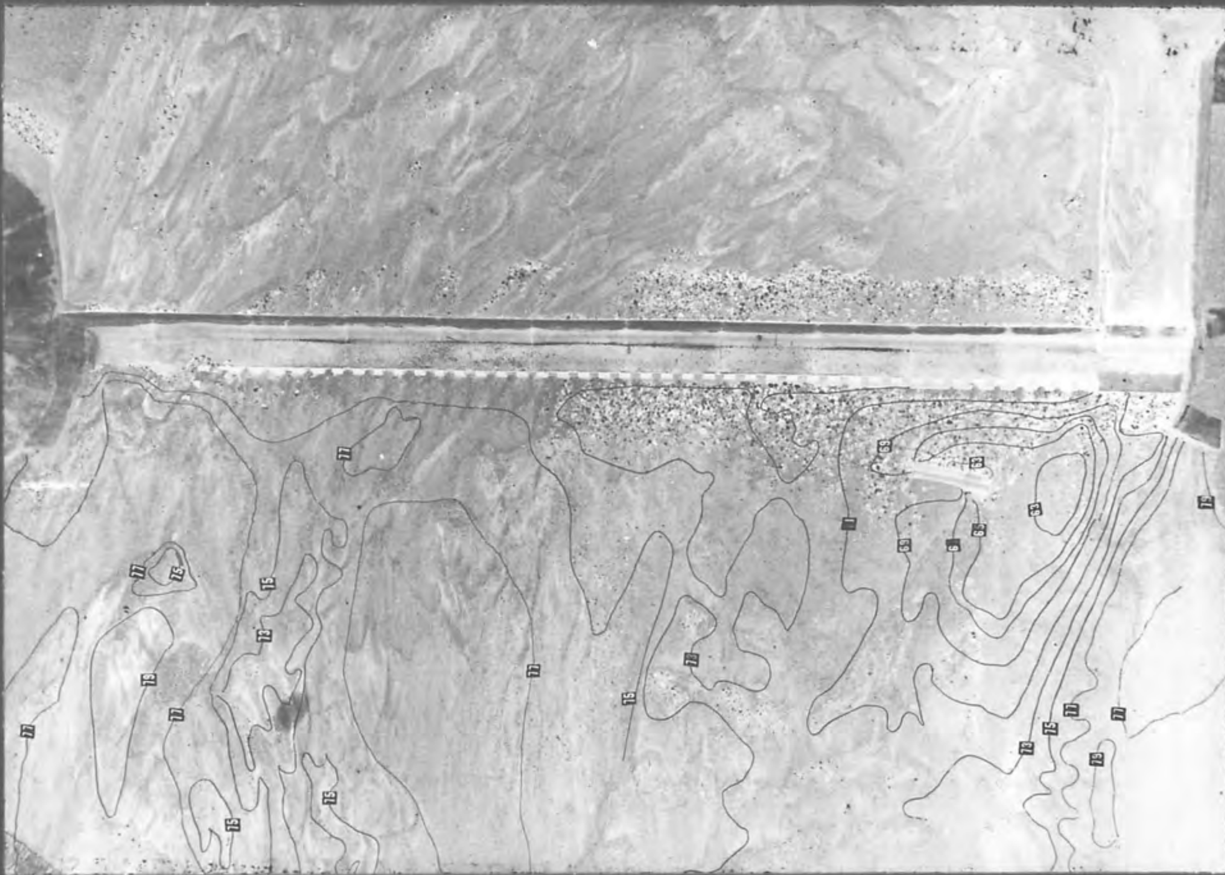

Figure 14 - "Lost" Section of Original Salt River Power Canal Diversion Dam.

GRAND COULEE SPILLWAY CREST

The design of a spillway crest where extreme depths of flow are involved offers another problem to the designing engineer. A deficiency of section under a sheet of overflowing water will result in the formation of a negative pressure between the downstream face of the dam and the nappe of water. With this condition three undesirable conditions will obtain:

(1) The overturning force on the spillway section of the dam will be increased due to the reduction of back pressure;

(2) The instability of the negative pressure with its intermittent pressure change will cause a vibration accompanied by a localized disintegration of the boundary which is known as cavitation;

(3) The intermittency of the negative pressure caused by the unstable condition prevailing beneath the flow sheet may cause a

vibration in the dam. While the amplitude of this vibration is exceedingly small, the accumulation of forces within the dam can produce secondary stresses, particularly if the natural frequency of the structure bears a certain relation to that of the frequency of the oscillation of the nappe. This event may give rise to a movement resembling an earthquake in the proximity of the structure.

