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SPILLWAYS AND ENERGY DISSIPATORS

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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 (Fig. 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.

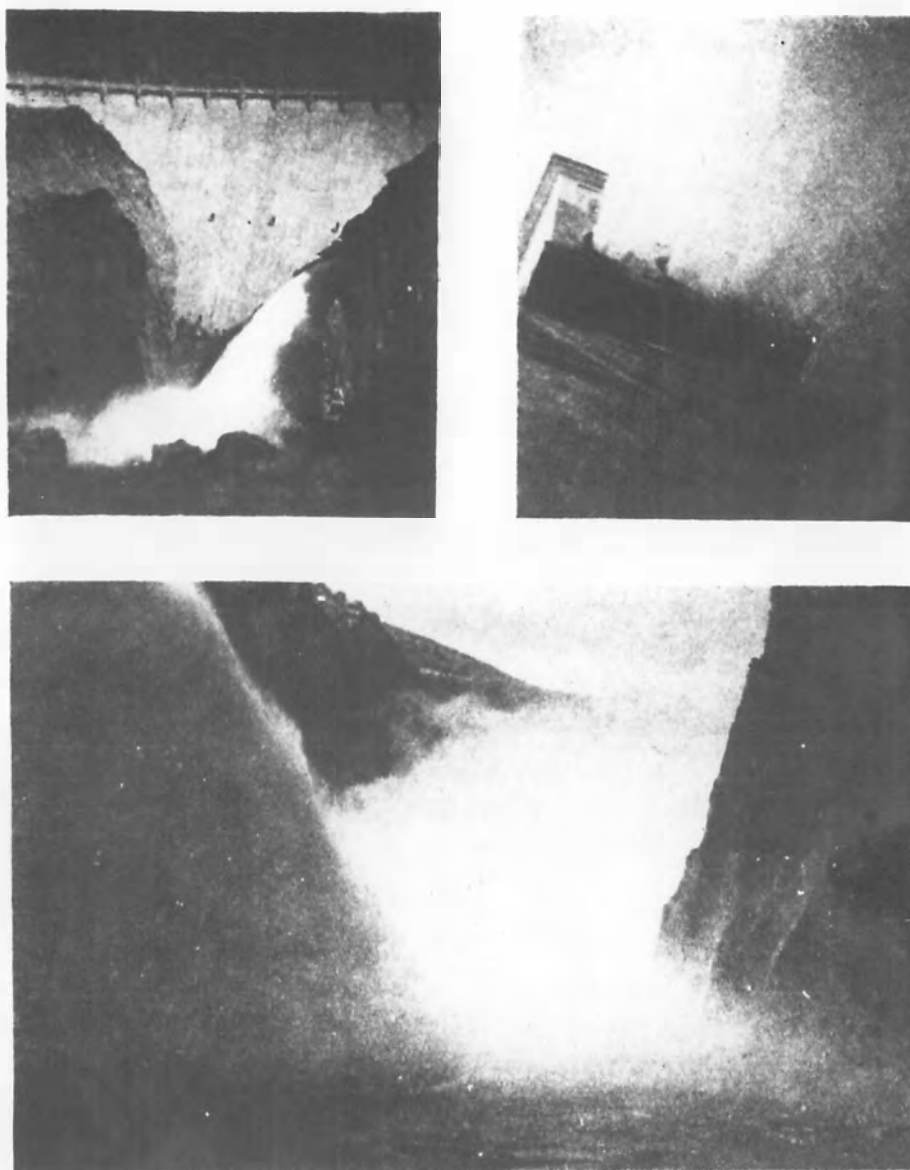


FIG. 1.—DISCHARGE FROM OWYHEE NEEDLE STRIKING THE TAILWATER SURFACE.

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 (Fig. 2) ended in impingement as a solution. Excessive tailwater caused by the bottom of the canal downstream from the outlets being located sev-

