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CONTEMPORARY DESIGN OF MAJOR SPILLWAYS AND ENERGY DISSIPATORS (*)

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INTRODUCTION

The Bureau of Reclamation design of spillways and their associated energy dissipators are influenced by several major factors. These are: (1) hydraulic considerations, (2) type of dam, (3) topographic and geologic considerations, (4) access to the structures, and (5) climatic conditions. When one or more of these factors results in an atypical design, the Bureau validates the initial design by constructing and studying a hydraulic model of the spillway and energy dissipator. In some cases where special problems arise, still another model of larger scale that concentrates on a particular feature of the spillway will be studied. This may occur with inlets or intakes to the spillway or areas adjacent to control gates where experience has shown that negative pressures could lead to cavitation damage of flow surfaces or where the hydraulic action of the discharge initiates unacceptable vibrations in the structure.

(*) *Conception actuelle des grands évacuateurs de crue et dissipateurs d'énergie.*

This paper deals with three recent examples of Bureau of Reclamation spillways designs, the problems associated with each and how they were resolved. Each spillway is of a different type and each was influenced by several of the factors enumerated previously.

CRYSTAL DAM

Crystal Dam and Powerplant, completed in 1977, is the last unit of a three unit project that was authorized to develop a section of the Gunnison River located in western Colorado. The primary purposes for the dam and reservoir are to function as an afterbay for a larger power generation dam, Morrow Point, located approximately 10 km upstream, to supply irrigation water, and to generate electric power.

The dam is located in a narrow, V-shaped canyon in the Colorado highlands. The foundation rock consists predominantly of a fine grained, highly siliceous rock which contains minor but variable amounts of micaeous minerals [4]. Geologically, the rock has been classified as a competent metaquartizite with significant areas of mica schist, and a minor amount of metadiorite. It is notably homogeneous with the exception of widely spaced intrusions of pegmatite bodies. All of these rock types are hard, strong and free of significant shears. However, a very definite joint pattern exists in the rock fabric and strikes about N 70° W and dips downstream. A subordinate and less conspicuous joint set strikes roughly normal to the predominant set. Overburden consists of talus and slope wash along the abutments and stream-deposited fill in the river channel. The overburden depths were approximately 25 m and 14 m on the lower left and right abutments, respectively, and 34 m in the river channel itself.

Crystal Dam was designed and constructed as a thin, double curvature concrete arch with a structural height of 107 m and a crest length of 189 m. A single-unit powerplant with a generating capacity of 24 000 kW is located at the toe of the dam. Waterways for the dam include the penstock serving the powerplant, river outlets consisting of two 1.37-m diameter steel conduits, and the spillway. Although the dam and powerplant are currently operated by Bureau of Reclamation personnel at the site, eventually the powerplant will be unmanned and supervised remotely from the town of Montrose, Colorado, about 32 km distant.

Several types of spillways were considered that ranged conceptually from a tunnel structure to a chute type constructed integrally with the dam and powerplant roof. The influencing parameters for spillway and energy dissipator selection were the narrowness of the canyon, difficulty of access, and the type of dam. Since Crystal Dam and Powerplant is to be unmanned in the future, and because access is extremely difficult in winter months due to snow and ice, it was decided that an uncontrolled spillway crest free of control gates

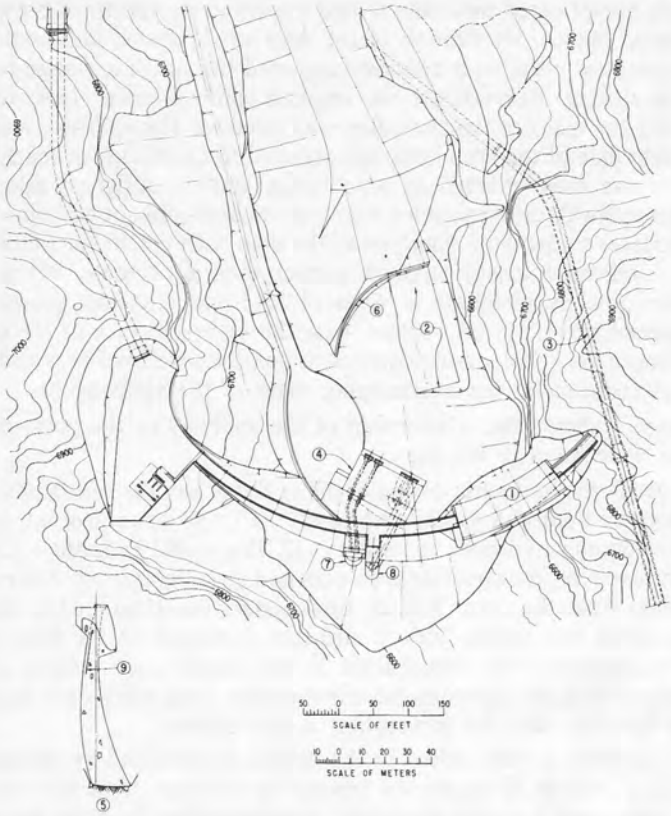

Fig. 1

General plan of dam, powerplant, and spillway.

(1) Spillway.
(2) Plunge pool stilling basin.
(3) Diversion tunnel.
(4) Powerplant.
(5) Section through dam and spillway.
(6) Plunge pool deflector wall.
(7) Intake for river outlets.
(8) Intake for penstock.
(9) Posttensioned groutable rock bolts.

Plan général du barrage, de la centrale électrique et de l'évacuateur de crue.

(1) Évacuateur de crue.
(2) Bassin d'amortissement.
(3) Galerie de dérivation.
(4) Centrale électrique.
(5) Coupe du barrage et de l'évacuateur de crue.
(6) Mur-déflecteur du bassin d'amortissement.
(7) Prise d'eau pour vidange ou lâchures en rivière.
(8) Prise d'eau pour conduite forcée.
(9) Barres d'ancrages à coulis post-tendues.

with their complicated mechanical and electrical equipment represented the most logical choice. Excavation of the deep sand, gravel, and boulder overburden complex in the river channel suggested the use of a plunge pool as an energy dissipator. Accordingly, an ungated spillway crest constructed integrally with the mass of the arch dam was selected. The spillway was located on the right side of the arch dam and oriented to discharge as nearly into the middle of the river channel as practicable. Bureau designers attempted to locate the spillway crest in such a way that its mass did not add objectionable tensile stresses to the dam. Analysis of the dam with the added spillway crest, however, indicated tensile stresses greater than acceptable. These stresses were overcome by installing a series of 50.8-mm diameter groutable rock bolts, spaced at 1.37 m on centers, near the upstream face of the dam. The posttensioned rock bolts added sufficient compressive load to overcome tensile loads induced by the overhanging mass of the spillway lip.