The study of the spillway crest of Grand Coulee Dam will serve as an illustration of the approach to the solution of this particular phase of spillway design. The 28- by 135-foot drum gates required an unusually heavy crest in vertical section. The problem was to design a crest shape which would coincide with the trajectory of a freely falling nappe. Bazin's experiments for a 2:3 approach slope were used in the preliminary design. A model of the upstream portion of the crest was constructed on a 1:30 scale with a sharp-crested weir at elevation 1256.18. The profile of the lower nappe was measured using a coordinometer. An excellent agreement was found between the measured nappe and the preliminary design. The measured trajectory was then duplicated insofar as feasible by a compound curve with three radii (figure 15). This crest was included in the model and pressure measurements made.

A region of negative pressure at all flow conditions occurred with this design. The indications were that the curvature of the crest was too sharp near the downstream edge of the drum gate. To eliminate this low pressure region, the radius of the large curve was lengthened (figure 16), the axis of the crest moved upstream, and a parabolic curve introduced to connect the simple curve to the 0.8 slope of the downstream face of the spillway. The tests on this section showed that the low pressure zone had been eliminated.

STILLING BASINS FOR SMALL STRUCTURES

So far, this discussion has dealt primarily with major structures, but the principles involved are applicable to the design of small structures. Two groups of small structures on which considerable study has been made are outlet works for earth-fill dams and drop and chute structures for canals.

EARTH DAM OUTLET WORKS

The problem of scour prevention below earth dam outlet works is being encountered with increasing frequency. Laboratory

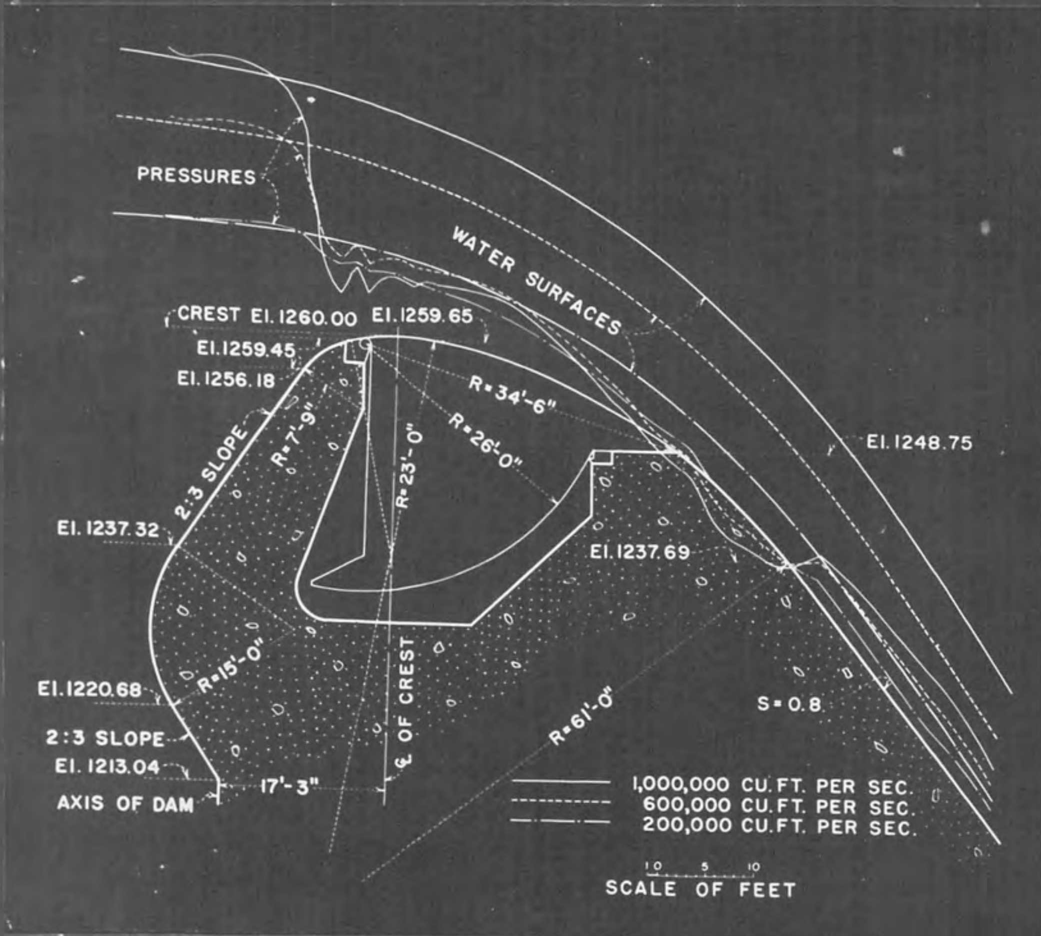

Figure 15 - Original Design of Grand Coulee Spillway Crest.

