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THE STABILITY OF A GABION STRAIGHT DROP STRUCTURE

THE UNIVERSITY OF ARIZONA M.S. 1984

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THE STABILITY OF A GABION STRAIGHT DROP STRUCTURE

by

Brian E. Ehlers

A Thesis Submitted to the Faculty of the

DEPARTMENT OF CIVIL ENGINEERING AND ENGINEERING MECHANICS

In Partial Fulfillment of the Requirements
For the Degree of

MASTER OF SCIENCE
WITH A MAJOR IN CIVIL ENGINEERING

In the Graduate College

THE UNIVERSITY OF ARIZONA

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Date

ACKNOWLEDGMENTS

In the undertaking of this study and ultimately leading to the preparation of this manuscript, there are many people I wish to thank. First, in helping me become familiar with the flume and apurtenances and their help in fixing parts, I would like to thank Bill Lichtenwalter and Lorenzo Luhan. For his support, guidance, and direction I wish to thank Dr. Laursen. Due to him I had the opportunity to use the newly constructed flume on the University of Arizona campus and had ample time to pursue my studies. Lastly, I wish to thank my parents for their love, support and help that they have given me over the years and especially this past summer while I was organizing and writing this paper.

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ABSTRACT

When designing a gabion straight drop structure it is of interest to the designer to know under what situations the structure will operate adequately. In this study, flow profiles were obtained for the straight drop structure, and design procedures are presented. It was found that the length of the apron and tailwater depths were the controlling factors that determined the operating characteristics of the structure. If the apron is less than 4 y_2 in total length, sediment will scour out at the end of the apron, and the structure will fail. This scour is not due to inadequate tailwater, for scour was observed at tailwater depths much higher than design values. If the apron length is adequate, the tailwater will dictate the flow profile over the drop. For a hydraulic jump to occur, it was found that there was a range over which the tailwater could vary. Scour was found to be insignificant under this flow condition, however, when sweepout occurred the basin failed.

CHAPTER 1

INTRODUCTION

It has been common practice in Arizona to build straight drop structures on rivers and washes within the state to protect bridges and culverts from undermining due to headcutting of the river. The most common material used for building drop structures is concrete. However, the practice of building drop structures out of gabions, or with piling, wire fabric and rock, has been used extensively in the Southwest.

Gabions are containers of various shapes, i.e., rectangular, cylindrical, square, etc., made of wire mesh fabric that are filled with rocks. Many of these containers wired together make a gabion structure. This type of structure is easy to build and is fairly inexpensive if the rocks can be obtained at the site. This form of protection has the added advantage that differential settling will not cause major problems because the gabions will conform to irregular ground surfaces. The first such structure in Arizona was built in 1932 and is still in operation today.

Intuitively, one would think that the gabion drop would be a better economic choice than concrete. This is generally the case in spite of the high maintenance cost of gabions. Replacement of torn fabric, corrosion or abrasion of the wire, and movement of the rocks are some of the problems that can be encountered.

All gabion drop structures are fundamentally the same but each has its different peculiarities. A typical structure consists of 3' x 2' gabion containers filled with six-inch rock placed on top of a filter fabric. In some of the older structures, gravel was used instead of filter fabric as a base material. The gabion containers are tied together with heavy wire. Railroad rails are driven into the ground at the face of the drop to hold the gabions vertical.

Since gabions are elastic structures which require maintenance, there have been some modifications made to some structures. Wire mesh rusts and erodes after a number of years and concrete grouting has been used successfully to keep the rocks in place. However, the single biggest problem encountered is excessive scour downstream of the drop. One example is Flat Top wash where Interstate 40 crosses the wash south of Kingman. In 1981, the Arizona Department of Transportation (ADOT) constructed a gabion drop structure on the downstream side of the Interstate 40 bridge to protect the bridge from undermining. This structure consisted of three, 2.5-foot drops placed consecutively together. One year after the drop was put in, there was excessive scour downstream of the drop and it was endangered. To prevent failure of the gabions, another structure was added to the first. This structure resembled the first except that one of the three drops was 7.5 feet in height. It appears that this has solved the problem. Whether the scour downstream of the drop was due to inadequate design of the first drop or further headcutting of the wash is still debatable. A combination of the two factors was probably the case. Regardless of the cause, this is an example of how rapidly a drop can become endangered.

This example also brings up very important questions that designers should ask themselves. How many structures of this type are there? Are they in good repair? How many have failed? What would be the loss if even one failed? The State of Arizona has undertaken a study to evaluate the drops and to prepare cost estimates for repairing them. The repair costs are expected to be in the millions of dollars. The loss of a bridge could have significant impacts on the transportation facilities of this state as well as impose a great financial drain on the budget of the state, to say nothing of public liability in case of injury or death of those using the bridge.

Because more gabion structures are being installed and many of them are being rebuilt, it is of interest to analyze and determine the flow characteristics over these structures. This may help us to understand flow patterns and sediment movement more thoroughly and also to provide designers a better approach to solving problems associated with their use.

Study Objectives

It is the objective of this study to analyze the straight drop spillway and determine under what circumstances scour and failure will occur. Also of interest is the correlation between scour and the varying forms of the hydraulic jump.

To evaluate the drop, it is necessary to use a model. Energy and momentum equations aren't adequate to clearly define water and sediment movement. However, it is hoped that a mathematical expression encompassing these equations can be obtained which will describe or approximate the experimental results. Of particular interest is the flow profile when approach flow is at or near critical flow. If this condition is assumed, it will simplify the analysis somewhat. It is generally the case in rivers flowing in Arizona. Also of interest are the varying water surface profiles downstream of the drop and the required tailwater heights associated with them. For varying tailwater heights, scour patterns at the end of the basin will change. Documentation of the patterns, as well as depths, is of great importance. The ultimate objective is to experimentally determine how and why a gabion straight drop spillway will fail and to give designers a procedure so that they will be able to design a safe structure with full knowledge of the conditions under which it will safely operate.

CHAPTER 2

THEORY

The use of a straight drop structure is twofold. One, it is used to stop headcutting in a stream. Two, it is used to dissipate the kinetic energy of the flowing water so as to prevent local scour downstream of the structure. In the analysis of the straight drop structure a vertical drop and horizontal apron will be evaluated. Since the apron is constructed of gabions it will be considered flat with no baffle blocks. Therefore, the dissipation of kinetic energy will be solely due to the hydraulic jump that forms on the apron,. There will be a loss of energy as the water drops over the vertical drop, but this is thought to be small compared to that lost in the hydraulic jump.

Previous Works

Basic information concerning the characteristics of the hydraulic jump, on a level surface, is published in many books on hydraulics and fluid mechanics. The hydraulic jump was first investigated experimentally by Bidone, an Italian, in 1818. Thereafter, many studies were made and the results were quoted by many writers. Significant contributions were made by Darcy; Ferriday and Merriman; Gibson; Kennison; Lindquist; Einwachter; Bakhmetoff and Matzke; Escande;

Pitrini; Nibbia; Kindsvater; Blaisdell; Moore and Morgan; Rouse, Sina and Nagaratum; and many others.

As Moore and Morgan (1957) report,

> Rouse, Bhoota, and Hsu investigated the stabilizing characteristics of an abrupt drop in a channel bottom. They found that two different types of flow may occur at a given drop, the transition from one to the other being characterized by an undular wave. The type of flow which will form is dependent upon whether the downstream depth is above or below that which produces the undular standing wave. The conditions which produce the undular stage cannot be foretold by equations utilizing only the momentum and continuity relationships, but must be obtained by experimental measurements. Rouse, Bhoota and Hsu presented such data which verified their analysis and indicated a systematic transition between the two regimes of flow depending on the Froude number. To simplify the analyses for the two types of jumps, the pressure on the face of the drop was assumed to be hydrostatically distributed and assumed to be dependent on the downstream depth if the jump is upstream from the drop and dependent on the upstream depth if the jump is downstream from the drop.

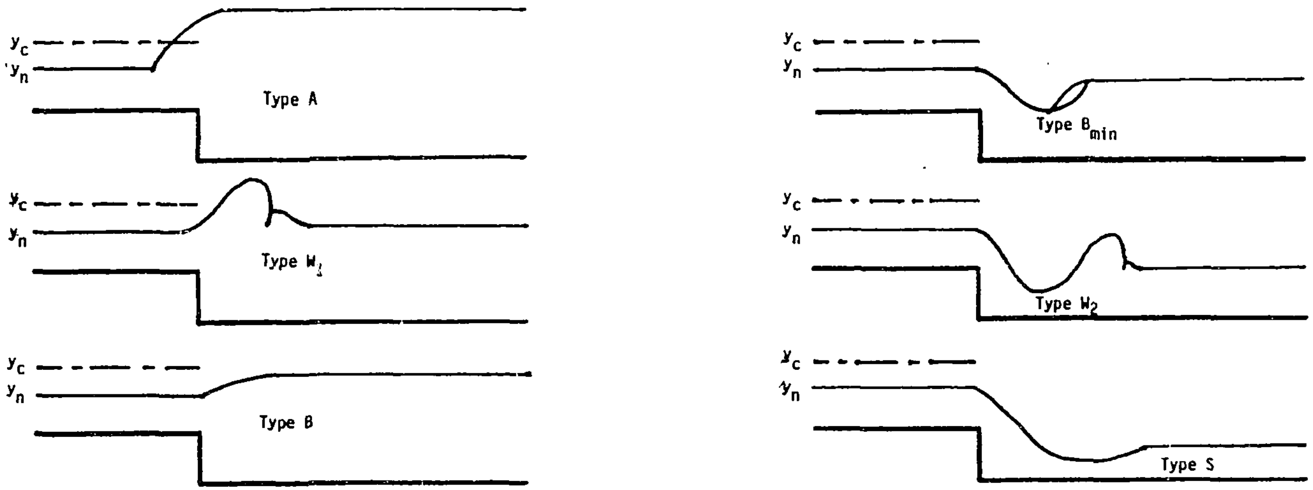
Analysis

The different flow profiles which will occur at an abrupt drop in a rectangular channel are shown in Figure 1. The subcritical and critical profiles are identical because the abrupt drop forces the flow to critical depth at the crest. There is, however, a wide variety of profiles when the approach flow is supercritical. Of these, A, W_1, B, B_{min}, and W_2 are forms of the hydraulic jump. S is the surface profile for sweepout. If the downstream channel is steep, no jump will form.

In Figure 2, the geometry of the drop is labelled. The symbols shown will be referred to throughout the remainder of this report.

