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MODEL STUDY OF GREEN MOUNTAIN DAM SPILLWAY
COLORADO-BIG THOMPSON PROJECT

By

J. H. Douma

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Denver, Colorado
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MODEL STUDY OF GREEN MOUNTAIN DAM SPILLWAY

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A model was utilized in the hydraulic design of the Green Mountain Dam spillway and was the basis for important decisions regarding several features of the design. Particular studies were made of the inlet transition, spillway crest and gate structure, the high-velocity chute, and the erosion of the river bed downstream from the spillway. Primary objectives in the model studies were: (1) to establish the minimum radius of curvature of the inlet transition for satisfactory hydraulic performance, (2) to determine the efficiency of the spillway crest and bridge piers, (3) to obtain the most economical and effective hydraulic design of the spillway chute, and (4) to determine the extent of erosion downstream from the spillway and the feasibility of employing some type of stilling basin for scour prevention.

A plan and elevation of the final design of the concrete spillway are shown in figure 1. The inlet transition is curved in plan with a minimum center-line radius of 200 feet. The spillway crest has an elevation of 7,928 feet and a net length of 75 feet. Piers 2 feet 6 inches wide support three radial gates, each 22 feet high and 25 feet wide, that control the upper 42,000 acre-feet of storage. The spillway chute converges at varying rates from a width of 80 feet at the crest to 30 feet at station 14+22.19. A smooth transition from a 30-foot wide rectangular section to a trapezoidal section with a 20-foot wide bottom and 1/2 to 1 side slope is provided between station 14+22.19 and station 15+94.13; the trapezoidal section extends to the end of the chute. The slope of the chute, varying from zero at the crest and end of the channel to 50 percent 130 feet from the end of the channel, was determined to fit the mountainside topography, providing for a total fall of 228 feet.

The model was designed and constructed in the Bureau of Reclamation, Denver, hydraulic laboratory to a linear scale of 1 to 40, and required a maximum discharge of 2.47 second-feet to represent the prototype design flood of 25,000 second-feet. Except for the sloping inlet and chute walls, which were of neat cement mortar, the model was constructed of smooth pine and red-