Figure 1 shows the relationship of the spillway to the powerplant and the other waterways in the dam.

To verify the hydraulic design of the spillway and the action of its plunge pool energy dissipator, a model with a scale of 1 : 36 was constructed and tested in the Bureau's hydraulic laboratory [1]. The model included 122 m of the upstream reservoir, the concrete arch dam and powerplant, and 244 m of river downstream from the dam. Twenty piezometers were installed in the model spillway along two radial lines to measure pressures of the flow surfaces. Canyon topography was constructed in the model from 7.62-m contours. Pipes representing the penstock and river outlets were connected between the reservoir headbox and the powerplant in the tailbox.

The spillway profile, tested in the model, is described by the equation : $X^2 = -24\,Y$, where X equals the horizontal distance from the axis of the spillway crest, and Y equals the vertical distance below the crest. As shown in Figure 1, the spillway profile terminates in a flip or skijump horizontal exit bucket which trajects the discharge downstream and away from the toe of the dam. A radius of 4.57 m defines the exit bucket. The spillway profile was designed for a partial vacuum crest at the design head of 4.88 m. Pressures were measured along the 10 piezometers located along each of two radial lines. One line was near the center of the spillway, the other was 8.23 m from the left training wall. Pressure measurements along the two lines were essentially equal and were observed for several discharges. A minimum pressure of minus 0.79 m was recorded at piezometers located 1.83 m downstream from the crest at the design discharge of 1 171 m^3/s. The initial spillway location produced a very poor flow condition for discharges above 850 m^3/s due to protrusion of the canyon wall. To improve the reservoir approach, the spillway was moved 9.14 m toward the center of the dam and away from the right abutment.

Several pier designs were tested in an attempt to reduce the abrupt water surface drawdown observed around the initial pier concept. The preliminary concept turned the flow too abruptly and caused a very pronounced draw-

down, resulting in a very rough flow along the training walls as the flow accelerated along the spillway. The elliptical pier configuration finally selected was developed based on work by Rouvé [5].

The preliminary spillway profile with the horizontal exit from the bucket lip, did not traject the spillway discharge an adequate distance from the dam to ensure protection against rock erosion and undercutting of the right abutment and base of the dam. To traject the discharge farther into the pool, the radius of the bucket was extended 137.16 mm above the bucket invert, where the bucket lip terminated at a 4 to 1 tangent. As a result of the modified bucket design and improved reservoir approach, the discharge impinged well within the excavated plunge pool.

The initial plunge pool design conceptualized a basin excavated to rock or to an elevation of 1 970 m, whichever was deeper and approximately rectangular in configuration with the longer dimension in the upstream-downstream direction. This excavation insured a plunge pool with a minimum depth of 18 m increasing to 30.5 m at maximum spillway discharge. The downstream end of the basin was sloped upwards on a 3 : 1 horizontal to vertical grade until the invert of the existing river channel was reached. Initial tests indicated violent surface boiling near the powerplant access road. Excavation into the right abutment originally thought necessary was found to be unnecessary but more excavation was needed on the left side of the channel to avoid the high surface turbulence. To test the action of the submerged discharge jet on the large riprap that was to protect the 3 : 1 slope at the downstream end of the basin, rock fragments simulating 0.91 m riprap were installed in the model.

The model was tested for 1 hour at a spillway discharge of 1 161 m^3/s (design discharge is 1 171 m^3/s). A considerable amount of the large riprap was carried up the slope and deposited along the left bank. To deflect the submerged discharge up and away from the riprap, a 4.57-m high deflector wall with a 1 : 4, horizontal to vertical, batter was placed on the floor of the plunge pool model. Rock representing riprap with an average diameter of 0.58 m, which would be more common in the vicinity of the dam, was placed behind the wall and on the 3 to 1 slope. The model was tested again for 1 hour at a spillway discharge of 1 161 m^3/s. This time the riprap was relatively undisturbed.

To measure the impact pressures on the floor of the plunge pool, a grid of 16 piezometers was placed in an area 14.5 m wide by 43.9 m long. Maximum pressures (water manometer) were recorded for several discharges and corresponding tailwaters. The maximum pressure for a spillway discharge of 1 174 m^3/s was 48.5 m of head. Later in the testing, six additional piezometers were located on the spillway centerline, 84.7 m from the axis of the dam. These piezometers were equipped with pressure cells immediately below the floor of the plunge pool. Dynamic pressures were recorded for several discharges. The highest average value observed on the six pressure cells for a discharge of 1 161 m^3/s represented a total pressure head of 44.8 m. The maxi-

mum instantaneous dynamic pressure represented an elevation of 2 062.2 m, the reservoir elevation near a 1 161-m^3/s discharge. This would indicate that at times the instantaneous energy level on the rock invert of the prototype plunge pool basin reaches the potential energy level of the reservoir.

As a result of the model testing, the following modifications were made to the original designs and incorporated in the specifications : (1) the spillway was shifted 9.14 m towards the center of the dam; (2) the exit bucket lip was sloped upwards above the horizontal to traject the spillway discharge farther downstream; (3) an improvement was made to the configuration of the spillway piers which minimize drawdown effects; and (4) the stilling action of the plunge pool energy dissipator was improved by the addition of a concrete deflector wall along the downstream side of the basin. In addition, the model validated that the spillway was capable of discharging 1 171 m^3/s (discharge resulting from routing of maximum probable flood) at the design head of 4.88 m. To date, the prototype spillway has not operated.

AUBURN DAM

Auburn Dam and its waterways have been discussed in several technical publications including a paper prepared for the International Commission of Large Dams Congress held 1973 in Madrid, Spain [3]. At that time, Auburn Dam was conceptualized as a thin double-curvature concrete arch structure. On August 1, 1975, a seismic event, Richter magnitude 5.7, occurred near Oroville Dam located adjacent to the Foothills Fault System some 105 km from the Auburn damsite. This event required reevaluation of the seismicity of the Auburn area since the damsite is near the same fault system as Oroville Dam. As part of the Bureau of Reclamation's reevaluation process, other dam alternatives in addition to the arch type are currently being studied.