Subcritical Approach Flow

Critical Approach Flow



Supercritical Approach Flow

Figure 1. Flow Profiles

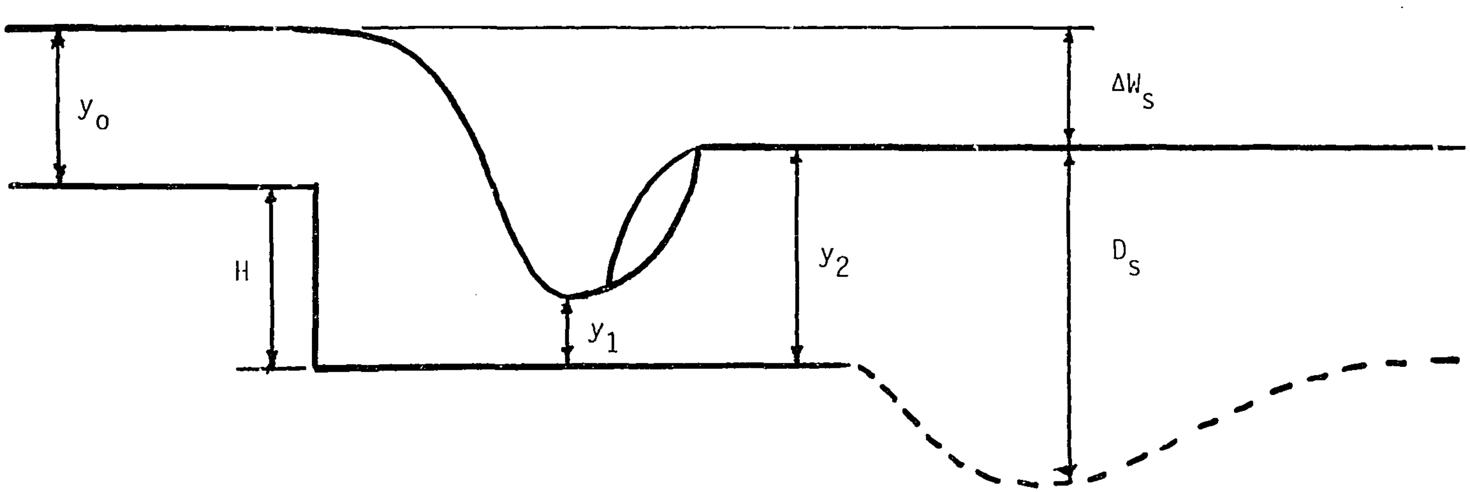


Figure 2. Drop Structure Geometry

Hydraulics

The relative height of a stationary jump y_2/y_1 will be dependent upon the dimensionless parameters H/y_0 and the Froude number $V_0/\sqrt{gy_0}$ of the entering flow. For the condition where the piezometric head at the drop equals $y_0 + H$ and the force due to boundary shear is neglected, the momentum equation is:

$$pV_0^2 y_0 + \frac{\gamma}{2}(H + y_0)^2 - \frac{\rho V_0^2 y_0^2}{y_2} + \frac{\gamma}{2} y_2^2 \qquad (2.1)$$

This reduces to:

$$\mathbb{F}_0^{\,2} = \frac{\left[\frac{y_2}{y_0}\right]^2 - \left[1 + \frac{H}{y_0}\right]^2}{2\left[1 - \frac{y_0}{y_2}\right]} \qquad (2.2)$$

A graph of the values obtained using this equation can be seen in Figure 3.

Equation 2.2 assumes a pressure distribution on the face of the drop that is equal to $y_0 + H$. In reality, this may or may not be the case. From experimental analysis [10], it can be seen that the theoretical approximation falls close to the middle of the hydraulic jump range and is therefore a valid approximation for approach flows that are highly supercritical. However, for approach flows near

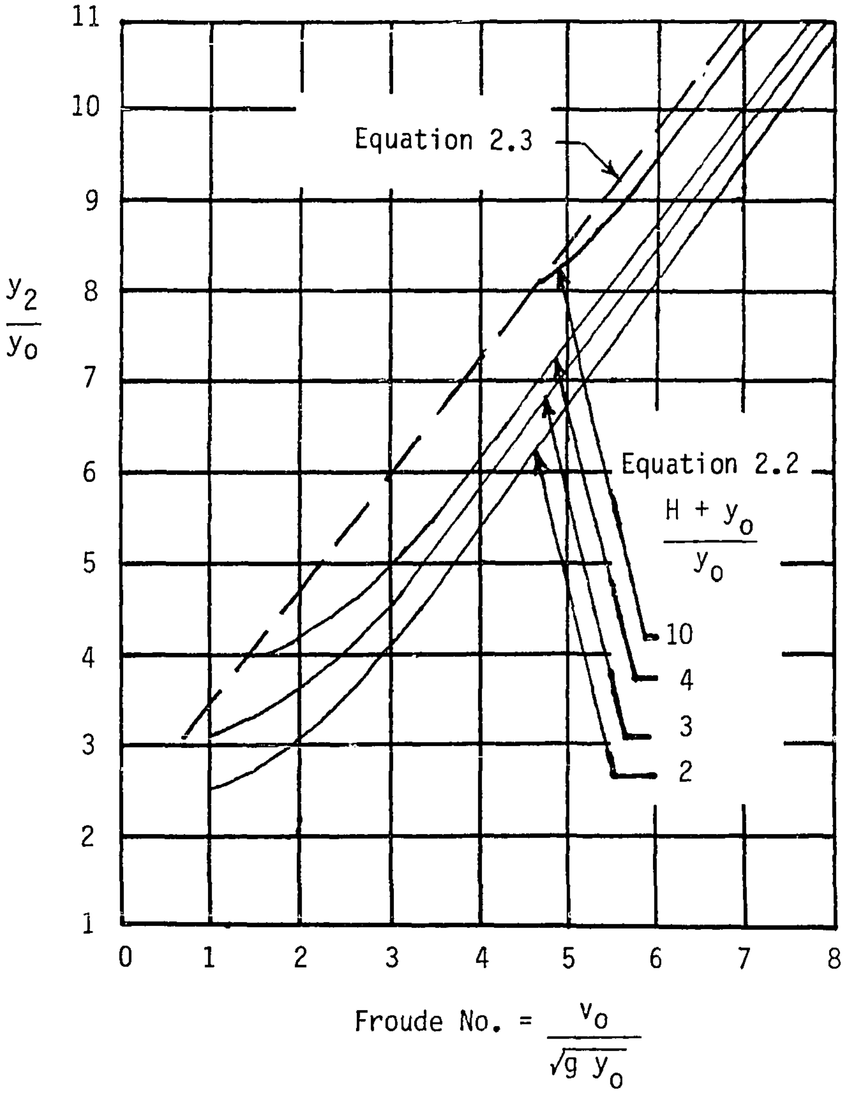


Figure 3. Relative Downstream Depth as a Function of Froude No. and Piezometric Head at the Drop (after Moore and Morgan, 1958)

critical the approximation is questionable because the theoretical values don't fall in the middle of the jump range.

Assuming the piezometric head equal to y_2 and substitution into equation 2.1 the following results:

$$\mathbb{F}_0^2 = \frac{\left[\frac{y_2}{y_0} - \frac{H}{y_0}\right]^2 - 1}{2\left[1 - \frac{y_0}{y_2}\right]} \tag{2.3}$$

This gives the upper limiting curve in Figure 3.

After experimental analysis, Moore and Morgan developed a set of graphs which delineated a range of tailwaters for the different forms of the jump. These are presented in Figures 4, 5, and 6.

To evaluate all the flow profiles and the scour patterns associated with supercritical and subcritical flow would indeed be a major undertaking. In fact, time will not permit an examination of even a major portion of all the possibilities that could exist with flow, height of drop, scour patterns, and variable tailwater in the supercritical range. Nor can it be justified, because in Arizona these drop structures are being built to prevent headcutting which is associated with lower tailwater depths. The main jump characteristics of interest are water surface profiles for critical flow and jump types B and W_2 in supercritical flow. This corresponds to the lower left-hand corner of Figures 4, 5, and 6.

For critical flows, it has been shown that the flow profile over a straight drop can be associated with what is known as a drop

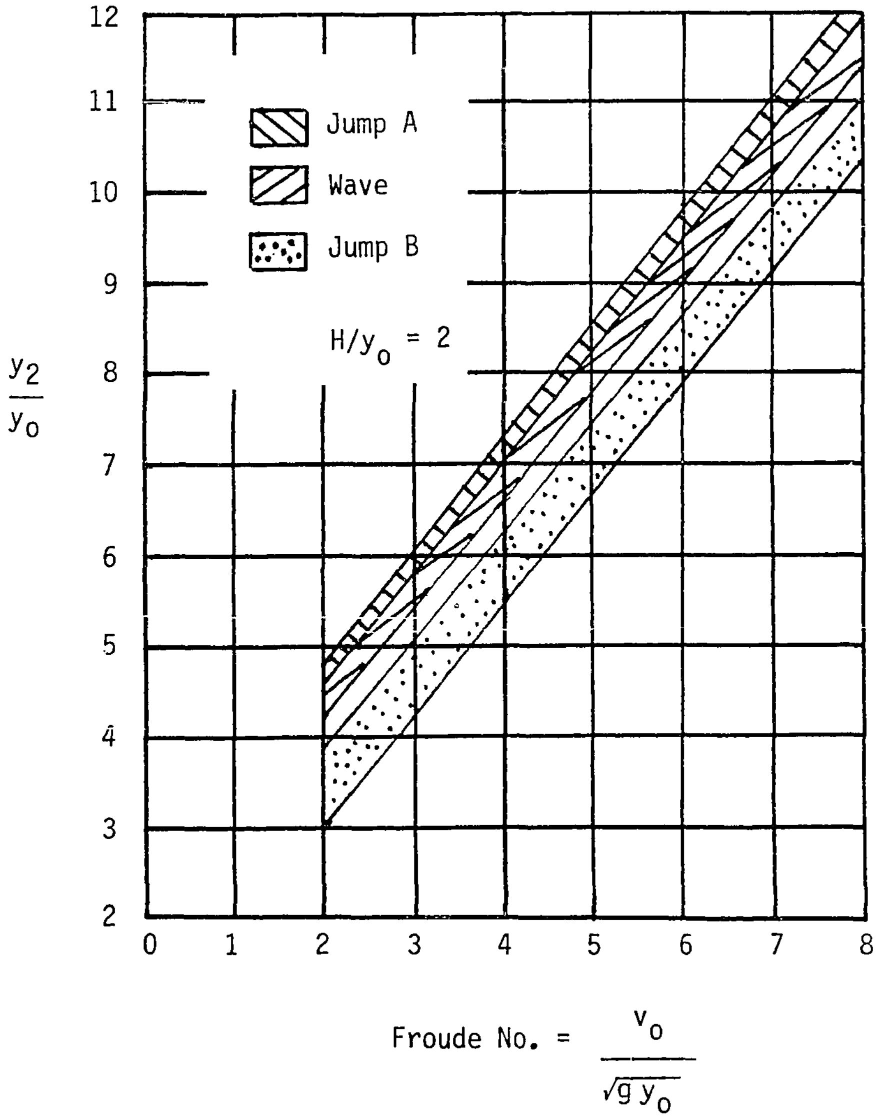


Figure 4. Forms of the Hydraulic Jump as a Function of Froude No. and Relative Downstream Depth (after Moore and Morgan, 1958)