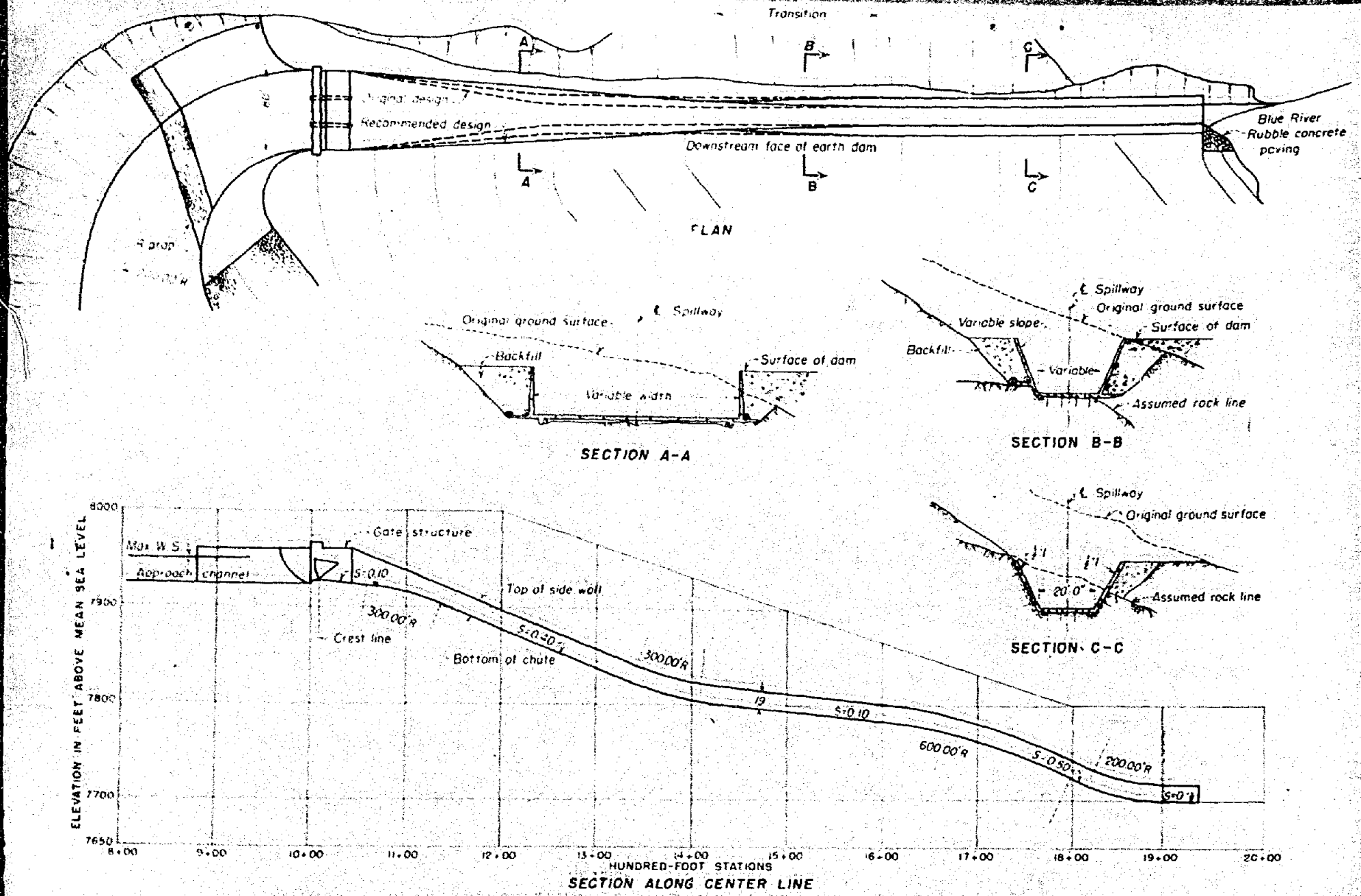

FIGURE 1 – PLAN AND SECTIONAL ELEVATIONS OF SPILLWAY – GREEN MOUNTAIN DAM

wood, painted with gray enamel for a protective covering. It was estimated that the value of n in Manning's formula for the finished surface was 0.012.

INLET TRANSITION

A problem of considerable economic importance involved the size of the inlet channel to the chute. Since the material to be excavated is suitable for construction of certain zones of the earth structure and is also conveniently located, a large inlet is feasible, provided rock is not encountered. Should rock be encountered, the smallest radius inlet hydraulically suitable will be the most economical.

Tests of 400-foot and 200-foot center-line radius inlets were studied in the model. Except for some disturbance in the form of surface waves at the inside wall of the inlet, observations and measurements showed the smaller inlet to be practically as satisfactory hydraulically as the larger inlet. Water-surface profiles at station 9+90 and the discharge coefficients were identical for the two cases. Due to the surface wave disturbance in the 200-foot inlet, it was considered undesirable to further reduce the inlet radius.

Observations and measurements of the water surface in the chute indicated slightly unsymmetrical but not severe flow conditions which undoubtedly originated in the curved inlet. A study of the unbalanced flow pattern in the chute led to the conclusion that energy of flow per unit width entering the chute was greater on the left side than on the right side. From the theory of a curved channel, to have equal energy distribution in a transverse section, superelevation of the bottom must be introduced. Accordingly, the inlet bottom was superelevated, but there was no improvement in chute flow conditions. It is apparent that other factors influence the flow through this type of curved channel. The unsymmetrical change from a trapezoidal to rectangular section is probably very significant in the development of the flow pattern.

SPILLWAY CREST CALIBRATION

To check the capacity of the spillway crest, careful discharge measurements were made with the 200-foot center-line radius inlet. The results are shown in figure 2. The experimental discharge coefficient in the universal formula $Q = CLH^{3/2}$