eral 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 con-

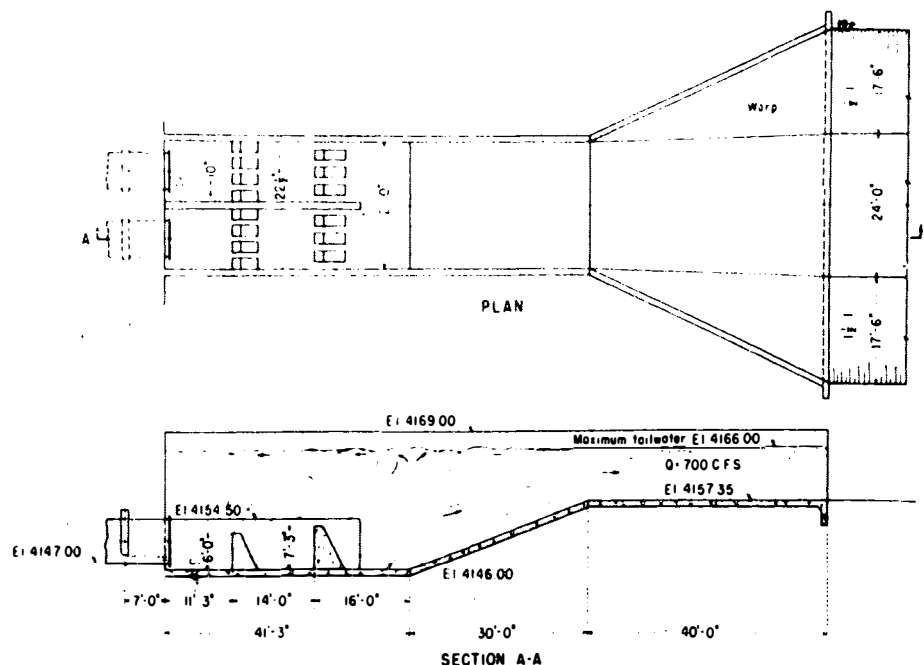


FIG. 2.—STILLING POOL FOR IRRIGATION-OUTLET WORKS AT CONCHAS DAM, TUCUMCARI PROJECT.

ditions the exit velocity will be 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 be unwatered readily 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. The details of the design of this structure were contained in an article by the author in the November 1936 issue of *Civil Engineering*.

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

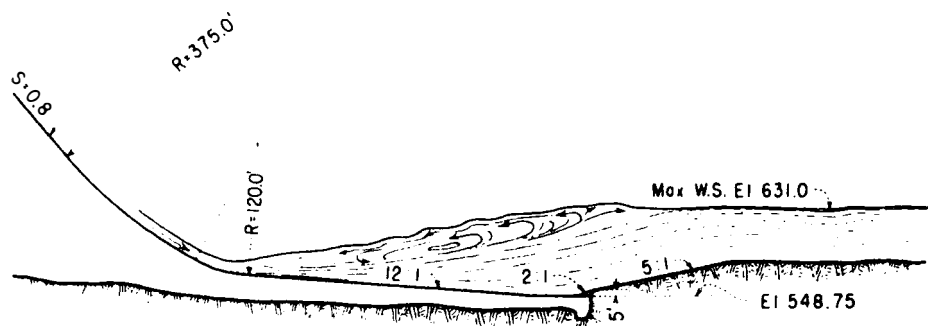


FIG. 3.—FINAL DESIGN OF STILLING POOL FOR SHASTA DAM.

are the Norris Dam, constructed by the Tennessee Valley Authority; the Madden Dam, built by the Panama Canal Zone; and the Shasta Dam (Fig. 3), now under construction by 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 and 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 alternative, 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.

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.

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 (Fig. 4) 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 hydraulic jump

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

the values of D_2 for different discharges were computed. With the pool entrance as origin, length of pool as abscissa, and elevation as

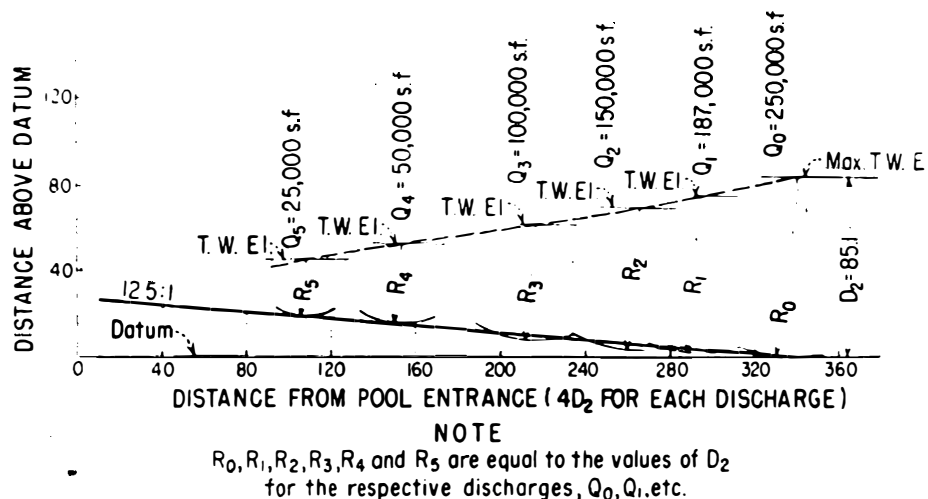


FIG. 4.—DESIGN OF SLOPING APRON FOR SHASTA DAM.