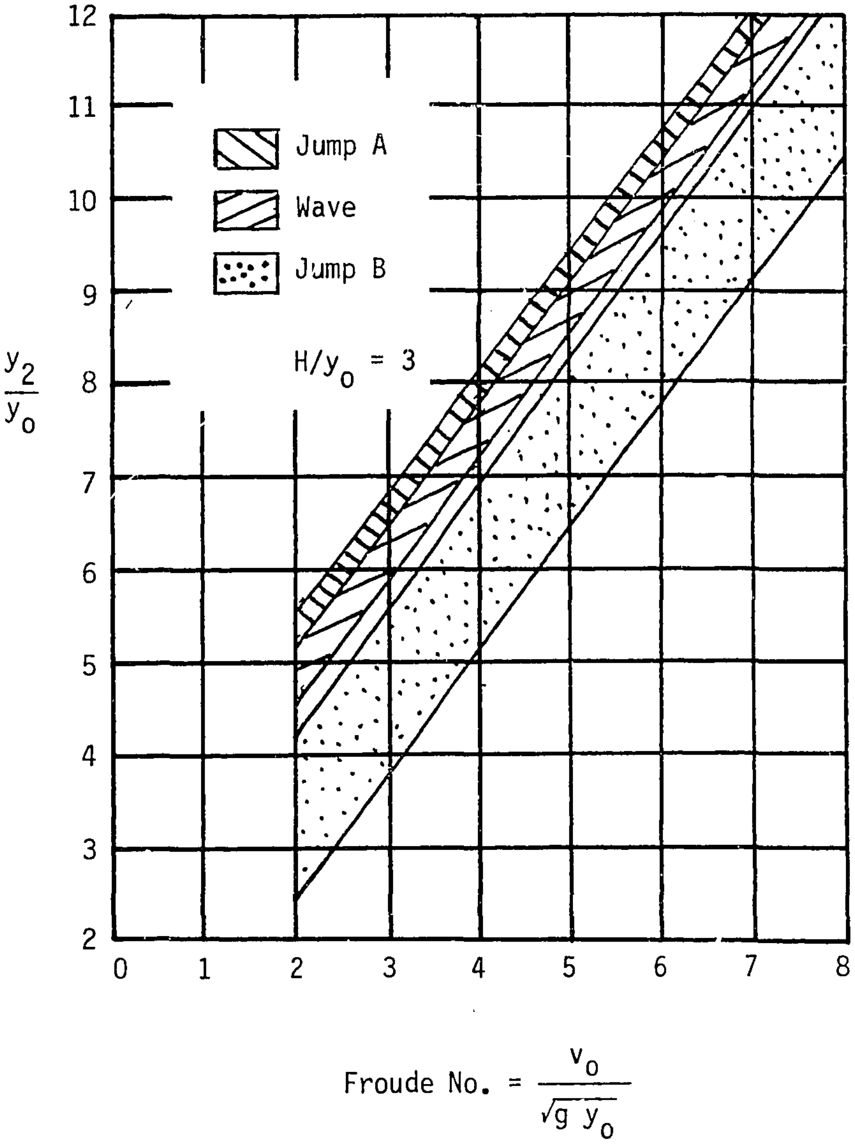


Figure 5. Forms of the Hydraulic Jump as a Function of Froude No. and Relative Downstream Depth (after Moore and Morgan, 1958)

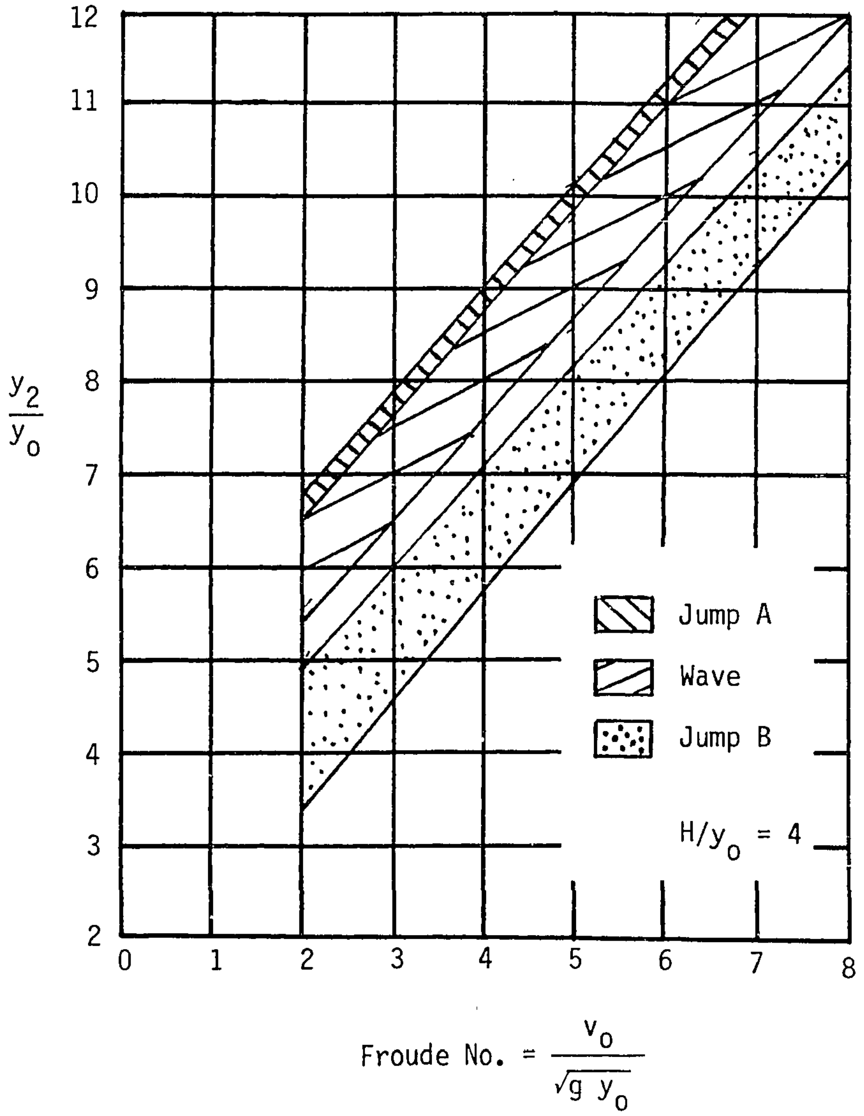


Figure 6. Forms of the Hydraulic Jump as a Function of Froude No. and Relative Downstream Depth (after Moore and Morgan, 1958)

number (D_n) (Bakhmetoff and Feodoroff, 1943; Rand, 1955). This is defined by the equation:

$$D_n = \frac{q^2}{gH^3}$$

where q is the discharge per unit width of the crest overfall, g is the acceleration due to gravity, and H is the height of the drop. For a given H and discharge, a drop number can be determined. It is known that for critical flows at the drop (i.e. subcritical flows in rivers), this relationship holds but it is not yet understood if this relationship is valid for supercritical flows.

CHAPTER 3

EXPERIMENTAL PROCEDURE

Experimental observations were made on various drops constructed in a 30 foot long by 2 foot wide flume located in the hydraulics laboratory of the Department of Civil Engineering and Engineering Mechanics. The flume is pictured in Figure 7 and shown schematically in Figure 8. The drop heights ranged from 0.2 to 0.8 feet and the length of the spillway apron ranged from 3 to 6 times the critical depth. Water was supplied from a constant head tank. An 8 inch gate valve was used to control the flow rate. The flow was measured by a triangular weir for which the equation for discharge is $Q = 2.5\ H^{2.5}$. The smallest increment on the point gauge, which was used to read the head over the weir, is ±0.001 feet.

A gate at the end of the flume was lowered and raised to control tailwater depths. Since the drop was constructed in the middle of the flume (end to end), it was high in relation to the gate that controls the tailwater (Figure 8). This being the case, all runs were started with high tailwater and the gate was lowered to lower the tailwater.

The experiment was set up so that there was a long approach to the drop. This was done by using half-inch plywood cut to the width of the flume and made approximately 5 feet long. The plywood was