experiments have evolved four general types of stilling pools: (1) free jet, (2) chute, (3) hump, and (4) impact. The application of each is established by the relative position of the outlet and the tailwater; the type of outlet, whether slide gate or needle valve control; and the character of the downstream river channel.

By a series of experiments, the limits of application of each type have been fairly well established. Where the outlet is above the tailwater and the channel below is comparatively stable, a pool into which the jets will plunge is sufficient. In the case of the Tieton Dam, the pool was excavated by the action of the jets themselves. A natural grading carried away the smaller material and left the large gravel and boulders as riprapping. Recently,

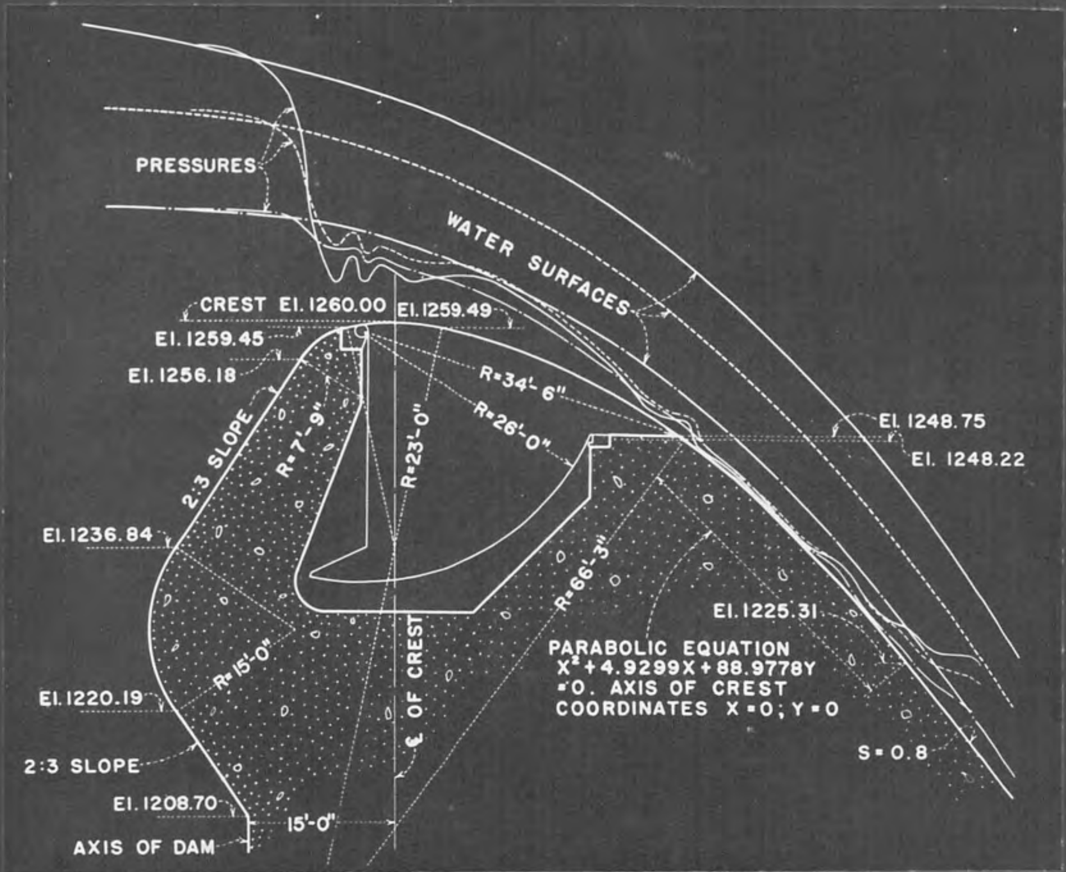

Figure 16 - Final Design of Grand Coulee Spillway Crest.

model studies of the outlet works for Grassy Lake and Deer Creek dams were made based on the results at Tieton Dam. A stilling pool was developed to be excavated and riprapped as part of the construction program. The basin was simply designed by determining the pattern of the eddies formed by the jets in the pool and determining the extent of riprap required.