The spillways described below and modeled extensively in the Bureau's hydraulic laboratories represent the design which will be adopted if the dam type selected is either the arch structure or a curved concrete gravity structure at the existing site.

Twenty orifice openings located 45.72 m below its crest, each controlled by 3.15- by 5.18-m fixed wheel gates, comprise the spillways. The orifice openings, arranged 10 on each side of the center of the dam, are separated by piers which terminate at the downstream face of the dam, and discharge along an open chute. The 10 orifices on the left side of the dam are identified as the service spillway; the remaining 10 orifices on the right side of the dam make up the auxiliary spillway. The spillways are identical structures, except for a splitter wall located in the service spillway, until the inclined chutes are reached, refer to Figure 2. The orifices making up the auxiliary spillway discharge their flows into an inclined chute that conveys the flows into a superelevated, near horizontal reach terminating in an exit bucket with a serrated

sill. The bucket trajects the flows into the river channel downstream where the energy is dissipated in the river.

The service spillway flows, once beyond the chute constructed integrally with the mass of the dam, discharge into a split level inclined chute. Each level of the split chute terminates in an exit bucket that trajects flows along

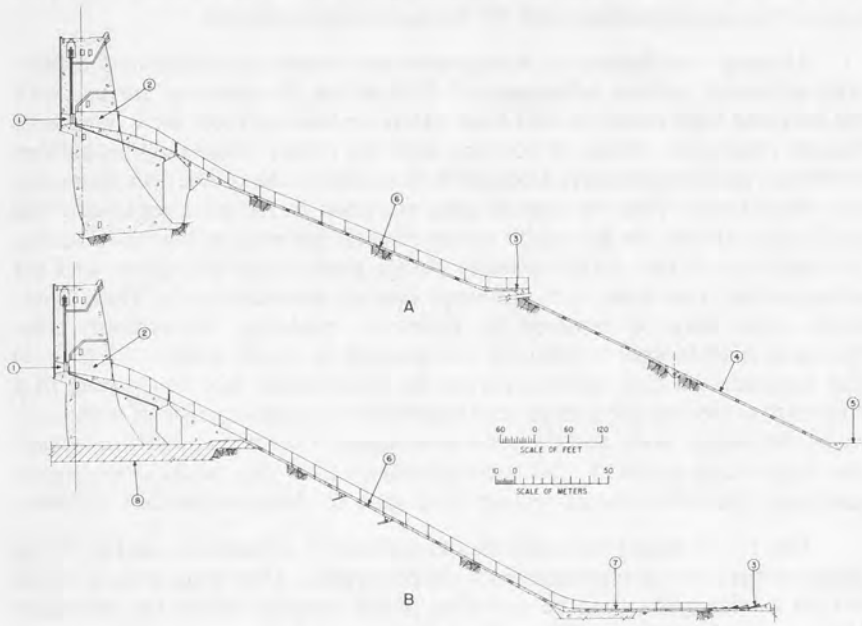

Fig. 2

(A) Elevation through dam and service spillway.

(1) Orifice intake section.
(2) Orifice outlet section.
(3) Exit bucket structure.
(4) Reinforced concrete apron.
(5) Plunge pool stilling basin.
(6) Chute structure, inclined

(B) Elevation through dam and auxiliary spillway.

(1) Orifice intake section.
(2) Orifice outlet section.
(3) Exit bucket structure.
(6) Chute structure, inclined.
(7) Chute structure, horizontal.
(8) Foundation treatment concrete.

(A) Coupe longitudinale de l'évacuateur principal.

(1) Coupe transversale de l'orifice d'entrée.
(2) Coupe transversale de l'orifice de sortie.
(3) Sortie de l'auge de pied.
(4) Radier en béton armé.
(5) Bassin d'amortissement.
(6) Coursier incliné.

(B) Coupe longitudinale de l'évacuateur auxiliaire.

(1) Coupe transversale de l'orifice d'entrée.
(2) Coupe transversale de l'orifice de sortie.
(3) Cuillère de dissipation.
(6) Coursier incliné.
(7) Coursier horizontal.
(8) Béton de traitement de la fondation.

an inclined reinforced concrete apron and thence into the plunge pool. For low discharges, up to 1 698 m^3/s, the exit buckets traject their flows directly on the apron. It is only at the higher discharges from 1 698 m^3/s to 4 250 m^3/s that the plunge pool energy dissipator comes into play. The frequency of occurrence for the larger flow is approximately once in 100 years. Both spillways enter the river channel at oblique angles – 60° from the channel centerline for the service spillway and 25° for the auxiliary spillway.

Although the Bureau of Reclamation has some experience with orifice-type spillways, orifices submerged 45.72 m below the reservoir surface with the resulting high velocities and their effects on flow surfaces were outside of Bureau experience. Areas of concern were the orifice intakes configurations (to ensure positive pressures along all flow surfaces), the orifice area immediately downstream from the control gate, vibration in the piers separating the individual orifices, the hydraulic action of each spillway acting individually, the operation of the service spillway plunge pool energy dissipator, and the action in the river when both spillways operate simultaneously. These questions could only be resolved by hydraulic modeling. Accordingly, two hydraulic models were constructed and studied. A model, scale 1 : 72 (Fig. 3) that incorporated both spillways from the downstream face of the dam to a point 500 m beyond the plunge pool studied the hydraulic action of both spillways, the plunge pool, and the river downstream from the plunge pool. Another larger scale model (1 : 24) concentrated on the orifice intakes, the control gate, and the orifice outlet section from gate to downstream face of dam.

The 1 : 72 model included the two spillways, powerplant, and a 750-m length of the river downstream from the powerplant. Operating criteria required the service spillway to be operating at full capacity before any operation of the auxiliary spillway. The split level concept of the service spillway was adopted after extensive testing had shown that the trajected flow could be directed into the plunge pool more efficiently for up to six-gate operation. (The capacity of each gate is 425 m^3/s.) For more than six gates operating, flow conditions into the plunge pool were the same for either concept.