Figure 7. Flume for Study

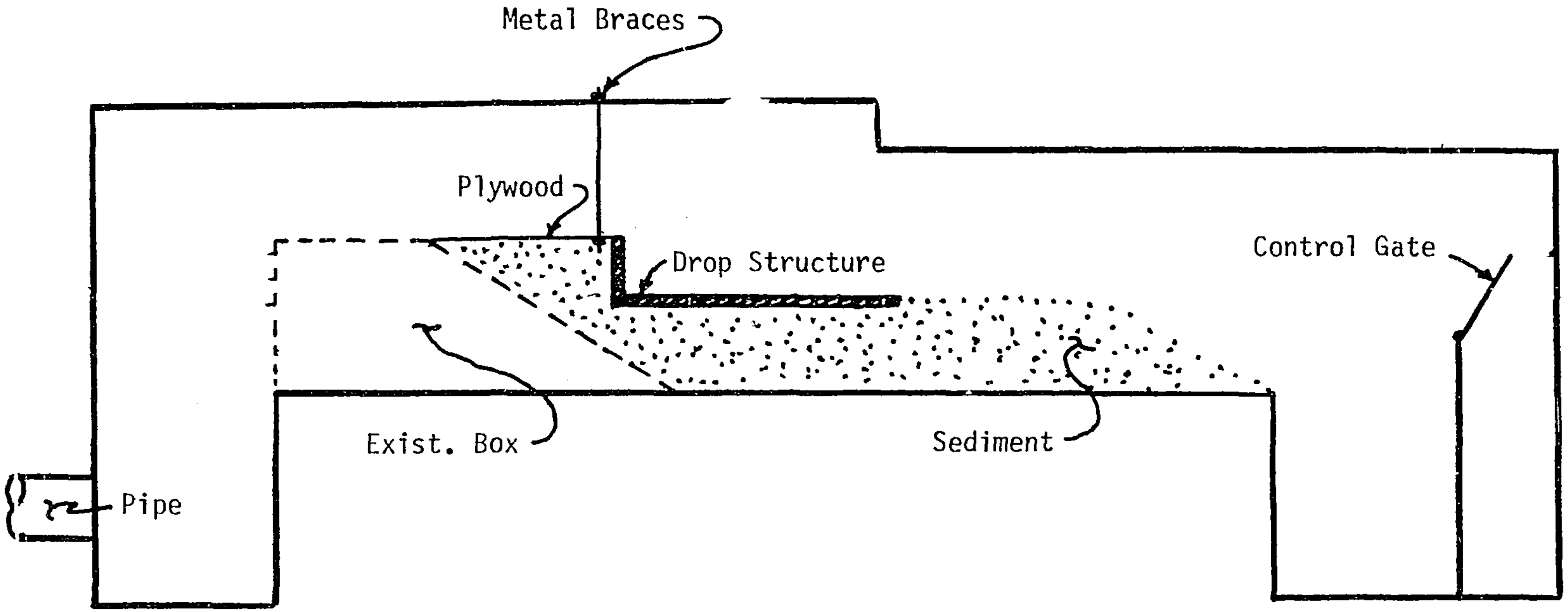


Figure 8. Experimental Set Up

attached to a box which was already in place at the entrance to the existing flume. The plywood approach section was exactly level. The height was maintained by attaching two metal bars to the plywood, then connecting these bars to a metal brace on the top of the flume. To prevent seepage between the edge of the plywood and the wall of the flume, a sealer was applied. Also, it was found that sediment placed beneath the plywood would support the plywood and flowing water, thus allowing a more uniform flow and reducing the tendency of increased flow depth in the center of the flume due to the hydraulic pressure of water over unsupported wood.

In initial trials, the gabions were made of 1/4 inch rebar welded in the shape of the drop, covered with the wire fabric, and filled with rock. This drop was used for several trials before it was found that the inelasticity of the structure was so great that it would not allow for failure because the entire structure was connected together too rigidly. The drops and spillway apron were fabricated of 1/8 inch welded wire fabric. In the next trials the drops and spillway apron were fabricated of 1%8 inch welded wire fabric. A single gabion drop and a single gabion apron, each made of one piece of fabric, were used. This worked well. The gabions stayed in place and better modelled a gabion structure. For the runs discussed in this thesis, the drop and spillway apron were made of gabion boxes each 0.3 foot in length by 0.2 foot in height and extending across the flume.

To model the filter fabric placed under rock gabions in the field, several substances were tried. Nylon stockings, filter paper, and fiberglass tape were used in several early trials. The filter paper was found to be more to the scale of the model and to more accurately reflect field conditions.

The vertical drop was 0.2 foot thick in the direction of flow and since it was to stand vertically, wire ties were used to tie the baskets internally to keep the structure from bulging in the center. The vertical drop structure was put in place at the same time the plywood was installed to obtain a good fit between the different materials. It also remained in place throughout the entire experiment.

Plans of several gabion installations received from the Arizona Department of Transportation showed that the depth of flow and height of drop were approximately the same at their installations. This being the case, the first tests made in this study used that geometry and flow. However, different length aprons as well as different geometries were evaluated. The flow conditions listed in Table 1 were selected for the first observations. $\mathbb{F}$ is defined as the Froude number of the approach flow and D_n as the drop number. Data from all runs can be seen in Appendix A.

Three different sediment gradations were used in each of the above cases, as shown in Figure 9. The median diameters are 0.33 mm, 0.66 mm, and 6.1 mm (0.012, 0.026, 0.24 inches).

TABLE 1

GEOMETRIES OF DROP STRUCTURES

y_c (ft)	H (ft)	H/y_c	Type of Jump	$\mathbb{F}^{(a)}$	$D_n^{(b)}$
.2	.2	1	W_2	1.27	.997
.3	.3	1	B, W_2	1.17	.89
.4	.4	1	B, W_2	1.16	.99
.2	.4	2	B, W_2	1.27	.12
.3	.6	2	submr., B	1.17	.11
.4	.8	2	submr., B	1.16	.11
.3	.4	1.3	B, W_2	2.94	.38

(a) $\mathbb{F} = V_0/\sqrt{gy_0}$

(b) $D_n = q^2/gH^3$

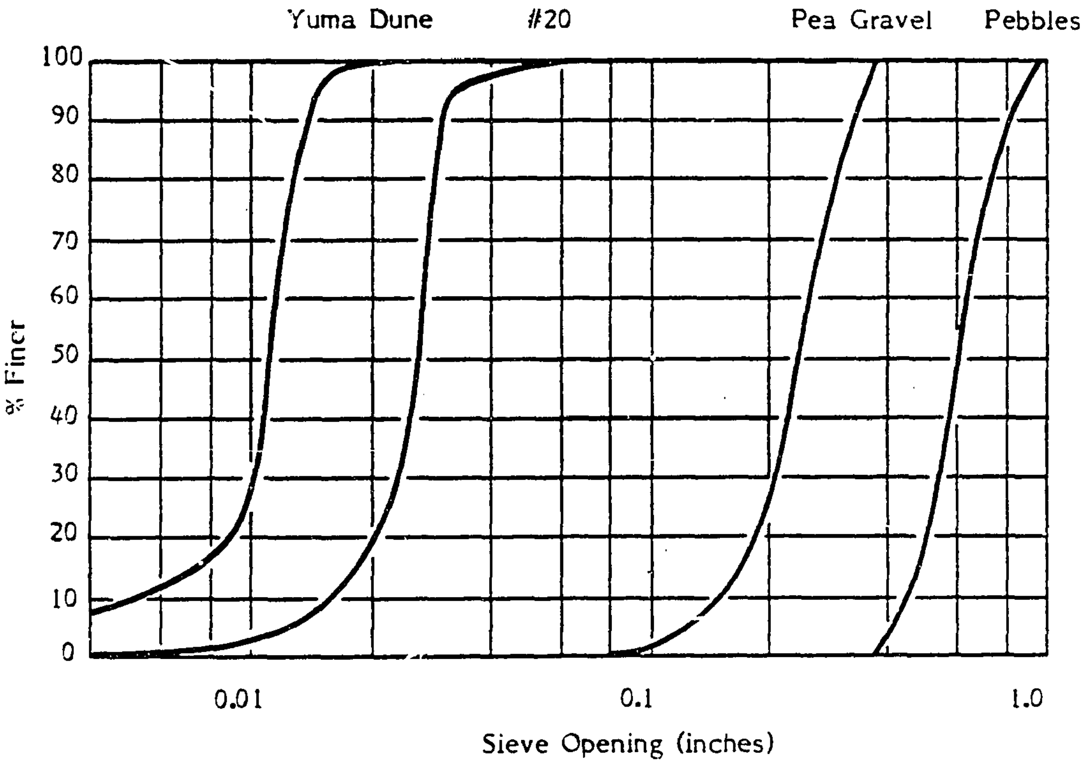


Figure 9. Size Distributions of Sands and Gravels
(after Laursen and Flick, 1983)

CHAPTER 4

DISCUSSION

There are two distinct parts to the evaluation of the data obtained in this experiment, one being the hydraulic or flow patterns over the straight drop spillway, the second being the scour patterns downstream of the apron. The hydraulic results and design procedures will be presented after a discussion of the flow patterns, and then the scour results will be evaluated with regard to the hydraulic performance of the drop.

Experimental Results

In evaluating the tests run on the straight drop spillway, it was found that there were three distinct flow profiles over the spillway: the hydraulic jump, wave 2, and sweepout. In all cases it was found that the tailwater depth was the controlling factor that determined which profile occurred. This can cause problems in the design of the basin because the tailwater is the single most difficult value to determine because it depends on the flow, bed degradation, slope of channel, etc. However, it was also found from experimental analysis that there is a range of tailwater depths over which each water surface profile occurs. This gives the designer some leeway in design and provides a safety margin if the river changes its flow characteristics.

The first and most important flow profile is that of the hydraulic jump. The hydraulic jump is a water surface profile where high velocity flow is transformed to low velocity flow. This is done by a rapid change of depth of flow. A direct advantage of designing for this surface profile is that it is the shortest of the three profiles, and the dissipation of kinetic energy of flow and its transformation to head is the greatest. An example of this profile is shown in Figure 10. The objective of all designers is to design structures so that this flow profile will occur. It represents the smallest structure because of the short profile; therefore, it is the most economical in terms of materials. A specialized type of hydraulic jump is the submerged jump, shown in Figure 11. In this case the tailwater depth is significantly higher than that needed for a true hydraulic jump to occur; generally, this is when the tailwater is greater than $2y_c + H/2$. The jet plunges into the water and is dispersed rather than hitting the bottom of the apron and being deflected as in the case of the hydraulic jump. The submerged jet is longer than the hydraulic jump.

The second profile is that of the wave 2 profile,shown in Figure 12. In this case, the tailwater is lower than that required for the hydraulic jump. The flow profile is such that the high velocity incoming jet will rise to a depth equal to $H + y_c$ which is approximately the same height as that of the incoming flow over the drop. The wave will not be contained in the basin but will cross the end of the apron and continue down the streambed. If the flow is from left to right, as in the case in Figure 12, then the wave causes a clockwise

Figure 10. Hydraulic Jump Water Surface Profile

Figure 11. Submerged Jump Water Surface Profile

Figure 12. Wave 2 Water Surface Profile

circular motion over the end of the basin and the sediment movement is back towards the basin so no degradation of the structure is observed.

The last profile of importance is sweepout which occurs when the tailwater becomes much less than that required for the wave 2 profile. In this case, no jump occurs, and the water surface is extremely flat. At the end of the basin the jet moves up and down until it eventually plunges into the sediment instead of riding a roller at the end of the basin. Figure 13 shows an example of this profile.

In all these cases the apron is sufficiently long so that scour and failure due to length of apron is not a problem.

Hydraulic Design

From the test data summarized in Tables 2 and 3, the design chart in Figure 14 was produced. As can be seen from this figure, the drop height and drop number determine the values of y_1, y_2, L_p and L_d, where y_1 is the depth of the high velocity jet on the spillway apron; y_2 is the tailwater depth; L_p is the jet length needed for a submerged jet; and L_d is the jet length needed for the hydraulic jump to form (Figure 15). The experimental values are shown on the figure and are representative for approach flows that are subcritical or supercritical. In addition to the runs already discussed, Appendix B lists other runs to verify the design chart presented as Figure 14.