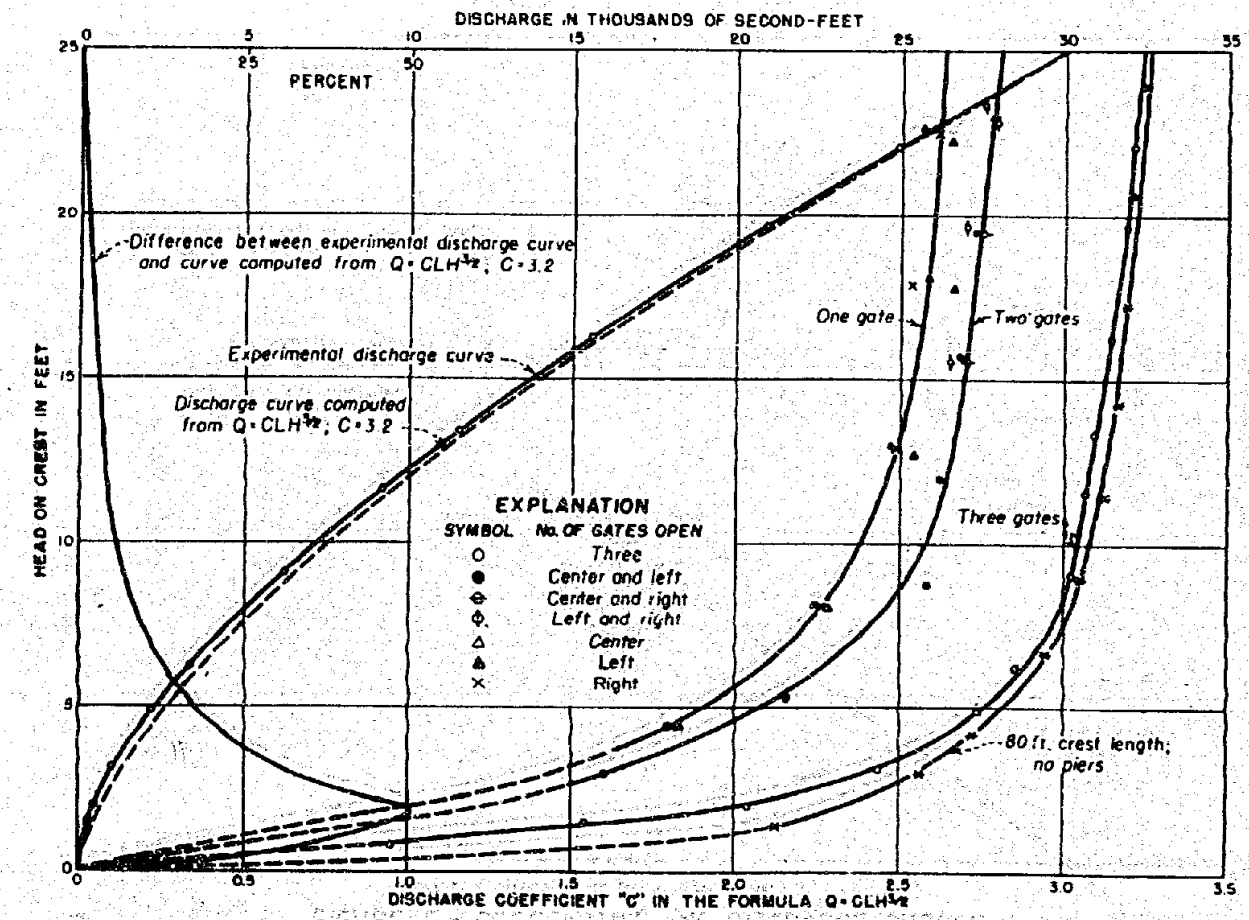

FIGURE 2 - COEFFICIENT AND DISCHARGE CURVES

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for all gates open and 25,000 second-feet is 3.21, which is in excellent agreement with the value of 3.20 used for the original design. For the maximum operating head, the two-gate and one-gate discharges are 58 and 27 percent of the three-gate discharge, and the corresponding coefficients are 2.76 and 2.62, respectively. For one-gate operation, the center gate has the largest coefficient, the left slightly smaller, and the right smallest. The variation, however, is less than four percent.

The usual procedure in establishing a discharge rating curve for design purposes is to select a discharge coefficient to suit the particular type of crest involved and compute several points of the curve using the selected coefficient for all heads. The coefficient for any crest shape is by no means constant but decreases with decrease in head, as shown in figure 2. The percent difference in experimental and design rating curves becomes large for small heads, being 50 percent for a prototype head of two feet in this case. The design rating curve, based on a single constant coefficient, therefore, is not reliable in determining small discharges.

An expression for the variation in discharge coefficient with head, determined to fit the experimental coefficient curve for three-gate operation, was found to be

$$C = \left[10\left(\frac{H}{H'}\right)^{0.007} - 9 \right] \left[1 - \frac{0.0007N}{(H/H')^2} \right] C',$$

where H' and C' are the maximum design head and coefficient, H is any head less than maximum and C is the corresponding coefficient, and N is the number of contractions. Two piers yield four contractions, since the sides of the crest opening are considered completely suppressed with the smooth inlet transition employed.

Since shape and length of crest and number, size, and shape of bridge piers influence the coefficient, the above expression is strictly applicable to the Green Mountain structure. It is useful, however, in determining approximate rating curves for generally similar structures. For very low heads, friction becomes more significant in the model than the prototype, so the model coefficients are somewhat smaller than those for the prototype for corresponding heads. Prototype measurements are required to determine the relationship between model and prototype coefficients for very low heads.