The configuration and depth of the plunge pool were determined so as to provide maximum energy dissipation, retention of the jet in a controlled area so that there was no deflection up the opposite bank under the auxiliary spillway; and minimal waves, surges, and water surface drawdown in the powerlant tailrace upstream of the plunge pool. The best configuration for the plunge pool was found to be a rectangular section 70 m wide and 100 m long. The pool was about 15 m deeper than the river channel in the main impact area and 12 m deeper on the downstream side (opposite the service spillway). The change in elevation between the two depths took advantage of sound rock in the area and acted as a deflector to help turn the service spillway jet upward. On the bank opposite the service spillway, it was found necessary to place a 5-m high concrete wall to help deflect the flow downstream when more than six service spillway gates were operating.

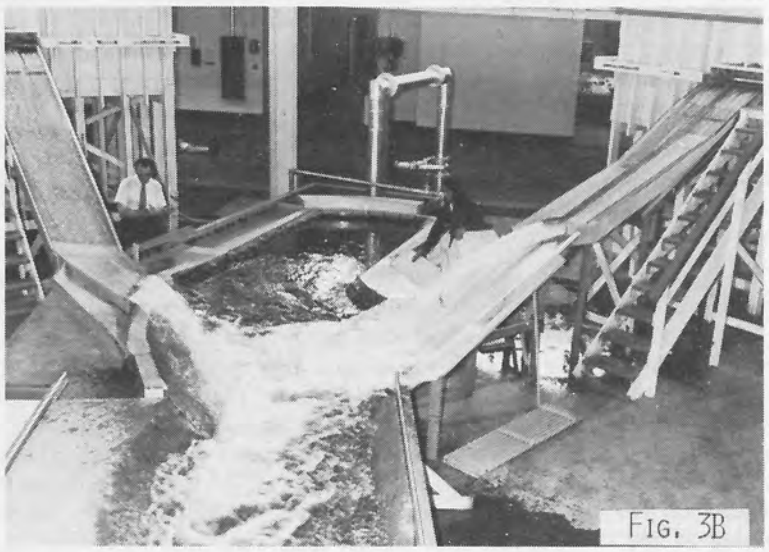

Fig. 3

(A) Hydraulic model used to study service and auxiliary spillways at Auburn Dam.
(B) Ten gates operating in service spillway, five gates operating in auxiliary spillway.

(A) Modèle hydraulique utilisé pour l'étude de l'évacuateur principal et de l'évacuateur auxiliaire du barrage d'Auburn
(B) 10 vannes en service à l'évacuateur principal, 5 vannes en service à l'évacuateur auxiliaire.

With only the service spillway operating, there was an average 600 mm water surface drawdown in the tailrace and waves less than 300 mm high, superimposed on a long period surge of about 1 m. There was extreme turbulence in the plunge pool but no turbulence upstream of the pool in the tailrace area. Downstream of the pool the river was extremely turbulent with more than six gates operating and no doubt will erode the river channel as much as if subjected to a comparable discharge without the dam in place. Impact forces on the concrete wall were as great as 35 m of water for the 10-gate operation.

The configuration of the exit bucket structure on the auxiliary spillway evolved after many tests. The final structure was superelevated at an 8 : 1 slope and a series of upward curved dentates placed at the end. The dentates varied in height from 2 m at the left side to 3.5 m on the right. The right side of each dentate was battered on a 3 : 1 slope to alleviate low pressure conditions measured during the testing program. The final bucket structure was very effective in deflecting the flow over a road at the end of the spillway and into the river channel. The flow was well distributed along the river and was not concentrated during any discharge. When six or more gates were operating, the jet opposed the end turbulence from the service spillway flow and helped suppress the heavy surges at the deflector wall. The impact pressures on the deflector wall were slightly lower when four or more auxiliary spillway gates were operating in addition to the 10 service spillway gates.

The model study also indicated that when it became necessary to operate the auxiliary spillway, the service spillway should be shut down to six gates and the auxiliary spillway operated at six gates. Any further increase in flow should be equally divided between the two spillways.

A 1 : 24 scale model was constructed to study the initial configuration of the bellmouth intake entrance, the spillway gate chamber equipped with 3.35- by 5.18-m upstream seal fixed-wheel slide gates, and the free trajectory chute. Refer to Figure 4. Three gated flow passages were included in the initial bellmouth to investigate pressures at critical locations. The initial bellmouth intake was composed of a 3 : 1 ellipse on the crown and invert which became tangent to the gate chamber 5.18 m downstream from the headwall. The 4 : 1 sidewall ellipse became tangent to the conduit wall 3.35 m downstream from the head-wall. The model was equipped to test aeration offsets in the sidewalls beginning at the downstream side of the gate slots and extending to the downstream side of the dam. One passage contained no offset, the second a 76-mm offset, and the third a 152-mm offset.

Undesirable negative pressures were found along the upper corner of the bellmouth entrance, just downstream from the tangent point of the sidewall ellipse. The pressures were very near the cavitation range, requiring a new design.

With partial gate openings, water impinging in the gate slots caused separation to occur along the lower portion of the wall as flow moved past the

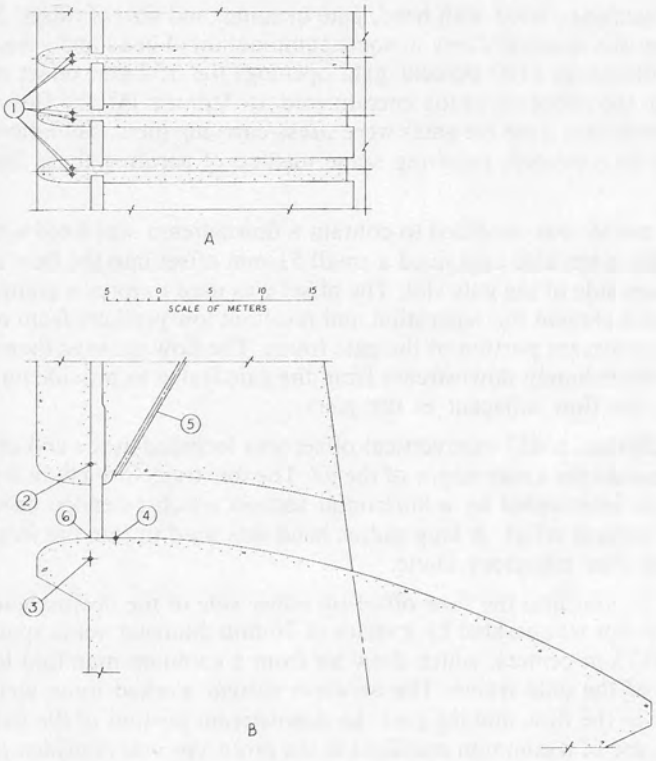

Fig. 4

(A) General plan initial design.