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 un-

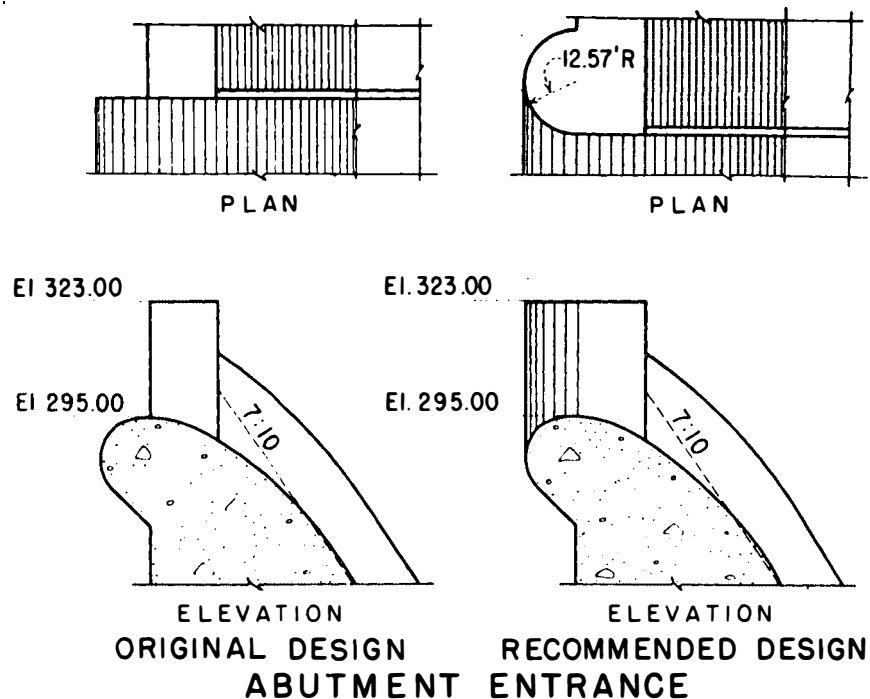
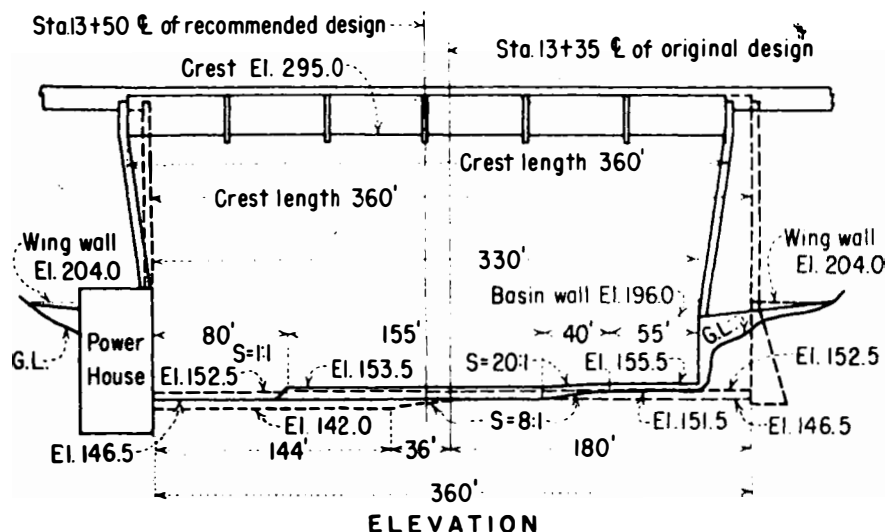


FIG. 5.—CHANGE OF ENTRANCE CONDITION ON DOS BOCAS DAM.