The efficiency of the bridge piers was determined by removing the piers and calibrating the crest with one opening 80 feet in width. The coefficient is increased only 0.7 percent to 3.23 for no piers and the maximum head. Figure 2 further shows that the efficiency of the piers decreases with decreasing head.

SPILLWAY CHUTE

Operation of the originally designed chute, shown by dashed lines in figure 1, showed that the channel convergence below the gate structure was too rapid, resulting in the transverse-wave phenomenon often encountered in tapered channels. Starting with waves which originated at the sides of the upper end of the chute, the waves crossed forming a swell approximately midway between the side walls, traveled laterally to the sides, and were reflected back to the center. This action was repeated continuously, forming four swells in the length of the chute. As indicated in the model, the waves, if reproduced in corresponding degree in the prototype, would overtop the originally specified 15-foot high chute walls. Several smaller angles of convergence were tested and the design shown in figure 1, while not completely eliminating the waves, proved satisfactory. The height of chute side walls was increased to 19 feet, providing a minimum freeboard of 7 feet, which provides allowance for considerable entrainment of air in the prototype chute flow. Since rock was encountered at about station 15+00, it was economical to change to a trapezoidal section with 1/2 to 1 side slopes. A minimum length of transition from the upstream rectangular section to the lower trapezoidal section of 170 feet was required to give satisfactory flow conditions in the lower part of the chute. No serious adverse flow conditions within the chute resulted from any combination of one- and two-gate operation.

An important design feature for successful hydraulic operation of the chute is the design of vertical curves. Generally, vertical curves are introduced to fit the topography. Too steep vertical curves result in springing of the high-velocity stream from the chute bottom seriously limiting safe operation. The theoretical equation for the jet trajectory based on the mean velocity at the beginning of the curve is

$$y = -x \tan \theta - \frac{g}{2V_m^2 \cos^2 \theta} x^2.$$

Where x and y are the horizontal and vertical coordinates with the P.C. of curve as origin, θ is the angle the chute bottom makes with the horizontal at the P.C., V_m is the mean velocity in a transverse section at the P.C., and g is the acceleration of gravity. This equation was used in designing the vertical curve of the Kittitas chute, Yakima project. Recent operation of this chute showed the vertical curve to be entirely too steep. So much water sprang from the vertical curve that safe operation was limited to about one-half of the maximum design discharge.

To prevent any disruption of the flow, the trajectory should be designed for at least the maximum velocity in the transverse section at the beginning of the trajectory. Assuming a maximum velocity equal to 125 percent of the mean velocity, the trajectory equation becomes

$$y = -x \tan \theta - \frac{g}{3.15 V_m^2 \cos^2 \theta} x^2.$$

When a mass of water-air mixture passes over a convex vertical curve at high velocity, the reduction of internal pressure within the mass due to centrifugal action results in an expansion of the air bubbles within the mass and an increase in the tendency toward disruption of the flow. A more conservative design formula would perhaps then be

$$y = -x \tan \theta - \frac{g}{4 V_m^2 \cos^2 \theta} x^2.$$

All of the convex vertical curves for the Green Mountain spillway fall well within the maximum limit given by the latter expression, so satisfactory flow conditions over the curves are expected.

An error is commonly introduced in the hydraulic computations for flow in steep chutes by use of the mean velocity. The average energy of flow at any given cross section is correctly determined from the average velocity. Average velocity, as here used, is the arithmetical or algebraic average of the velocities of all the particles, whereas the mean velocity, as ordinarily used, is the average velocity over the cross section of the channel taken by areas. Unless the velocity is uniform over the entire cross section of the channel, these two terms are not identical. The assumption that the velocity is uniform over a given cross section is never strictly true. In recent computa-

tions a kinetic energy factor, α, has been introduced so that the true average kinetic energy or velocity head is given by

$\alpha \frac{V_m^2}{2g}$, where V_m is the mean velocity.

Measurements and computations were made in the model to determine the value of α for high velocity flow. An apparent experimental energy line was plotted by adding apparent velocity heads, as determined by $\frac{V_m^2}{2g}$, to water-surface elevations along the chute. The true energy line was obtained by subtracting the channel friction loss, as determined by Manning's formula, from the reservoir elevation. The discrepancy between the apparent and true energy lines is a measure of the value of the coefficient α. In computing the hydraulic values of table I, converted to prototype quantities from model values, no attempt has been made to adjust prototype values for possible difference in prototype and model roughness values because of the uncertainty of these values and the added effect of entrained air. The point to be noted is that the coefficient, α, inherent in the velocity distribution, is important in determining the true energy line for high velocity flow. A point of interest is that α increases in value as the velocity increases, indicating that the velocity distribution becomes less uniform in a cross section as the velocity increases.