(1) Origin of bellmouth side ellipses.

(B) Sectional elevation initial design.

(2) Origin of bellmouth crown ellipse.
(3) Origin of bellmouth invert ellipse.
(4) Origin of chute.
(5) Air vent.
(6) Gate invert.

(A) Vue en plan du projet initial.

(1) Foyers du côté des ellipses d'entonnement.

(B) Coupe verticale du projet initial.

(2) Foyer de l'ellipse supérieure de l'entonnement.
(3) Foyer de l'ellipse inférieure de l'entonnement.
(4) Début du coursier.
(5) Trou d'air.
(6) Seuil de vanne.

downstream corner of the slot into the discharge chute. The impingement in the slots prevented the possibility of aerating flow adjacent to the conduit sidewalls and provided the potential for cavitation damage. The severity of

slot impingement varied with head, gate opening, and size of offset; however, each offset was unsatisfactory at some combination of head and gate opening. For free discharge (100 percent gate opening) the 152 mm offset remained vented for the range of heads encountered at Auburn. As the flow passage walls downstream from the gates were stress-carrying piers, cavitation erosion could not be tolerated, requiring some method of aerating these flow boundaries.

The model was modified to contain a downstream seal fixed-wheel gate. This modification also contained a small 51-mm offset into the flow along the downstream side of the gate slot. The offset was used to force a contraction in the flow and prevent the separation and resultant low pressure from occurring on the downstream portion of the gate frame. The flow passage then enlarged laterally immediately downstream from the gate frame to provide an aeration offset for the flow adjacent to the piers.

In addition, a 457-mm-vertical offset was included at the end of the gate frame to aerate the under nappe of the jet. The free trajectory chute was retained and was intercepted by a horizontal section which extended downstream from the vertical offset. A long radius bend was used to join the straight section to the free trajectory chute.

The 51 mm into the flow offset on either side of the downstream corner of the gate slot was aerated by a series of 76-mm diameter vents spaced vertically on 0.73-m centers, which drew air from a common manifold located in the sides of the gate frame. The aeration system worked quite well in supplying air to the flow moving past the downstream portion of the gate frame. However, use of a common manifold in the prototype was considered impractical because water could fill a portion of the manifold in areas where pressure was positive and choke off the vents supplying air to low-pressure areas. As space was not available in the prototype to introduce air through a number of independent vents, this concept was abandoned.

The invert offset downstream from the gate frame was also found to fill with water when operating at partial gate openings, requiring additional investigations.

To eliminate the above problem areas, a new concept was developed which included a bellmouth entrance which accelerated the flow on all surfaces throughout its length, reaching the gate chamber at the tangent point of the minor axis. Refer to Figure 5. The sidewall ellipse was 6 : 1 beginning on the upstream end of the stoplog guide and extending to the upstream side of the gate slot. The 2.48 : 1 crown and invert ellipse began at the headwall and extended to the upstream side of the gate slot. The intake width was increased by 203 mm on the upstream side of the gate slot to accommodate further decrease in flow passage width due to an encroaching elliptical shape included on the sides of the downstream portion of the gate frame. The elliptical shape began at the downstream corner of the gate slot and extended to the end of the gate frame. The shape consisted of the beginning portion of a 6 : 1 ellipse

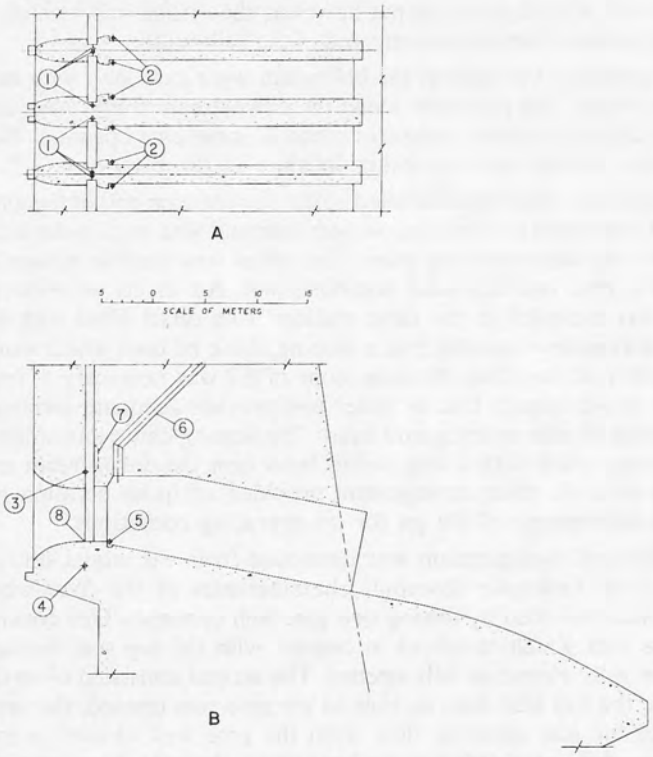

Fig. 5

(A) General plan recommended design.

(1) Origin of bellmouth side ellipses.
(2) Origin of downstream frame ellipses.

(B) Sectional elevation recommended design.

(3) Origin of bellmouth crown ellipse.
(4) Origin of bellmouth invert ellipse.
(5) Origin of chute.
(6) Air vent.
(7) Top seal seat offset.
(8) Gate invert.

(A) Vue en plan du projet proposé.

(1) Foyers du côté des ellipses d'entonnement.
(2) Foyers du cadre des ellipses aval.

(B) Coupe verticale du projet proposé.

(3) Foyer de l'ellipse supérieure d'entonnement.
(4) Origine de l'ellipse inférieure d'entonnement.
(5) Début du coursier.
(6) Trou d'air.
(7) Élargissement pour le siège du joint supérieur.
(8) Seuil de vanne.

which extended 1.22 m downstream and encroached 254 mm into the flow passage on each side. The downstream crown of the gate frame was also

equipped with the elliptical shape; however, the origin was located 152 mm above the crown of the downstream end of the bellmouth.