Either L_d or L_p should be used in determining the length of the basin, depending on flow conditions. Figure 14 is to be used to design drops so that a hydraulic jump will occur. The overall length of the

Figure 13. Sweepout Water Surface Profile

TABLE 2

HYDRAULIC PARAMETERS

y_c (ft)	H (ft)	Tailwater Req'd for Hydraulic Jump (ft)		y_1 Max. (ft)	y_1 Min. (ft)	Design y_1 (ft)	Computed y_2 (ft)	$\Delta W_s/y_c$		L_d (ft)		L_p (ft)
		High	Low					High	Low	High	Low	
.2	.2	prejump	prejump	.20	.13	.13	.29	--	--	.50	.50	--
.3	.3	.50	.40	.30	.16	.18	.44	.3	1.0	.62	.65	--
.4	.4	.80	.50	.40	.23	.25	.60	.5	1.0	.85	1.00	--
.2	.4	.45	.30	.25	.10	.10	.35	.7	∞	.55	.55	--
.3	.6	.75	.45	Submerg. @ $\approx 2.25\ y_c$	.115	.13	.55	.7	∞	--	--	.80
.4	.8	1.10	.60	Submerg. @ $\approx 2.25\ y_c$	.13	.18	.81	.7	∞	--	--	1.10
.3	.4	.65	.45	.25	.15	.176	.50			.82	.82	--

TABLE 3

DESIGN CRITERIA--DEPTHS VERSUS DROP HEIGHT

y_c (ft)	H (ft)	D_n	$\frac{y_1}{H}$	$\frac{y_2}{H}$	$\frac{\text{Min. T.W.}}{H}$	$\frac{L_D}{H}$	Apron Length	
							4 y_2	$L_d + H + 2y_c$
0.2	0.2	0.997	0.63	1.43	1.33	2.5	1.16	1.1
0.3	0.3	0.89	0.60	1.46	1.33	2.2	1.76	1.55
0.4	0.4	0.99	0.625	1.50	1.25	2.5	2.40	2.20
0.2	0.4	0.12	0.25	0.88	0.75	1.38	1.40	1.35
0.3	0.6	0.11	0.23	0.92	0.75	1.33	2.20	2.00
0.4	0.8	0.11	0.23	1.0	0.75	1.38	3.24	2.70
0.3	0.4	0.427	0.44	1.25	1.08	2.0	2.00	1.82

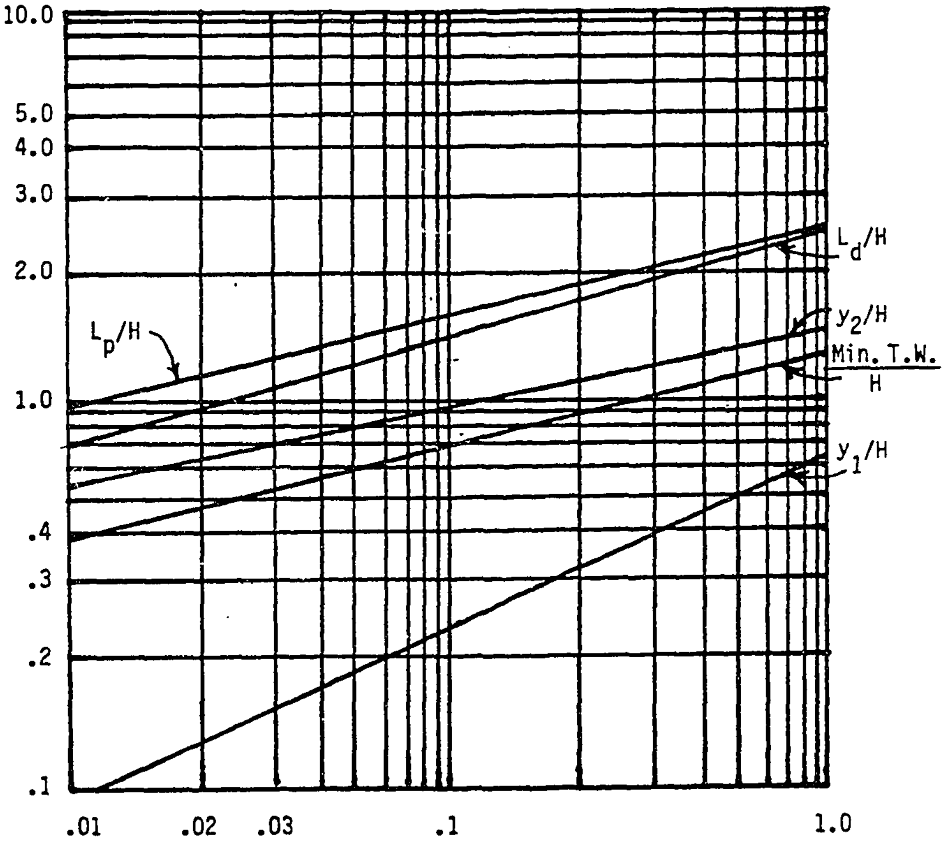
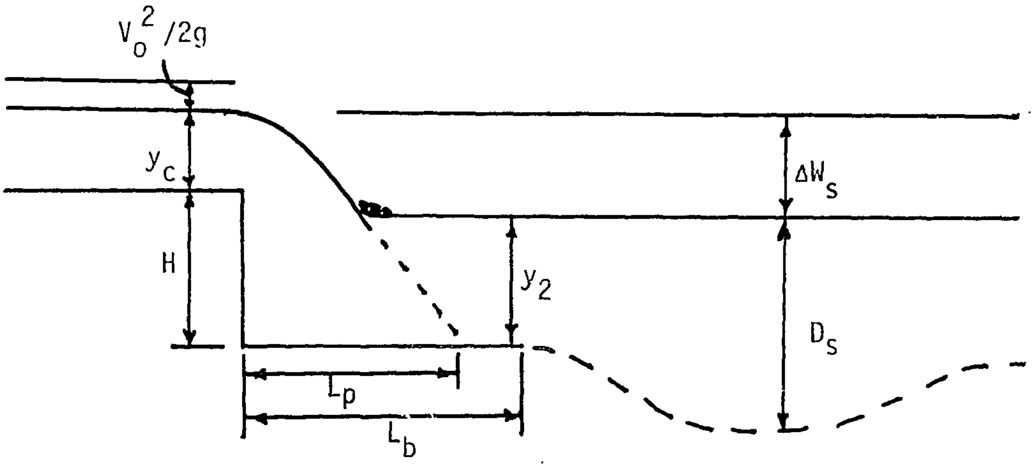


Figure 14. Design Values for Straight Drop Spillway

Submerged Jump



Hydraulic Jump

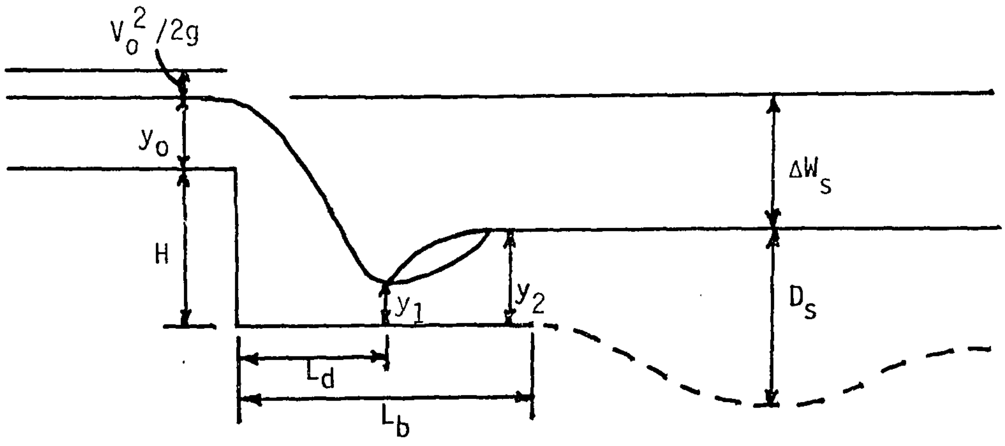


Figure 15. Different Forms of Hydraulic Jump

apron should be made equal to $4y_2$ or $(L_d + H + 2y_c)$, whichever is greater. During experimental runs $4y_2$ was found to be an excellent length to construct the apron of the structure. After evaluation of data, $L_d + H + 2y_c$ was also found to approximate the length required from experimental data and related the apron length to the geometry of the drop. If the spillway apron length is shorter than these specifications, excessive scour can be expected to occur downstream of the apron.

Tests indicated that if the length of the basin is not adequate to contain the jump, scour will result at the end of the apron, and the structure will fail. For all tailwater depths, the flow pattern moves sediment away from the toe of the basin and creates a scour hole. As the hole grows larger, more sediment moves out from under the apron until the gabion mattress drops and settles along the upstream side of the scour hole. It should be emphasized that this erosion pattern occurred even when tailwater depths were in the design range and was not the result of inadequate tailwater depths. Of course, failure will be dependent on a time parameter and the scour hole will fill in after the peak flow passes. It should be noted that the depth of scour didn't change appreciably for the three sediment gradations.

If the tailwater is such that the jump is submerged, then the length of the basin apron should be $(L_p + 2y_c + H)$.

To design the basin, the tailwater elevation has to be determined or assumed initially, and then the other parameters are determined. Also, for the design to work, degradation must be carefully estimated. For example, if the degradation is assumed to be 3 feet and actually is 5 feet, there will be a difference in tailwater of at least 2 feet. As stated earlier, the drop structure is very sensitive to tailwater, and a decrease in water surface of 2 feet in an ordinary-sized flow could change the water surface profile from a hydraulic jump to the sweepout profile and ultimately cause failure. Figures 16 and 17 are charts that show the hydraulic jump as determined from Moore and Morgan and the experimental data obtained from this thesis. Also, design values from Figure 14 are presented. From Figures 15 and 16 it can be seen that the design values obtained from Figure 14 lie in the center of the range of tailwater depths for the hydraulic jump range. Accordingly, if tailwater depths differ from the design value by more than 20% then the hydraulic jump will not form and another profile will occur. Twenty percent may seem like a great deal, but if a tailwater depth of 6 feet, which is on the high side in Arizona, is assumed, then a jump will form only between depths of 4.8 feet to 7.2 feet. If this information is used in the example above where degradation was 2 feet more than expected, it can be seen that, indeed, the hydraulic jump will not form and sweepout will occur. But, if one wanted to be on the safe side and set the basin so there was 7 feet of tailwater, then degradation could occur until a tailwater of 4.8 feet is reached (corresponding to 2.2 feet of degradation), and the structure would still operate. However, it must be emphasized that if deposition occurs and tailwater is in excess of 7.2 feet, a hydraulic jump will not form, and waves will dominate the water surface.