If the channel is narrow and erodible, where scouring may be dangerous to the structure, a chute basin (figure 17) should be used for all conditions with the outlets above the water surface. The chute basin can also be used for all cases between outlet invert at tailwater elevation and outlet centerline at river bed elevation. If the outlet centerline is lower than the river bed elevation, the hydraulic jump will move back against the outlet

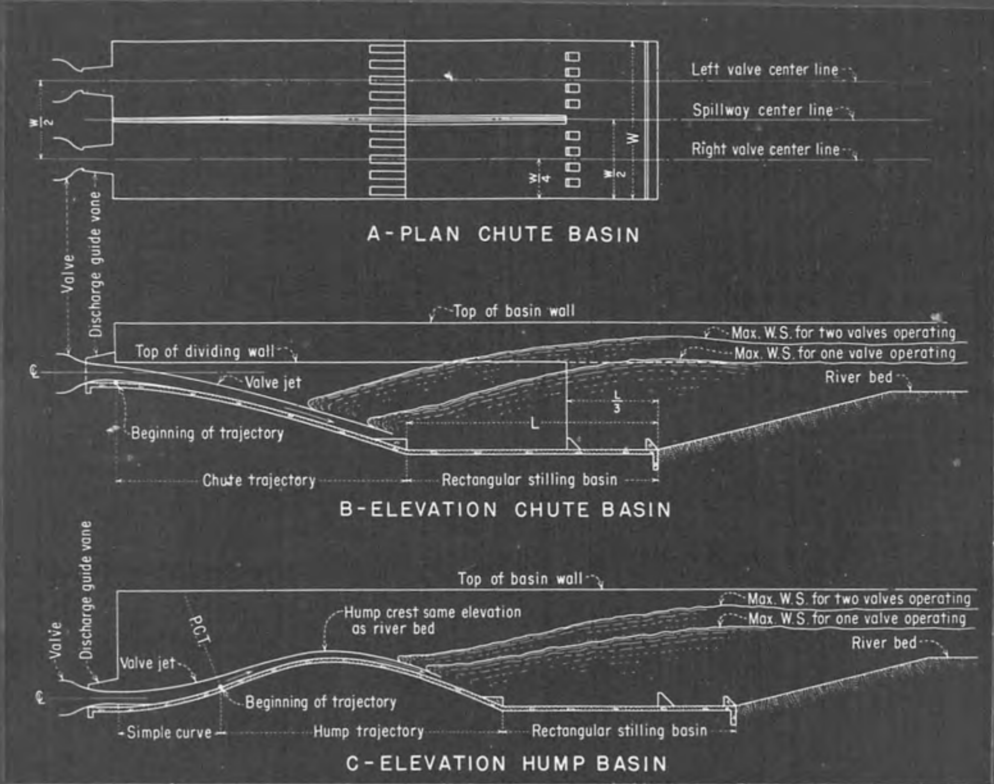

Figure 17 - Design of Outlet Works Stilling Pool.

causing undesirable flow conditions. The hump basin (figure 17) should then be used to form the jump downstream from the outlets. If the outlets are extremely low and the control is by slide gates, an impact pool can be used as was described in the first part of this paper.

The floor of a chute basin is designed to fit the maximum trajectory of the valve jet. The pool should be symmetrical with respect to the valve centerlines, with a width equal to twice the valve spacing. A dividing wall along the center line of the pool is required for satisfactory jump formation for one valve operation. The top of this wall should be at the maximum tailwater elevation for one valve operating and it may terminate at a point two-thirds down the basin. The hydraulic jump basin is designed in the same manner as in a spillway stilling pool. For single valve operation, each half of the basin functions as a unit, so the maximum tailwater for one valve operation should be used in determining the floor elevation.

The floor in a hump basin consists of a simple curve at the valve end and a trajectory designed to fit the maximum jet at the stilling basin end. The elevation of the hump crest should be about river bed level. In this case, a dividing wall is not essential to satisfactory flow for single valve operation, since the hump is effective in spreading the jet over the width of the basin. The dimensions may be determined as previously outlined, using the maximum tailwater for all valves operating to determine the floor elevation in this case.