TABLE I - HYDRAULIC PROPERTIES OF CHUTE FLOW - 25,000 SECOND-FEET

Station	Mean depth D_m	Hydraulic radius R_m	Mean velocity V_m	Apparent vel. head $V^2_m/2g$	True vel. head $V^2_m/2g$	Kinetic energy factor α
9+90	22.50		13.6			
10+50	10.38	8.24	30.3	14.2	15.0	1.06
11+10	8.22	6.69	42.2	27.7	29.2	1.05
11+77	7.14	5.80	56.4	49.4	52.6	1.07
13+00	7.73	5.68	75.3	88.0	96.5	1.10
14+22.19	10.10	6.05	82.5	105.6	123.6	1.17
15+19	10.87	6.33	83.9	109.4	128.7	1.18
15+94.13	11.25	6.40	86.5	116.2	133.0	1.15
16+97	11.10	6.34	88.0	120.2	149.2	1.24
17+96	10.20	6.00	97.3	147.0	181.0	1.23
18+61	9.88	5.86	101.2	159.0	201.0	1.26
19+33	9.76	5.84	102.5	163.2	196.6	1.22

INVESTIGATING EROSION OF THE RIVER BED

The channel of the Blue River at the Green Mountain Dam site is of solid rock formation with a 15-foot covering of bed material, consisting of a combination of sand and gravel, closely interspersed with larger rock fragments, all intimately associated in a cohesive mass. Due to the improbability of excessive scouring by the high-velocity chute flow, it was considered economical to let the flow shoot out into the river without the use of any type of stilling basin. There was some concern, however, as to the formation of a gravel bar downstream from the powerhouse. Such a bar would raise the tail water in the powerhouse tailrace, effectively reducing the power head.

In order to study the erosion of the river bed, it was first necessary to select a material that would suitably represent the natural alluvium. A scouring material representative of the river overburden was thought to be approximated by a mixture of 75 percent of 1/4- to 1-inch pit-run gravel and 25 percent of Cherry Creek sand. The sand, scaled to prototype dimensions, would be representative of small pebbles and the 1-inch gravel to 3-foot 4-inch rock. The use of an accurately analogous bed material to obtain reliable quantitative scouring is a practical impossibility, but it was felt that satisfactory qualitative results could be obtained by the use of the above cohesionless and less stable model material.

A series of tests was made to investigate the formation of the downstream bar and to determine the possible existence of destructive currents at the downstream toe of the earth dam. For no conditions of flood did destructive eddies form at the downstream toe of the earth dam, and scouring occurred well downstream. The resulting bar became higher and moved farther downstream for floods up to 7,500 second-feet. For larger floods, the bed material was carried out of the model. A bar-crest envelope drawn from the experimental data, became horizontal for a flood of about 10,000 second-feet. The significance is that the highest bar will be formed for this flood. For larger floods, bottom velocities over the crest of the bar become sufficiently above critical for the bed material so that the bar is flattened out and does not build as high.

Due to the complexities inherent in similarity of scour and bed material, the model gives only an indication that a bar of considerable height will be formed. Whether the bar will be as high or higher than that shown by the model will depend on

similarity of securing and duration and rate of subsidence of floods. For a slow rate of subsidence, much of the bar may be washed downstream as the downstream water level decreases. The best field plan is to investigate the bar after each flood to determine the feasibility of excavating the bar to regain the lost power head. Tests showed that when a bar, resulting from a previous flood, is excavated to the original river bed, a second flood of the same intensity will form a second bar farther downstream but not as high as the first. As the high-velocity stream shoots down the canyon, its excess kinetic energy is expended in turbulence and boundary friction. When sufficient energy has been expended to result in tranquil flow the hydraulic jump forms. After several floods of varying intensities, the river-bed overburden will be carried downstream beyond the distance required to produce tranquil flow in the rock channel. With the erodible material beyond reach of the high-velocity stream, further formation of bars need not be anticipated.