satisfactory. 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 (Fig. 5) 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 entirely satisfactory flow conditions for the maximum designed discharge of 200,000 second-feet and for all other discharges except in the region of 50,000



second-foot. 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 (Figs. 6 and 7) consisted of shifting the entire spillway crest fifteen feet to the right and converging the spillway and stilling-pool walls 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 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 which the whirl formed was eliminated. To again establish symmetry of design, the right wall was rotated a similar amount to the left. No adverse ef-

feet 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

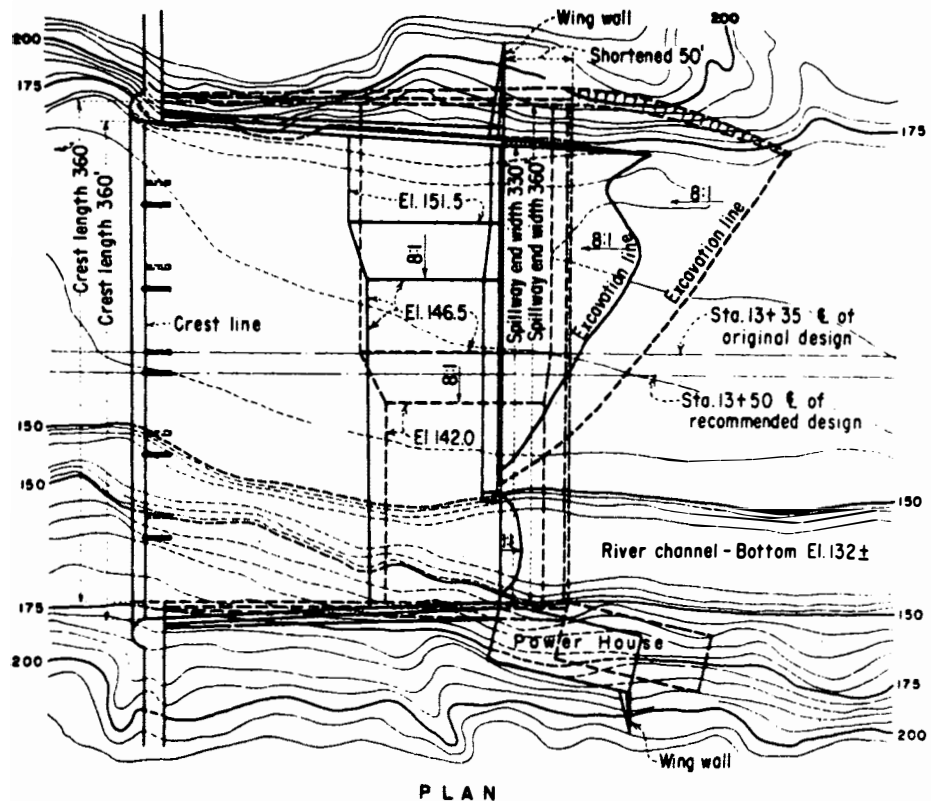


FIG. 7.—PLAN OF DOS BOCAS SPILLWAY SHOWING ORIGINAL AND RECOMMENDED DESIGNS.

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 (Fig. 8) 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. Certain last minute changes could not be included at Grand Coulee because of the construction progress.

In the original design of these conduits, they were placed hor-

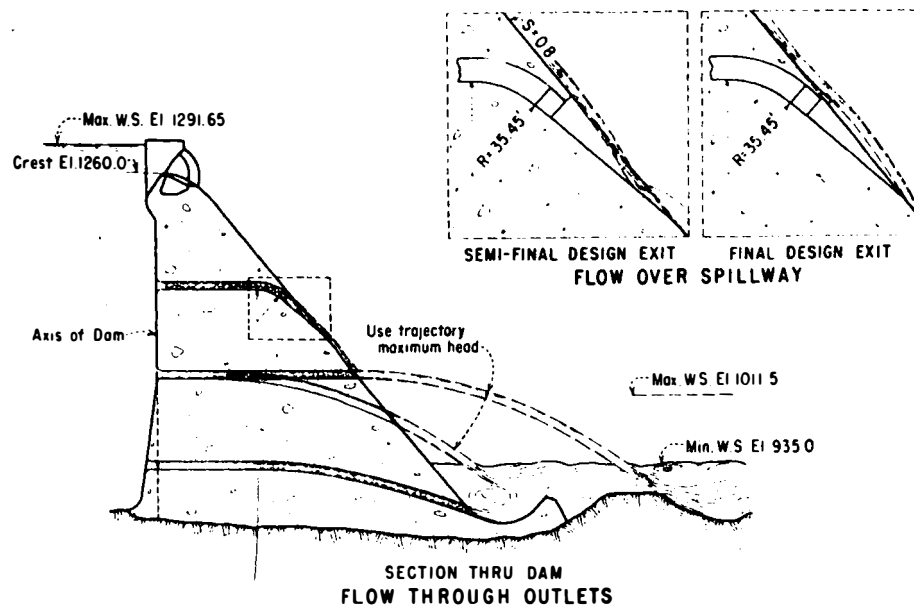


FIG. 8.—DESIGN OF RIVER OUTLETS FOR GRAND COULEE DAM.