MINOR SPILLWAYS, CANAL CHUTES AND DROPS

In irrigation systems many drop structures are required at changes in grade in a canal and at turnouts into the laterals. The total drop in water surface at these structures may be only a few feet or it may be several hundred feet, but in any event, it is necessary to provide some means at the lower level for dissipating a large part of the energy of flow, and for reducing the velocity before the flow proceeds along its natural course.

It has been only recently recognized that the hydraulic jump is one of the best means of obtaining effective energy dissipation and velocity reduction.

One type of structure which has been quite extensively used, principally because of its economy of construction, is the trapezoidal inclined drop (figure 18). The design from a hydraulic

Figure 18 - Poor Flow Conditions in a Trapezoidal Inclined Drop.

standpoint is fundamentally wrong. The high-velocity jet down the slope of the drop is concentrated in the center of the structure by the sloping sides. The jet prevails throughout the stilling basin structure with very little dissipation of energy. This is finally accomplished by channel friction and a semblance of a hydraulic jump, but the turbulent zone continues for a considerable distance downstream with sufficient force to scour the banks even at flows below capacity.

In the case illustrated, where approximately 75 percent of the maximum capacity of 114 second-feet is flowing with a drop of 7 feet, the original riprap was piled by the water into the center of the canal below the structure. Heavier riprap did not stay in place because the severe turbulence occurred downstream from the structure instead of on the concrete apron. The riprap was finally grouted to hold it. The extent of the maintenance can be seen in the illustration.

In contrast to the above case, a chute (figure 19) is shown with a total drop of 75 feet and with a flow of 100 percent

Figure 19 - Good Flow Conditions in a Chute with Rectangular Stilling Pool.

of its total capacity of 26 second-feet. The flow down the chute and through the hydraulic jump was well distributed with no return flow in the pool. Even though the water is turned 90 degrees a short distance downstream from the stilling pool, there was no erosion except a little "beaching" due to surface waves.

The length and depth necessary to form a hydraulic jump has been reduced in canal structures and minor spillways by the use of chute blocks, floor blocks, and end sills. In other words, impingement has been combined with the hydraulic jump (figure 20).

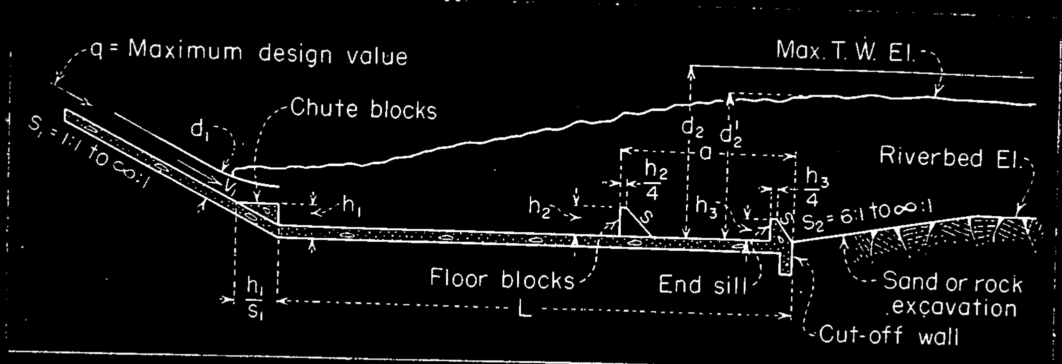

Figure 20 - Design of a Rectangular Stilling Basin.

With these devices the depth necessary to produce a hydraulic jump on a level floor is about 85 percent of the theoretical depth as determined by the momentum formula. This means that the basin floor may be raised 15 percent of the theoretical depth. Furthermore, the length of the basin may be reduced about 25 percent of that required for a level floor with no blocks or sill. The function of the chute blocks is to break the high velocity sheet of water entering the basin into a number of small jets, or in effect increase the depth at the entrance to the jump. The increased depth produces greater turbulence, hence greater energy dissipation. The floor blocks and the end sill aid in checking the high velocity and in maintaining the jump within the basin. The end sill is also effective in developing a ground roller at the end of the basin which not only prevents scouring at the cutoff wall but actually causes a deposition of material downstream from the sill.

CONCLUSION

It is not anticipated that the near future will produce many structures of the magnitude of Boulder, Grand Coulee, and Shasta dams, but from the applications which have just been discussed, it can be seen that the methods of design employed in the refinement of their hydraulic properties may be applied to small structures. This fact is being recognized by designing engineers, and as a result, structures formerly designed entirely by precedent are now being referred to the hydraulic laboratory.

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