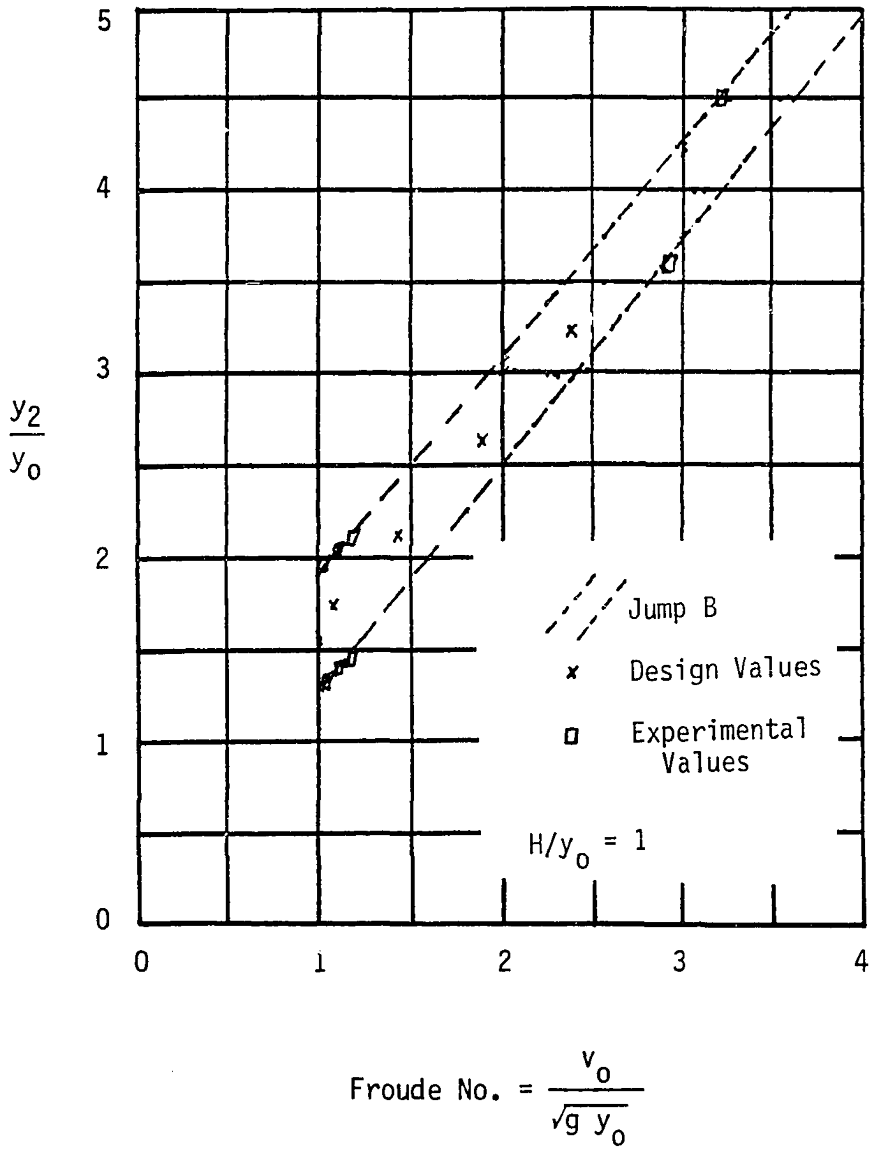


Figure 16. Tailwater for Hydraulic Jump as a Function of Froude No. and Relative Downstream Depth

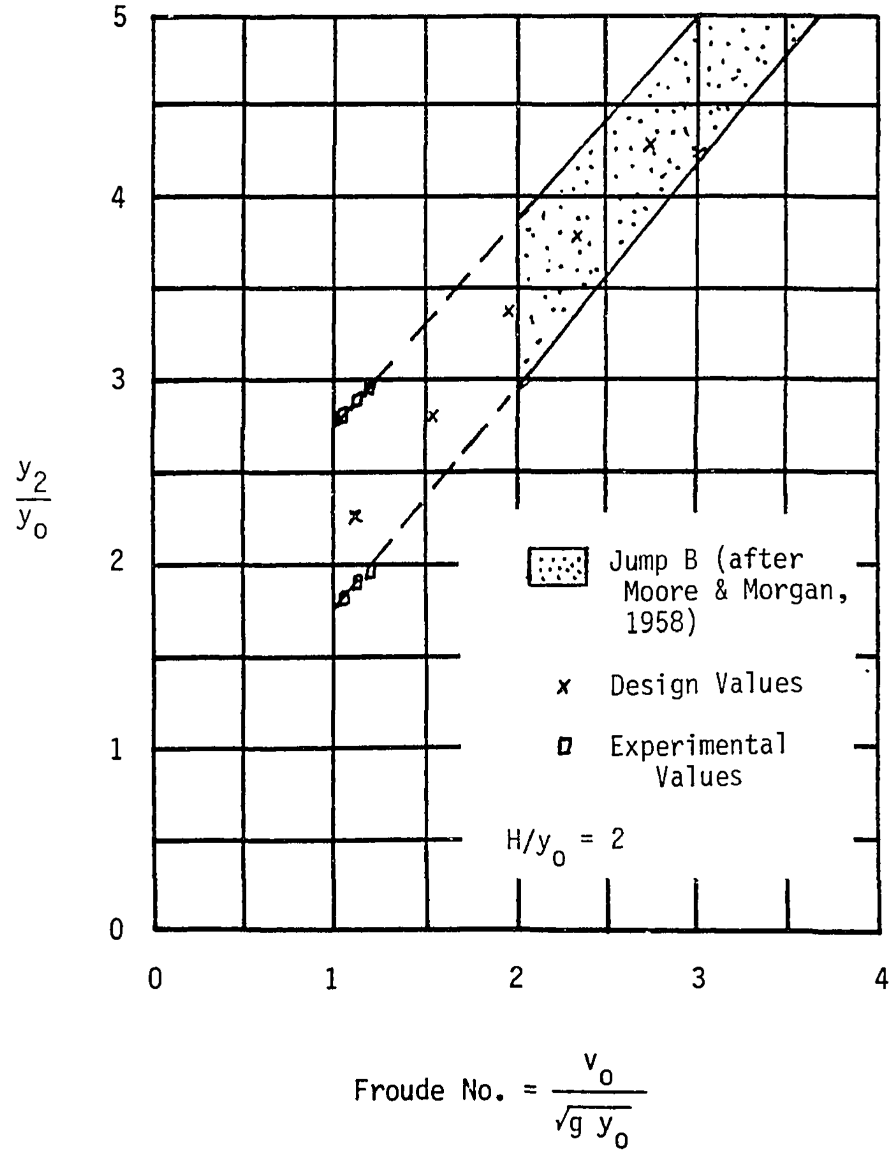


Figure 17. Tailwater for Hydraulic Jump as a Function of Froude No. and Relative Downstream Depth

If this were an actual case in the field, it would be very difficult by observation to estimate future degradation within 2 feet. However, when a flood occurs, such an error could surely result in flow conditions that might cause severe damage to the structure.

So, from analysis of the hydraulics of the straight drop spillway it can be shown that the design values obtained from Figure 14 are representative of flow characteristics of the drop, but that this structure is very sensitive to changes in tailwater. A 20% difference from design tailwater conditions will give inadequate conditions for a hydraulic jump to form and damage to the structure could occur.

Scour

The typical scour patterns found at the end of the spillway apron is shown in Figure 18. In most cases the depth of scour was not more than three times the drop height,and the equation $2y_c + H$ gives the best estimate for the value of D_s. Assuming that the tailwater is at least the critical depth, then the deepest that the scour hole measured from the bed would be is the drop height plus y_c. This is not extremely deep, especially in comparison with estimated scour values for the sloping sill or an unriprapped bed. Since most drops in Arizona are not over 5 feet, scour holes downstream of a drop structure with a horizontal apron would not be over about 8 to 10 feet. This is the case for all three sediments tested.

In any case, when the drop operated in the hydraulic jump range and the apron length was adequate, the direction of movement of the bed particles in the scour hole was back towards the apron on the

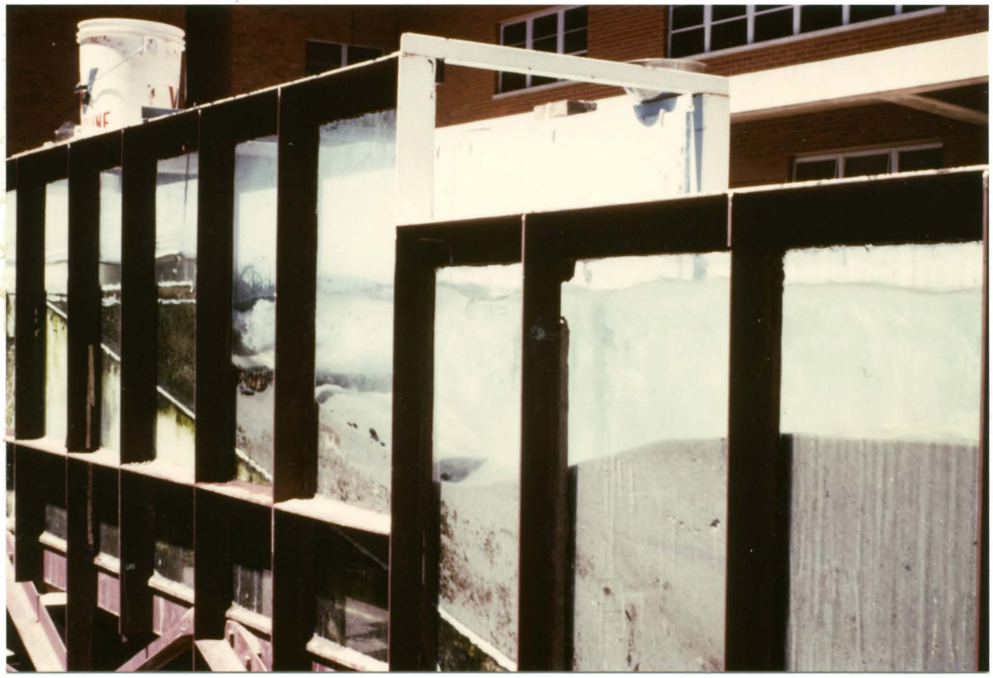


Figure 18. Scour Pattern at End of Spillway

upstream side of the hole and away from the apron on the downstream side of the hole. This type of bed load movement caused the apron to be stable over the range of flows encountered. It was not until sweepout occurred that bed materials were moved from under the spillway apron. The depths to the bottom of the scour hole varied with the size of the sediment used, but as was stated earlier, the greatest value was not more than $2y_c + H$. The depths of scour as determined from experiment are shown in Table 4.