STILLING BASIN INVESTIGATIONS

Due to the possible excessive cost of excavation of bars or loss of power revenue, it was desirable to investigate the possibility of incorporating a stilling basin in the design which would prevent the formation of bars. The roller-bucket and horizontal hydraulic jump type of stilling basins were investigated. For the purpose of determining the proper bucket dimensions, a volume-energy curve, figure 3, for satisfactory bucket operation, was drawn from available experimental data. The depth-energy curves for bucket radii of 40, 60, and 75 feet were obtained from the volume-energy relationship. Green Mountain discharge energy curves for several widths are also plotted. The procedure in determining the proper bucket dimensions by the use of these curves is to select a trial bucket width and radius and determine the required bucket depth corresponding to the design discharge and energy. Cost estimates should be made for several trial widths and radii and the corresponding bucket depths, and the proper combination of dimensions to use is the one of least cost.

The design curves are not presented as being conclusive but merely as a guide in the design of a roller-bucket stilling basin. The experimental data from which these curves are drawn are far from sufficient, and whether the variables are properly related has as yet not been established. For example, it is not

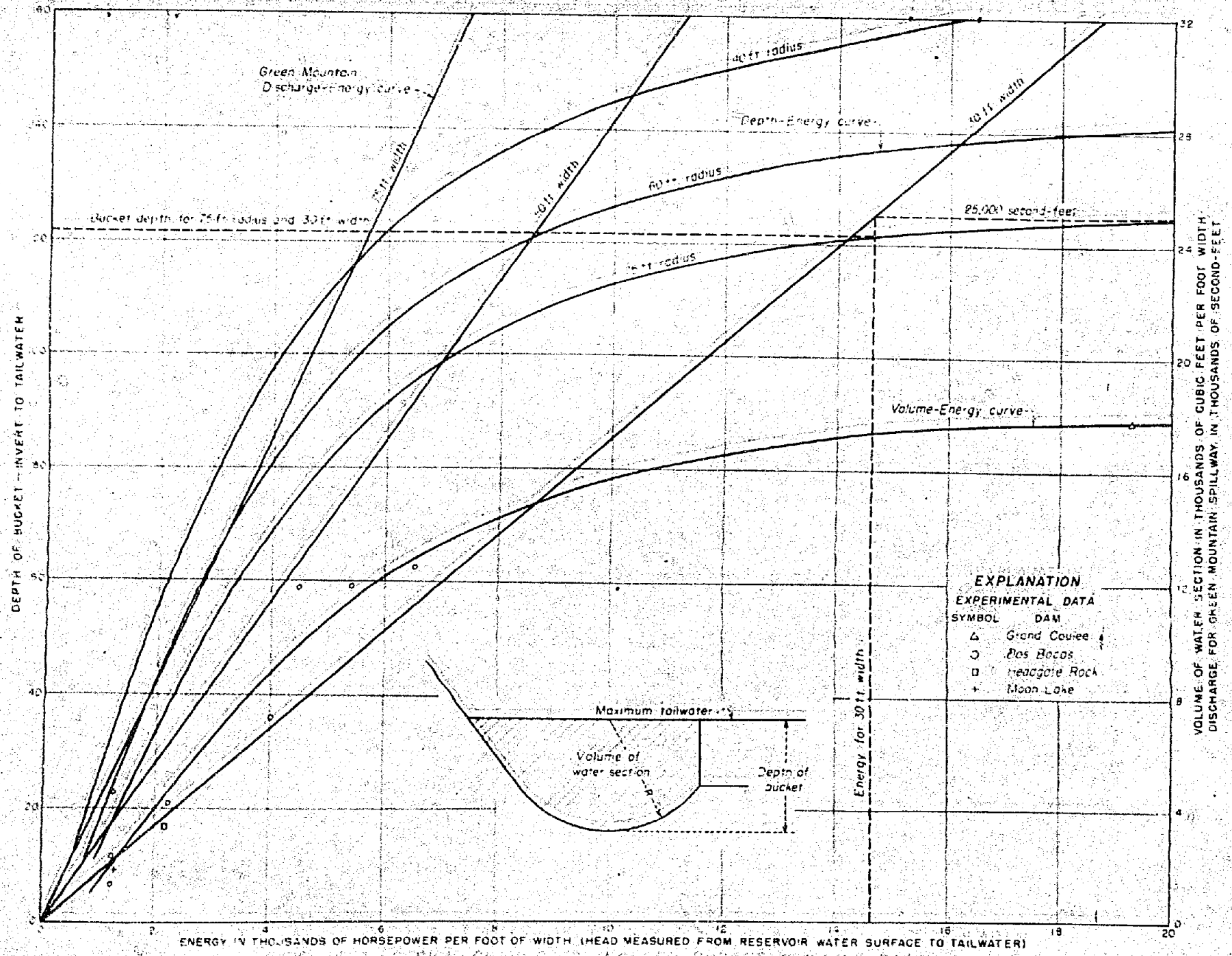

FIGURE 3 - DESIGN CURVES FOR ROLLER-BUCKET STILLING BASIN

known whether more of the water section should be included in the volume variable and whether the volume-energy relationship is independent of the bucket width.