The pressures throughout the bellmouth were excellent, with no separation zones noted. The pressures along the downstream frame were also good, with only slightly negative pressures noted at some gate openings. No cavitation damage should occur at either location in the prototype.

An aeration offset was installed at the downstream end of the gate frame. The offset amounted to 406 mm on each side and was used to aerate the flow adjacent to the stress-carrying piers. The offset was sized to remain open for all possible gate opening-head combinations. An invert aeration offset of 305 mm was included at the same station. This offset filled with water for lower head releases requiring that a sloping chute be used which would drain in the vicinity of the offset. A chute slope of 0.2 was necessary to insure that the offset would remain free of water and provide adequate aeration for all combinations of gate opening and head. The sloping chute joined the original free trajectory chute with a long radius bend near the downstream end of the dam. The aeration offset arrangement provided adequate aeration along the sides and undernappe of the jet for all operating conditions.

A gate well configuration was developed from the model studies which minimized the hydraulic downpull characteristics of the fixed-wheel gate. This was accomplished by testing two gate well concepts. One consisted of a continuous seat which remained in contact with the top seal throughout its travel from fully closed to fully opened. The second consisted of an open section above the top seal seat, so that as the gate was opened, the seal moved away from the seat allowing flow from the gate well to vent to the downstream side of the gate, reducing the head above the gate. By using a combination of the two concepts, a workable design was achieved which reduced the peak downpull values at intermediate gate openings, and added stability to the gate at larger openings.

CHOKE CANYON DAM

Choke Canyon Dam will be located on the Frio River, a short distance upstream from the confluence of the Frio and Nueces Rivers, about 120 km northwest of Corpus Christi, Texas. It will be the principal feature of the Nueces River Project and will provide annually 30 000 000 m^3 of supplemental municipal and industrial water for the city of Corpus Christi. Recreation, fish and wildlife, and incidental flood control benefits will also accrue from construction of the project.

The reservoir created by Choke Canyon Dam will impound 880 800 000 m^3 of water and will cover an area approximately 9 000 hectares. The dam will be a homogeneous embankment 35 m high and

about 5.6 km long. The design includes a chimney drain, drainage blanket, and toe drains.

The dam will be located in a broad valley with gently sloping abutments. Bedrock at the damsite, upon which the dam cutoff trench and the concrete structures will be founded, belongs to the Catahoula formation and typically consists predominantly of pyroclastic materials. The Catahoula formation is made up of siltstone, claystone, sandstone, tuff, brecia, and shale and is found to be at depths of over 50 m.

The alluvial flood plain deposits overlying bedrock ranges in thickness from 0 to 15 m and consists of lean clays, silty sands, and sands and gravels. The damsite is in an area of apparent seismic stability where no earthquakes have been reported or recorded in historic time.

The inflow design flood with a 15-day volume of 3 420 000 000 m^3 is 9 148 m^3/s and a 15-day volume of 3 420 000 000 m^3. A moderate frequency flood with a peak of 4 850 m^3/s was also developed for use in developing designs for the spillway energy dissipator.

Designs for the spillway structure at Choke Canyon Dam were influenced by a number of factors. These factors include the large inflow design flood, reservoir right-of-way constraints, the extreme length of the dam (5.6 km), the unfavorable topography and geology, the highly erodible foundation, the relatively low Froude number of the energy dissipator, and the wide variation in tailwater elevations.

The inflow design flood with a 15-day volume of 3 400 000 000 m^3 is nearly four times the active capacity of the reservoir. Reservoir right-of-way constraints due to environmental considerations and the backwater effects on upstream communities and utilities greatly limited the acceptable amount of temporary flood storage space that could be used. In addition, the extreme length of the dam severely limited the maximum height of embankment that could be economically constructed to accommodate the inflow design flood. Unfavorable topography and geology conditions necessitated that the location of the spillway structure be far away from the natural stream resulting in long inlet and outlet channels. Because of the highly erodible characteristics of the Catahoula formation, an energy dissipator was deemed necessary for the terminal structure of the spillway. The low Froude number and the wide range and rapid change in tailwater elevations had a marked influence on the configuration of the energy dissipator.

In view of the above discussed factors, a wide reinforced concrete gated chute type with hydraulic jump stilling basin was chosen for the spillway structure (see Fig. 6). The structure will be controlled by seven 15.0-m-wide by 7.23-m-high radial gates with a design capacity of 7 130 m^3/s. Manually operated electric hoists, with a backup auxiliary power source, have been included for controlling the gates. In addition, an electric automatic gate control system has been provided to ensure planned gate operation in the absence of an operator. A hydraulic jump basin energy dissipator will be provided to

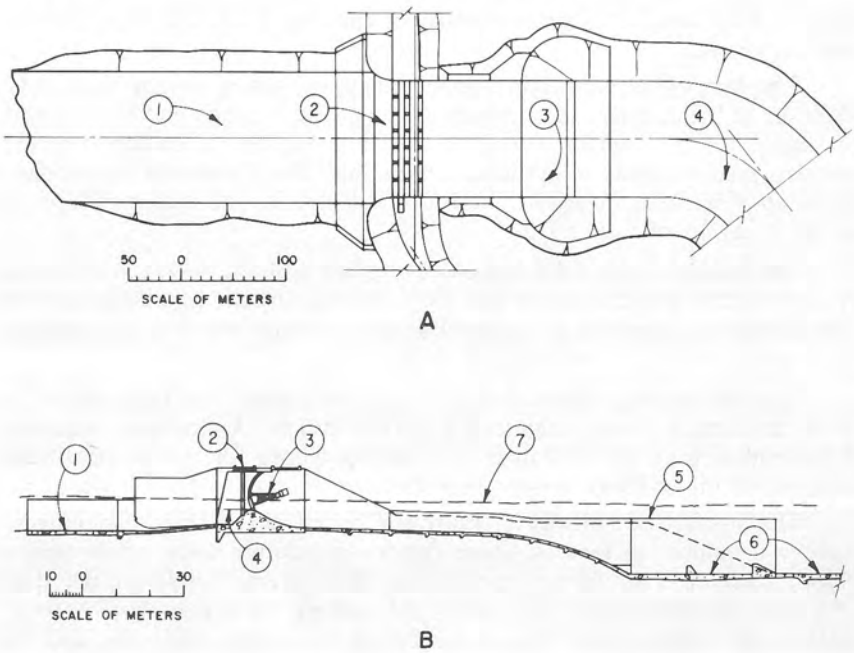

Fig. 6

(A) General plan.