izontally 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 assumption 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 out-

let 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 down the spillway 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.

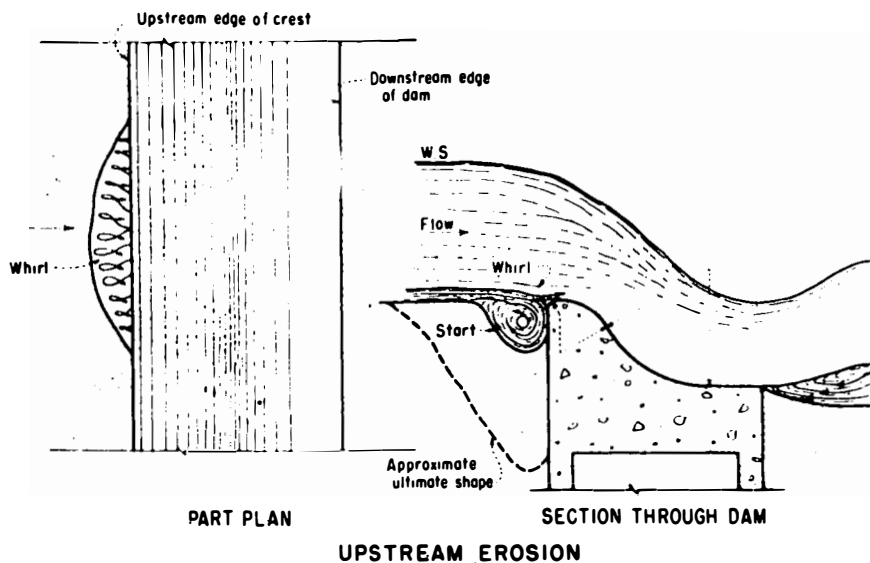
LOW-HEAD DIVERSION DAMS

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 (Fig. 9) 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 were formulated to rebuild the dam using the same 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 then gradually



UPSTREAM EROSION

FIG. 9.—EROSION UPSTREAM FROM CREST OF SALT RIVER POWER CANAL DIVERSION DAM.

filled again from the material being moved along by the stream, but while a particular hole was filling another would be forming elsewhere (Fig. 10). 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 of the failure was piping under the dam due to the reduction of percolation length by the formation of the holes upstream. Only one section 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.

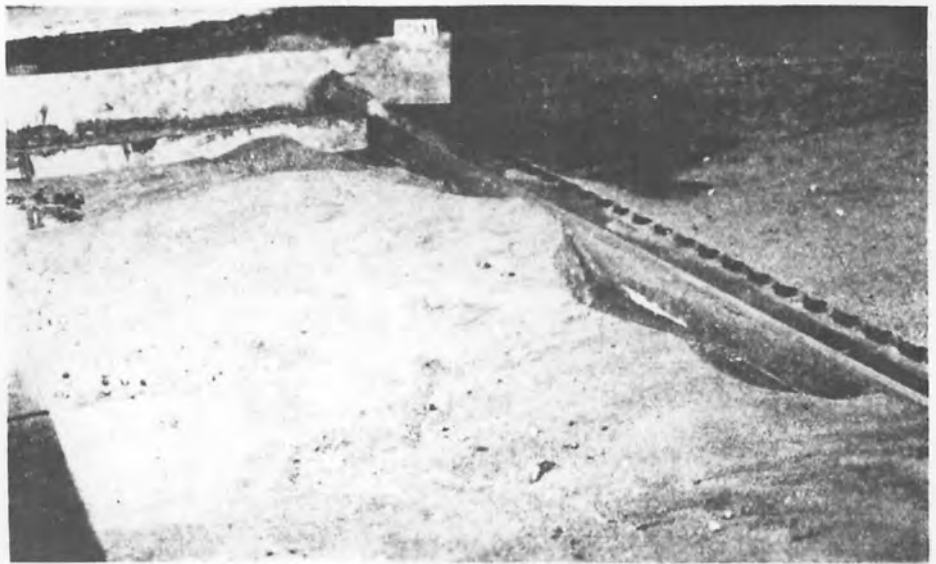


FIG. 10.—HOLES SCOUTED UPSTREAM FROM CREST OF SALT RIVER POWER CANAL DIVERSION DAM.