When the tailwater was decreased past the hydraulic jump and W_2 ranges, the wave would disappear and the sweepout condition developed. Under sweepout conditions, the high velocity jet of water would shoot off the end of the apron sill and plunge into the bottom sediment. In this case, the scour pattern was difficult to determine. What generally happened was that sediment scoured at the end of the basin to a sufficient depth to allow a hydraulic jump to form in the sediment beyond the end of the basin. At the same time, sediment was removed from under the basin, causing the basin apron to drop. This was classified as failure of the basin, and the tests were not carried further. At failure, the tailwater was recorded. Based on analysis of the data, it appeared that failure was dependent on tailwater rather than on sediment size. Tailwater values at failure are shown in Table 5.

TABLE 4

SEDIMENT SCOUR

y_c	H	Depth of Scour (D_s) and Location from end of Apron		
		Gravel	Sand	Yuma Sand
0.2	0.2	0.5 @ 1.0'	0.75 @ 2.0'	0.80 @ 2.0'
0.3	0.3	0.75 @ 1.0'	0.90 @ 2.0'	0.95 @ 2.0'
0.4	0.4	1.0 @ 1.0'	1.15 @ 2.0'	1.2 @ 2.0'
0.2	0.4	0.7 @ 1.0'	1.0 @ 2.0'	1.15 @ 2.0'
0.3	0.6	1.0 @ 1.0'	1.2 @ 1.75'	1.3 @ 2.0'
0.4	0.8	1.4 @ 1.0'	1.55 @ 1.75'	1.65 @ 2.0'
0.3	0.4	0.80	0.94	1.0

TABLE 5

TAILWATER AT FAILURE

			y_2 at failure		
y_c	H	D_n	Gravel	Sand	Yuma Sand
.2	.8	.997	.05	.10	.15
.3	.3	.89	.15	.20	.25
.4	.4	.99	.25	.30	.35
.2	.4	.12	.05	.10	.15
.3	.6	.11	.25	.30	.40
.4	.8	.11	.45	.50	.60

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

From time to time nature has a way of showing us that we indeed are insignificant and natural forces are very powerful. This does not mean that we should succumb to these forces, but rather we should use past events and experiences to help define the planned objective of the future.

For the past 50 years ADOT has built gabion straight drop spillways, and probably has the most experience with these structures. Their designs have been used extensively in the Southwest and even into Northern California. From records, plans, and recent studies (Arizona Department of Transportation, 1979) it is evident that many of these designs are inadequate.

The main problem when designing a straight drop structure is determining what the tailwater will be. Once this is determined, the rest of the design is straightforward. However, if degradation occurs, then the tailwater may become lower than that which was assumed.

Two important results were discovered from this study. First, the length of the drop apron must extend past the area where the hydraulic jump occurs. If this is not done, excessive scour will occur at the end of the basin even under tailwater depths much higher than for the design condition. This poses serious problems because it has been the practice in the past to make the apron as short as possible to reduce costs. Reducing the length of the apron reduces the capability of the structure to carry design flows because lower flows will cause high erosion and, depending on a time parameter, might cause the failure of the structure. From the flume studies, it was found that failure came rapidly after the initial movement of soil from under the gabions.

Second, if the apron length is so designed that it is longer than the jump, the tailwater will control the scour depth. If the tailwater falls in the design range, the scour that will result can be estimated by the equation $2y_c + H$. This is independent of sediment size.

If more degradation occurs and the tailwater falls below the hydraulic jump range, a wave will form over the end of the basin. The scour depth is close to that obtained in the hydraulic jump range except that the geometry of the scour hole changes, as shown in Figure 12. If the tailwater falls below the range for which the wave forms, sweepout will occur which will result in failure of the structure.

It is recommended that designers lengthen rather than shorten drop aprons so that turbulence in the hydraulic jump area is reduced before the end of the apron. Also, the straight drop spillway should be designed using a range of tailwaters for a given flow to set the apron elevation and analyze the potential scour downstream of the structure.

An example using Figure 14 for the design of a drop structure in a fictitious channel can be referred to in Appendix C. This gives the reader a step by step procedure for the design of a structure. In Appendix D a comparison of the straight drop structure with the sloping sill is presented.

APPENDIX A

DATA

RUN NUMBER: 1 → 8

Sediment Size: gravel; dia = 6.1 mm = 0.24 in

Drop Height: 0.20 ft

Head over Weir = 0.697

$q = 0.507$ cfs/ft

y_c (computed) = 0.199 ft

y_c (measured) = 0.17 ft

$v_c = 2.98$ ft/sec

$F_r = 1.27$

$D_n = 0.997$

$L_b = 1.00$ ft

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
1	0.50	0.30	1.69	0.40	1.27	0	0.20
2	0.50	0.25	2.03	0.35	1.45	0.05	0.25
3	0.50	0.20	2.53	0.30	1.69	0.10	0.30
4	0.50	0.13	3.90	0.25	2.03	0.15	0.40
5	0.50	0.12	4.22	0.20	2.53	0.20	0.50
6	0.50	0.12	4.22	0.15	3.38	0.25	0.50
7	0.50	0.10	5.07	0.10	5.07	0.30	0.55
8	0.50	0.05	10.14	0.05	10.14	0.35	

RUN NUMBER: 9

Sediment Size: gravel

Drop Height: 0.40 ft

Head over Weir = 0.902

q = 0.966 cfs/ft

y_c (computed) = 0.307 ft

y_c (measured) = 0.30 ft

v_c = 3.15 ft/sec

F_r = 1.00

D_n = 0.453

L_b = 1.25 ft

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
9	.75	.20					

RUN NUMBER: 10 → 15

Sediment Size: gravel

Drop Height: 0.40 ft

Head over Weir = 1.13

q = 1.696 cfs/ft

y_c (computed) = 0.447 ft

y_c (measured) = 0.38 ft

v_c = 3.79 ft/sec; V_c = 4.46 ft/sec

F_r = 1.27

D_n = 1.395

L_b = 1.20 ft (3 Y_c)

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
10	0.80	0.40	4.24	0.75	2.26	0.097	1.05
11	0.85	0.35	4.84	0.70	2.42	0.147	1.05
12	0.9	0.28	6.06	0.675	2.51	0.172	1.15
13	1.0	0.20	8.48	0.65	2.61	0.197	1.25
14	1.1	0.20	8.48	0.55	3.08	0.297	0.80
15	1.1	0.20	8.48	0.46	3.69	0.387	0.81

RUN NUMBER: 16 → 30

Sediment Size: gravel

Drop Height: 0.30 ft

Head over Weir = 0.870

q = 0.882 cfs/ft

y_c (computed) = 0.290 ft

y_c (measured) = 0.260 ft

v_c = 3.39 ft/sec

F_r = 1.17

D_n = 0.89

L_b = 1.50 ft (5 Y_c)

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
16	0.55			0.60	1.47	-0.01	0.60
17	0.80			0.55	1.60	0.04	0.70
18	0.70	0.35	2.52	0.53	1.66	0.06	0.73
19	0.65	0.30	2.94	0.50	1.76	0.09	0.76
20	0.60	0.23	3.83	0.45	1.96	0.14	0.75
21	0.62	0.16	5.51	0.40	2.21	0.19	0.70
22	0.60	0.15	5.88	0.35	2.52	0.24	0.75
23	0.60	0.14	6.30	0.35	2.52	0.24	0.80
24	0.60	0.14	6.30	0.30	2.94	0.29	0.80
25	0.60	0.14	6.30	0.275	3.21	0.315	0.875
26	0.60	0.14	6.30	0.25	3.53	0.34	0.60
27	0.60	0.14	6.30	0.20	4.41	0.39	0.60
28	0.60	0.14	6.30	0.15	5.88	0.44	0.60
29	0.60	0.10	8.82	0.10	8.82	0.49	0.70
30	0.60	0.10	8.82	0.10	8.82	0.49	0.90

RUN NUMBER: 31 → 36

Sediment Size: gravel

Drop Height: 0.40 ft

Head over Weir = 1.056

q = 1.43 cfs/ft

y_c (computed) = 0.399 ft

y_c (measured) = 0.36 ft

v_c = 3.97 ft/sec

F_r = 1.16

D_n = 0.99

L_b = 2.0 ft (5 Y_c)

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
31	0.90	0.40	3.58	0.70	2.04	0.10	0.95
32	1.00	0.25	5.72	0.60	2.38	0.20	1.00
33	1.00	0.23	6.22	0.45	3.18	0.35	1.05
34	1.00	0.23	6.22	0.40	3.56	0.40	1.05
35	1.00	0.23	6.22	0.30	4.77	0.50	0.85
36	1.00	0.20	7.15	0.20	7.15	0.60	0.90

RUN NUMBER: 37 → 39

Sediment Size: Yuma Sand, dia = 0.30 mm = 0.011 in

Drop Height: 0.20 ft

Head over Weir = 0.697

$q = 0.50$ cfs/ft

y_c (computed) = 0.20 ft

y_c (measured) = 0.17 ft

$v_c = 2.94$ ft/sec

$F_r = 1.25$

$D_n = 0.97$

$L_b = 0.60$ ft (3 Y_c)

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
37	0.45	0.17	2.94	0.40	1.25	0	0.55
38	0.50	0.12	4.17	0.30	1.67	0.10	0.60
39	0.50	0.12	4.17	0.25	1.00	0.15	0.85

RUN NUMBER: 40 → 46

Sediment Size: Yuma Sand

Drop Height: 0.20 ft

Head over Weir = 0.697

q = 0.50 cfs/ft

y_c (computed) = 0.20 ft

y_c (measured) = 0.17 ft

v_c = 2.94 ft/sec

F_r = 1.25

D_n = 0.97

L_b = 1.0 ft (5 Y_c)

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
40	0.45	0.20	2.5	0.40	1.25	0	0.60
41	0.50	0.20	2.5	0.35	1.43	0.05	0.65
42	0.50	0.20	2.5	0.20	1.67	0.10	0.80
43	0.45	0.13	3.85	0.25	2.00	0.15	0.95
44	0.50	0.13	3.85	0.20	2.50	0.20	0.75
45	0.45	0.13	3.85	0.15	3.33	0.25	0.70
46	0.45	0.10	5.00	0.10	5.00	0.30	0.70

RUN NUMBER: 47 → 52

Sediment Size: Yuma Sand

Drop Height: 0.30 ft

Head over Weir = 0.870

q = 0.882 cfs/ft

y_c (computed) = 0.29 ft

y_c (measured) = 0.27 ft

v_c = 3.27 ft/sec

F_r = 1.13

D_n = 0.89

L_b = 1.50 ft (5 Y_c)