(1) Inlet channel.
(2) Gate structure.
(3) Energy dissipator.
(4) Outlet channel.

(B) Profile along spillway centerline.

(1) Elevation 54.4.
(2) Elevation 73.8.
(3) Seven 15.00-meter × 7.23-meter gates.
(4) Elevation 60.81.
(5) Elevation 57.9.
(6) Elevation 41.0.
(7) Original ground surface.

(A) Vue en plan.

(1) Canal d'amenée.
(2) Structures du vannage.
(3) Dissipateur d'énergie.
(4) Canal de décharge.

(B) Coupe longitudinale sur l'axe de l'évacuateur.

(1) Cote 54,4.
(2) Cote 73,8.
(3) 7 vannes de 15 m × 7,23 m.
(4) Cote 60,81.
(5) Cote 57,90.
(6) Cote 41,00.
(7) Niveau du terrain naturel.

protect the erodible foundation material at the terminal end of the structure. Appropriate appurtenances will be included in the basin to ensure acceptable energy dissipation and flow conditions. A 350-m-long inlet channel will con-

vey flows to the gate structure and a 950-m-long outlet channel will return flows to the natural stream well downstream from the dam. The inlet and outlet channels are of sufficient size to preclude erosion from high velocity flows. In addition, soil cement and armor rock are provided adjacent to the structure to protect the foundation against undercutting. For economic reasons, both the inflow design and the moderate frequency floods were used in developing the design of the hydraulic jump energy dissipator. The structure designs were optimized for the moderate frequency flood and then evaluated for satisfactory operation for the inflow design flood.

Hydraulic model studies were used to verify the performance of the low Froude number stilling basin, to determine the optimum configuration of the spillway inlet and outlet channels, and to develop a discharge rating curve for the gated spillway.

Figure 7 shows the 1 : 80 scale model with the preliminary configuration. The trapezoidal approach channel had a bottom width of 112 m and 2 : 1 side slopes. The guidewalls, or piers, on each side of the spillway consist of two 15.24-m radius quarter circles separated by a 6.10-m long tangent section. Flow through the spillway was controlled by seven radial gates on an ogee crest section. Downstream from the crest section, a near horizontal apron and

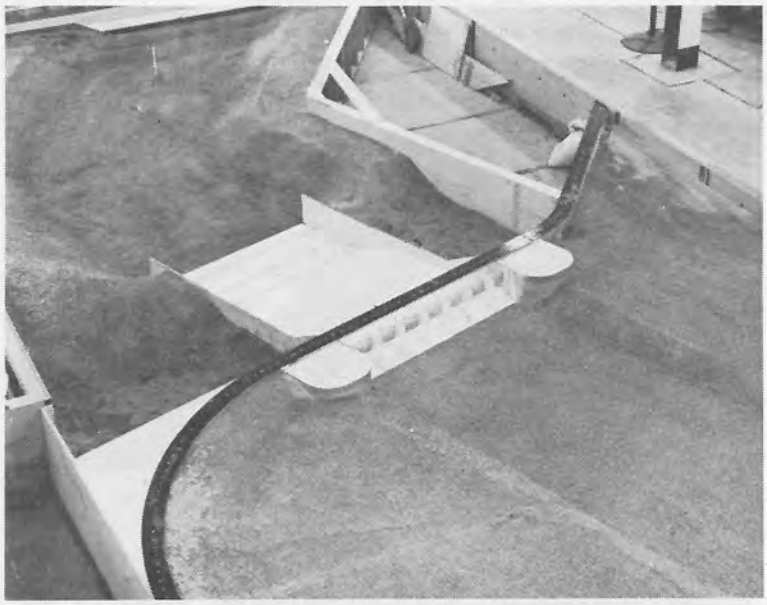

Fig. 7

1 : 80 Scale model preliminary arrangement.

Modèle au 1/80. Dispositions d'ensemble initiales.

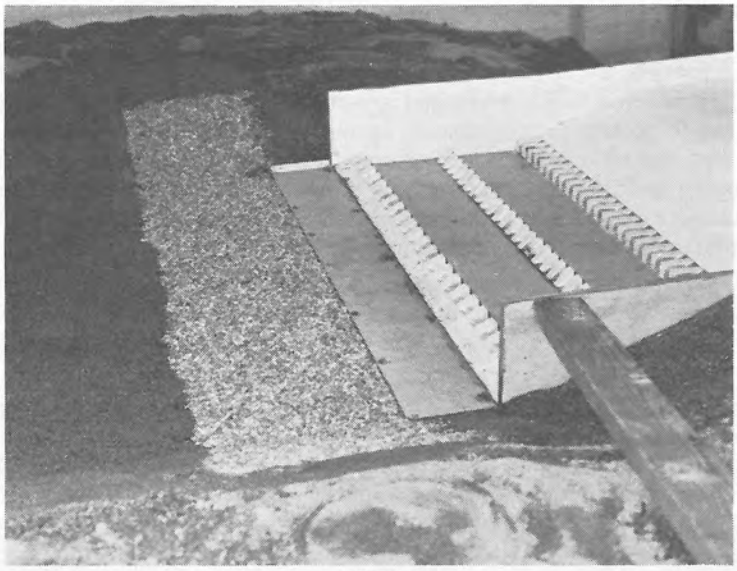

Fig. 8
Final stilling basin configuration.
Configuration finale du bassin d'amortissement.