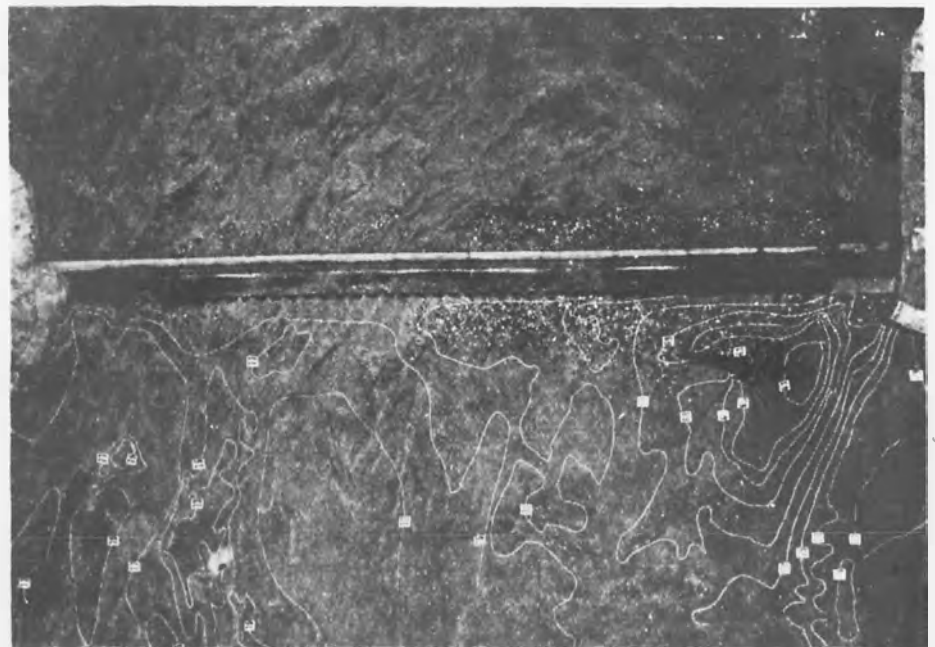


FIG. 11.—"LOST" SECTION OF ORIGINAL SALT RIVER POWER CANAL DIVERSION DAM.

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 or greater 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 (Fig. 11) 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.

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 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; by the type of outlet, whether slide-gate or needle-valve control; and by 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, 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

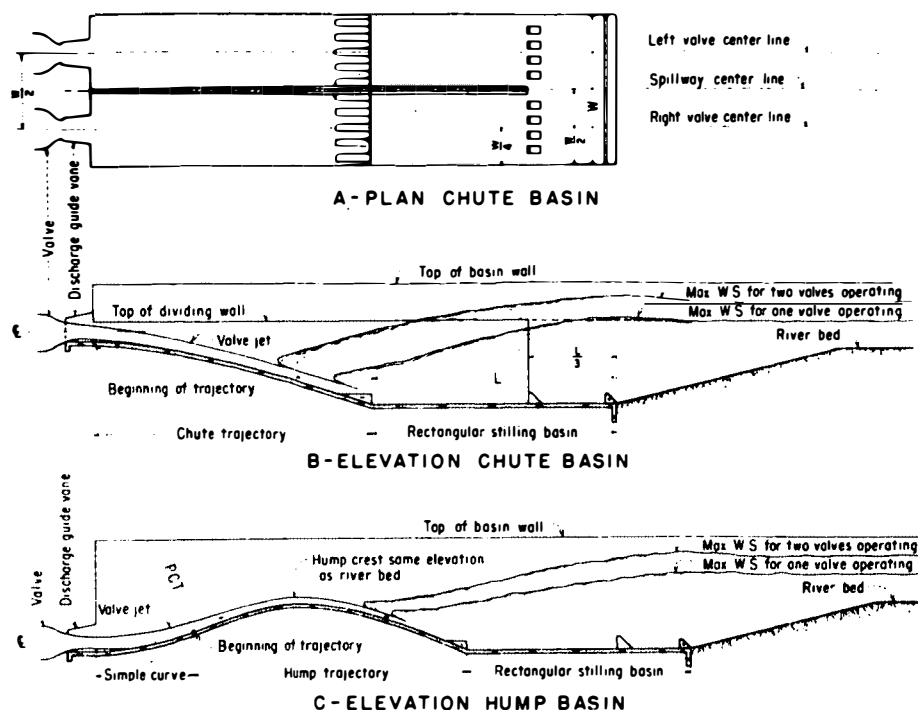


FIG. 12.—DESIGN OF OUTLET-WORKS STILLING POOL.