To check the validity of the design curves, the six designs shown in figure 4 were tested in the model. Bucket design No. 1 had the dimensions: 40-foot radius, 100-foot depth, and 30-foot width. Model operation showed this bucket to be satisfactory up to 5,000 second-feet as compared to 7,000 second-feet given by the design curves. The depth of bucket No. 2 was increased to 120 feet, and its satisfactory operation was 7,500 second-feet compared to 10,000 second-feet given by the design curves. Buckets Nos. 3, 4, 5, and 6 have the same principal bucket dimensions as No. 2, but the shape of excavation beyond the end of the bucket is different for each. All of these designs were inferior to No. 2, No. 6 being definitely the poorest design since the jet followed up the 1 to 1 upward slope with little retardation. These tests indicate that extent and shape of excavation beyond the end of the bucket influence required bucket dimensions.

From the design curves, a 30-foot wide bucket would require a radius of 75 feet and depth of 121.5 feet to operate satisfactorily for the maximum design flood of 25,000 second-feet. Studies showed the cost of any bucket design to be excessive so no further consideration was given this type of design.

The horizontal hydraulic jump type of stilling basin was next studied. No model studies were made of this type of basin, since reliable data from previous studies of other structures could be utilized in determining rather closely the required principal dimensions for the Green Mountain conditions. The required sizes of rectangular basins for 30-, 50-, and 90-foot widths are shown in figure 5. Again, the proper basin width must be determined by a cost analysis. Although the cost of the horizontal basin would be considerably less than that of the roller-bucket basin, it was also considered excessive.

Comparison of the two types of stilling basins leads to the conclusion that the use of each type of basin depends mainly on the tail-water conditions prevailing at the dam site. The depth of water required in the roller bucket for satisfactory operation is considerably more than that required to form a good jump in the horizontal basin. When the height of tail water above the river bed is much in excess of the required jump depth, due to the drowning of the hydraulic jump in the horizontal basin, better flow conditions will exist in the roller-bucket basin, and in some