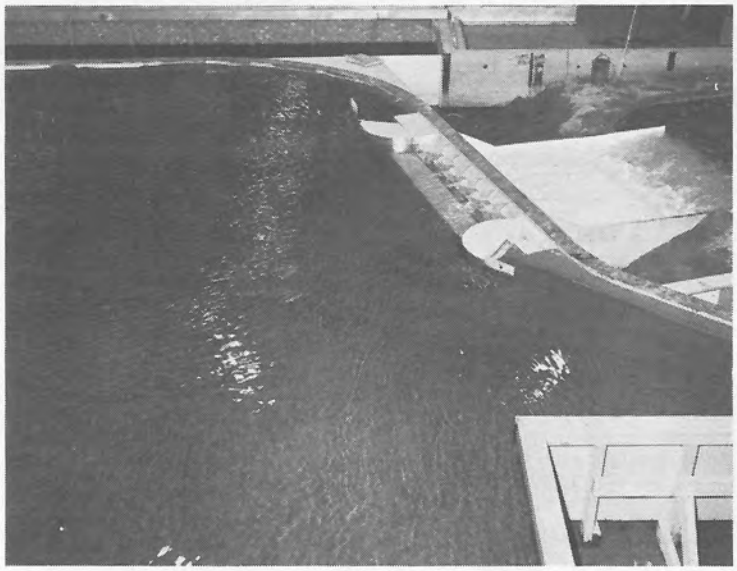

Fig. 9
Flow in final configuration. 7 100 m^3/s.
Écoulement de 7 100 m^3/s en configuration finale.

a vertical curve preceded the hydraulic jump stilling basin. The chute and stilling basin were 112 m wide. Downstream from the stilling basin, a trapezoidal excavated channel returned the flow to the river channel.

The chute blocks, baffle piers, and dentated end sill were dimensioned according to criteria developed in the Bureau of Reclamation hydraulic laboratory [2]. The chute blocks at the upstream end of the basin were 1.87 m wide, 2.74 m high, and spaced 1.87 m between blocks. The baffle blocks, placed 16.65 m downstream from the chute blocks, were the same size as the chute blocks. The baffle blocks were placed opposite the spaces in the row of chute blocks. The 6.94-m long end sill was 37.26 m downstream from the baffle blocks. The dentates on the end sill were 2.44 m wide and 3.15 m high. In the initial design, the 18.90-m high sidewalls and the basin invert continued 8.8 m beyond the end sill.

The model tests showed that a significant amount of erosion occurred in the spillway approach channel at the side piers. To alleviate this problem, the approach channel was deepened 1.68 m and widened 13.00 m, the 2 : 1 slope channel side slopes were transitioned to 3.5 : 1 side slopes in a 27-m-long section immediately upstream from the piers, and the piers were changed to 24-m radius semi-circular sections (90° on the left side and 108° on the right side) with short tangent wing walls extending toward the dam embankment. These modifications eliminated the erosion problems and improved the flow distribution entering the crest section. The stilling basin performed very well. However, for economy purposes, the sidewalls were shortened by 8.80 m. The apron length was not changed. With the shortened walls, there was some erosion near the left side of the apron downstream from the end sill. There was no erosion on the right side. A blanket of riprap placed on the invert and sloping sides of the trapezoidal channel for 30 m protected the channel from the unsymmetrical erosion. Figure 8 shows the final basin configuration. The basin operation at 7 100 m^3/s is shown on Figure 9.

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SUMMARY

The Bureau of Reclamation frequently must design spillways and their energy dissipators for which there is little prototype experience to rely on. These atypical designs must be verified by hydraulic model testing. Three recent spillway designs, only one of which has been constructed at this time, are discussed. Each spillway is of a different type. Crystal Dam Spillway is representative of an uncontrolled ogee spillway crest constructed integrally with the mass of a concrete arch dam. The spillway trajects its flows into a plunge pool energy dissipator located in a narrow canyon. The use of the hydraulic model not only identified several problems related to the spillway crest and piers and the plunge pool dissipator but determined the solutions.

Auburn Dam, still in the design stage, has orifice-type spillways located on each side of the dam. The spillway intakes submerged in the reservoir, the control gates, and the energy dissipators all presented questions which could only be answered by studying hydraulic models. Two models were constructed, one to study the overall hydraulic operation of the two spillways and the other to study problems associated with the intakes and the area in and adjacent to the control gates.

Choke Canyon Dam is under construction. The spillway is a gated chute type with a hydraulic jump stilling basin. The designs were influenced by several major factors which included a large volume inflow design flood with a 15-day volume four times the volume of the reservoir, an extremely long dam which tended to make surcharge storage uneconomic, unfavorable topography and geology, a highly erodible foundation, and a relatively low Froude number for the stilling basin. A hydraulic model was used to verify and improve the designs. Inlet flow conditions and stilling basin performance were improved by modifications developed in the hydraulic model study.

SOMMAIRE

Le U.S. Bureau of Reclamation doit fréquemment projeter des évacuateurs et leurs dissipateurs d'énergie pour lesquels on ne peut s'appuyer sur pratiquement aucune expérience antérieure. Ces projets d'un type nouveau doivent être vérifiés par des essais sur modèles hydrauliques. On discutera ici de trois projets récents d'évacuateurs, dont un seul a été construit jusqu'alors.

Chaque évacuateur est d'un type différent. L'évacuateur de Crystal Dam est un déversoir de surface, sans vanne, à coursier en S et qui est intégré au corps d'un barrage-voûte en béton. L'évacuateur envoie la lame déversante dans un bassin de dissipation situé dans cañon étroit. L'emploi d'un modèle hydraulique a servi non seulement à identifier plusieurs problèmes se rapportant à la crête, aux piles, et au bassin d'amortissement, mais aussi à trouver les solutions.

Le barrage d'Auburn Dam, encore au stade des études, est pourvu d'évacuateurs à orifice situés de chaque côté du barrage. Les orifices en charge de l'évacuateur, les vannes de commande et les dissipateurs d'énergie présentaient tous des problèmes dont la résolution ne pouvait être obtenue que par une étude sur modèle. Deux modèles ont été construits, l'un destiné à étudier le fonctionnement hydraulique général des deux évacuateurs, et l'autre, les problèmes relatifs aux prises d'eau et aux zones intérieures et adjacentes aux vannes régulatrices.

Le barrage Choke Canyon Dam est en cours de construction. Son évacuateur est du type à coursier, commandé par des vannes avec un bassin d'amortissement à ressaut hydraulique . Le projet a été conditionné par plusieurs facteurs importants notamment une forte crue de 15 jours et d'un volume 4 fois supérieur à celui de la retenue, un barrage au couronnement extrêmement long, tendant à rendre peu économique une surélévation du niveau en temps de crue, une topographie et une géologie défavorables, des fondations fortement érodables et un nombre de Froude relativement bas pour le bassin d'amortissement. Un modèle hydraulique a été utilisé pour vérifier et améliorer le projet. Les conditions d'entonnement et le fonctionnement du bassin d'amortissement ont été améliorés par des modifications étudiées sur modèle réduit.