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 (Fig. 12) 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, causing undesirable flow conditions. The hump basin (Fig. 12) 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 centerline of the pool is required for

satisfactory jump formation with one valve operating. 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 operating 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 and acts to prevent return flow. The dimensions may be de-



FIG. 13.—POOR FLOW CONDITIONS IN A TRAPEZOIDAL INCLINED DROP.

termined 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 recognized only recently 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 (Fig. 13). The design from a hydraulic 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 through the stilling-basin structure with very little dissipation of energy. This is finally accomplished by channel friction and the 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 per cent of the maximum capacity of 114 second-feet is flowing with a drop of only 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

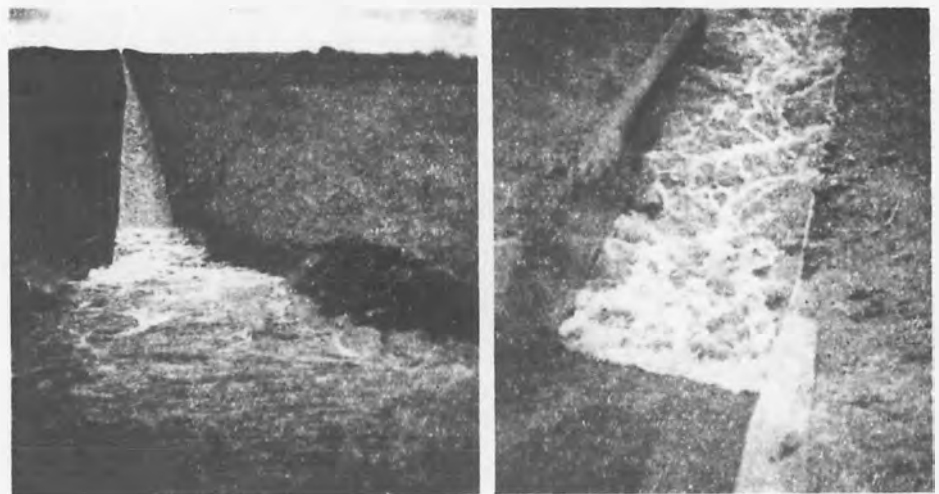


FIG. 14.—GOOD FLOW CONDITIONS IN A CHUTE WITH RECTANGULAR STILLING POOL.

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 (Fig. 14) is shown with a total drop of 75 feet and with a flow of 100 per cent 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 (Fig. 15). With these devices the depth necessary to produce a hydraulic jump on a level floor is about 85 per cent of the theoretical depth as determined by the momentum formula. This means that the basin floor may be raised 15 per cent of the theoretical depth. Furthermore, the length of the basin may be reduced about 25 per cent 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

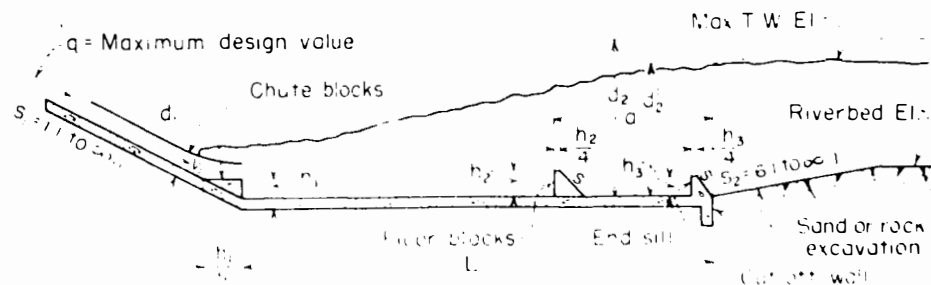


FIG. 15. DESIGN OF A RECTANGULAR STILLING BASIN.

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, Grande 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.