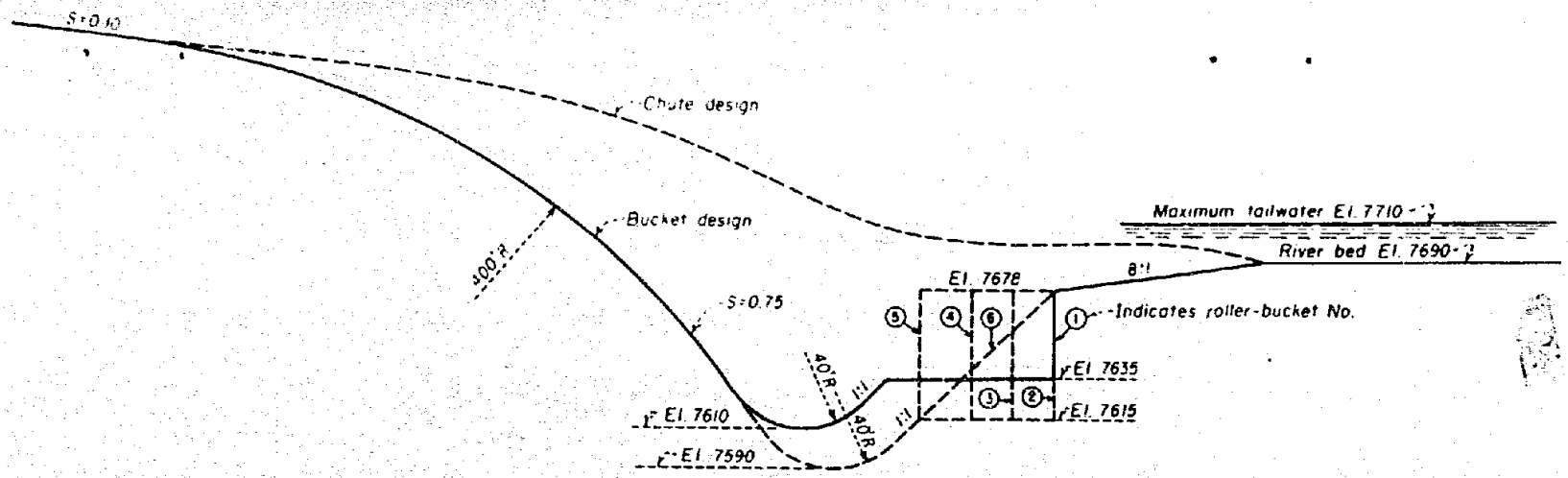

FIGURE 4 – ELEVATION OF ROLLER-BUCKET DESIGNS STUDIED

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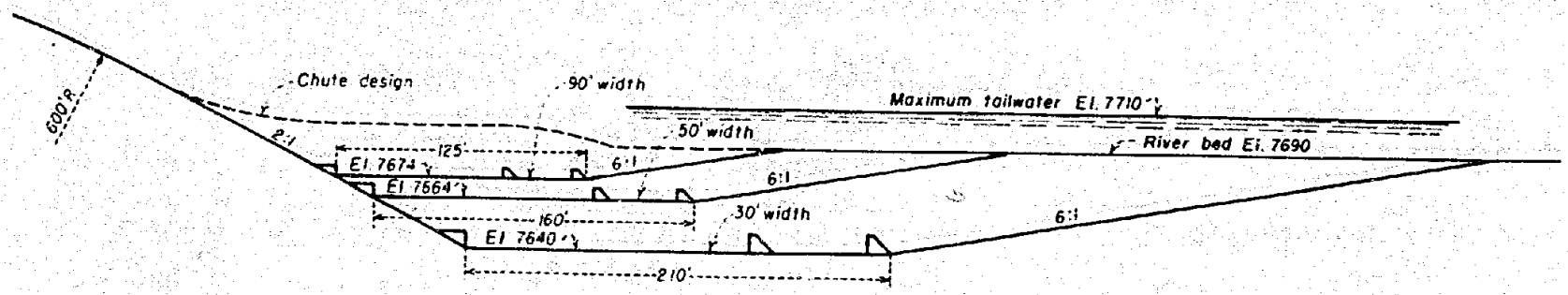

FIGURE 5 – ELEVATION OF HORIZONTAL STILLING BASINS

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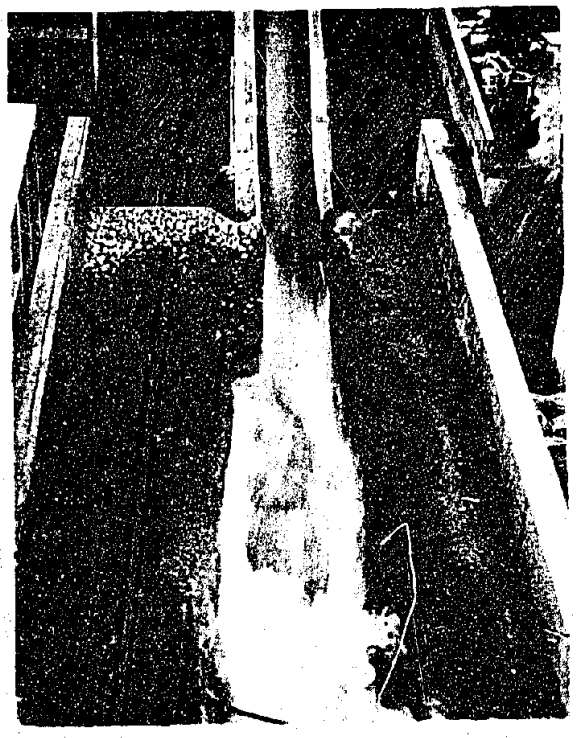
cases the cost will be less. When the height of tail water above the river bed is less than the required jump depth, the horizontal stilling basin should be used, since the cost in all cases will be less.

The hydraulic model study incorporated with the design of the Green Mountain Dam spillway was made in the hydraulic laboratory of the Bureau of Reclamation, Denver, Colorado, under the immediate supervision of the writer and general direction of J. E. Warnock, Research Engineer. All laboratories are under the supervision of Arthur Ruettgers, Senior Engineer, and R. F. Blanks, Engineer. All design work is under the general supervision of J. L. Savage, Chief Designing Engineer. All engineering work is under the direction of R. F. Walter, Chief Engineer; and all activities of the Bureau of Reclamation are under the direction of John C. Page, Commissioner.

Maximum discharge in 200-feet radius inlet.

Maximum discharge in final chute design.



7,500 second-feet discharging into river.