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
47	0.60	0.40	2.21	0.55	1.60	0.04	0.95
48	0.62	0.30	2.94	0.50	1.76	0.09	0.95
49	0.65	0.175	5.04	0.40	2.21	0.19	0.85
50	0.65	0.15	5.88	0.35	2.52	0.24	1.05
51	0.65	0.15	5.88	0.30	2.94	0.29	1.05
52	0.65	0.15	5.88	0.25	3.53	0.34	1.05

RUN NUMBER: 53 → 58

Sediment Size: Yuma Sand

Drop Height: 0.40 ft

Head over Weir = 1.056

q = 1.43 cfs/ft

y_c (computed) = 0.40 ft

y_c (measured) = 0.36 ft

v_c = 3.97 ft/sec

F_r = 1.16

D_n = 0.99

L_b = 2.0 ft (5 Y_c)

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
53	0.80	0.40	3.58	0.70	2.04	0.10	1.2
54	0.85	0.30	4.77	0.60	2.38	0.20	1.25
55	0.90	0.24	5.96	0.55	2.60	0.25	1.3
56	1.0	0.21	6.81	0.45	3.18	0.35	1.40
57	1.0	0.21	6.81	0.40	3.56	0.40	0.95
58	1.0	0.21	6.81	0.35	4.09	0.45	

RUN NUMBER: 59 → 64

Sediment Size: Yuma Sand

Drop Height: 0.40 ft

Head over Weir = 0.697

q = 0.50 cfs/ft

y_c (computed) = 0.19 ft

y_c (measured) = 0.17 ft

v_c = 2.94 ft/sec

F_r = 1.25

D_n = 0.12

L_b = 1.20 ft (6 Y_c)

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
59	submergence			0.55	0.91	0.04	0.65
60	0.50	0.25	2.00	0.45	1.11	0.14	1.15
61	0.55	0.10	5.00	0.30	1.67	0.29	0.80
62	0.55	0.10	5.00	0.25	2.00	0.34	0.90
63	0.55	0.10	5.00	0.20	2.50	0.39	0.80
64	0.55	0.10	5.00	0.15	3.33	0.44	0.90

RUN NUMBER: 65 → 69

Sediment Size: Yuma Sand

Drop Height: 0.60 ft

Head over Weir = 0.870

q = 0.882 cfs/ft

y_c (computed) = 0.29 ft

y_c (measured) = 0.26 ft

v_c = 3.27 ft/sec

F_r = 1.13

D_n = 0.11

L_b = 1.80 ft (6 Y_c)

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
65				0.85	1.04	0.04	1.15
66	1.0	subm.		0.70	1.26	0.19	1.30
67	0.80	0.20	4.41	0.50	1.76	0.39	1.30
68	0.80	0.115	7.67	0.45	1.96	0.44	1.30
69	0.80	0.115	7.67	0.40	2.21	0.49	1.30

RUN NUMBER: 70 → 74

Sediment Size: Yuma Sand

Drop Height: 0.80 ft

Head over Weir = 1.056

q = 1.43 cfs/ft

y_c (computed) = 0.40 ft

y_c (measured) = 0.36 ft

v_c = 3.97 ft/sec

F_r = 1.16

D_n = 0.99

L_b = 2.40 ft (6 Y_c)

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
70		subm.		1.20	1.19	.01	1.40
71				1.10	1.30	.10	1.40
72				1.00	1.43	.20	1.40
73	1.20	.20	7.15	.90	1.58	.30	1.50
74	1.10	.15	9.53	.80	1.79	.40	1.65

RUN NUMBER: 75 → 79

Sediment Size: Sand, dia = 0.66 mm = 0.026 in

Drop Height: 0.20 ft

Head over Weir = 0.70

$q = 0.51$ cfs/ft

y_c (computed) = 0.20 ft

y_c (measured) = 0.17 ft

$v_c = 3.00$ ft/sec

$F_r = 1.28$

$D_n = 1.00$

$L_b = 1.00$ ft (5 Y_c)

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
75	0.50	0.30	1.70	0.40	1.28	0	0.70
76	0.55	0.14	3.64	0.30	1.70	0.10	0.80
77	0.50	0.08	6.38	0.20	2.55	0.20	0.80
78	0.50	0.08	6.38	0.15	3.40	0.25	0.75
79	0.50	0.08	6.38	0.10	5.10	0.30	0.80

RUN NUMBER: 80 → 83

Sediment Size: Sand

Drop Height: 0.30 ft

Head over Weir = 0.870

$q = 0.882$ cfs/ft

y_c (computed) = 0.29 ft

y_c (measured) = 0.26 ft

$v_c = 3.39$ ft/sec

$F_r = 1.17$

$D_n = 0.90$

$L_b = 1.5$ ft (5 Y_c)

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
80	0.70	0.15	5.88	0.50	1.76	0.09	1.10
81	0.70	0.14	6.30	0.40	2.21	0.19	0.90
82	0.70	0.11	8.02	0.30	2.94	0.29	0.95
83	0.70	0.10	8.82	0.25	3.53	0.34	

RUN NUMBER: 84 → 86

Sediment Size: Sand

Drop Height: 0.40 ft

Head over Weir = 1.056

q = 1.43 cfs/ft

y_c (computed) = 0.40 ft

y_c (measured) = 0.36 ft

v_c = 3.97 ft/sec

F_r = 1.16

D_n = 0.99

L_b = 2.0 ft (5 Y_c)

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
84	1.0	0.22	6.50	0.60	2.38	0.20	1.20
85	1.0	0.20	7.15	0.50	2.86	0.30	1.00
86	1.0	0.20	7.15	0.40	3.56	0.40	1.10

RUN NUMBER: 87 → 89

Sediment Size: Sand

Drop Height: 0.40 ft

Head over Weir = 0.70

$q = 0.51$ cfs/ft

y_c (computed) = 0.20 ft

y_c (measured) = 0.17 ft

$v_c = 3.00$ ft/sec

$F_r = 1.28$

$D_n = 0.12$

$L_b = 1.20$ ft (6 Y_c)

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
87	0.60	subm.		0.45	1.11	0.15	0.95
88	0.60	0.10	5.10	0.30	1.67	0.30	0.70
89	0.60	0.10	5.10	0.25	2.00	0.35	0.65

RUN NUMBER: 90 → 91

Sediment Size: Sand

Drop Height: 0.60 ft

Head over Weir = 0.870

$q = 0.882$ cfs/ft

y_c (computed) = 0.29 ft

y_c (measured) = 0.26 ft

$v_c = 3.39$ ft/sec

$F_r = 1.17$

$D_n = 0.11$

$L_b = 1.80$ ft (6 Y_c)

Run #	L_d	y_1	v_1	y_2	v_2	ΔW_s	D_s
90	0.85	subm.		0.75	1.17	0.14	1.35
91	0.85	0.12	7.35	0.45	1.96	0.44	1.35

APPENDIX B

TRIALS TO CHECK HYDRAULIC DESIGN CHART

Trial	H	y_c	y_o	q	D_n	$\frac{L_d}{H}$	$\frac{y_2}{H}$	$\frac{\text{min. T.W.}}{H}$	L_d	y_2	measured L_d	measured y_2
1	0.40	0.31	0.15	0.979	0.465	2.1	1.3	1.1	0.84	0.52	0.80	0.50
2	0.60	0.507	0.20	2.05	0.17	1.55	1.05	0.84	0.93	0.63	1.0	0.65
3	0.60	0.214	0.10	0.563	0.045	1.06	0.76	0.59	0.64	0.45	0.70	0.45
4	0.40	0.29	0.30	0.882	0.427	2.0	1.25	1.075	0.80	0.50	0.82	0.50

APPENDIX C

APPLICATION

To illustrate the conclusions contained in this report and demonstrate use of the design charts, the following example is considered:

Channel 100 feet wide, banks 7 feet high, slope .003 ft/ft
n value equal to .035
Q design of 3000 cfs
Bed material median diameter 1/4 inch or 0.02 feet

A head cut of 3 feet is moving towards a highway crossing the stream. The job is to investigate a sill structure to stabilize the head cut and protect the crossing.

Assuming a rectangular channel, the following flow characteristics are found:

Normal flow: $y_n = 4.6$ feet, $V_n = 6.5$ fps
Critical flow: $y_c = 3$ feet, $V_c = 10$ fps

If the apron is placed 3 feet lower than the bed, then H will be equal to y_c, ($y_c/H = 1$). Flow per unit width is 30 cfs/ft. The drop number is 1.0. From Figure 14, the following relationships are found:

$\frac{y_2}{H} = 1.5$ $\qquad$ $y_2 = 4.5$ ft

$\frac{L_d}{H} = 2.5$ $\qquad$ $L_d = 7.5$ ft

$\frac{y_1}{H} = .63$ $\qquad$ $y_1 = 1.89$ ft

$\frac{\text{min. T.W.}}{H} = 1.35$ $\qquad$ $\text{min. T.W.} = 4.05$ ft

From Figure 16 for a Froude number of 1.0 it can be seen that tailwater values of 3.9 to 5.4 will result in the hydraulic jump. To estimate the scour, the following equation is used:

$$D_S = 2y_C + H$$

$$D_S \approx 9.0 \text{ feet}$$

The total length of the basin will be L_d plus the length required for the jump to form. Since $(L_d + H + 2y_C)$ equals 16.5 feet and $4y_2$ equals 18.0 feet, the basin apron should be made 18.0 feet long.

APPENDIX D

COMPARISON WITH SLOPING SILL

To compare the sloping sill with the straight drop spillway, the following example is used:

- * Channel 200 feet wide, banks 6 feet high, slope .005 ft/ft
- * n value equal to .035
- * Q design of 10,000 cfs
- * Bed material median diameter 1/8 inch or .01 feet

A head cut of 8 feet is moving up channel. The job is to stabilize the head cut by putting in the least costly structure.

Assuming a rectangular channel, the following flow characteristics are found:

Normal Flow:	y_n = 5.1 feet,	V_n = 8.9 fps
Critical Flow:	y_c = 4.0 feet,	V_c = 11.4 fps

If a sloping sill 1V:4H protected by 12 inch rock is considered, Laursen and Flick [8] estimate the depth of scour to be 28.4 feet. To build this structure would require an apron length of 117 feet. Total material would be 953 cy of rock. The depth of scour could be reduced by riprapping the scour hole.

If a straight drop spillway were designed,

$\frac{y_c}{H} = .50$

$q = 50$ cfs/ft

$D_n = .15$

From Figure 14 the following parameters are taken:

$\frac{y_1}{H} = .28$ $y_1 = 2.24$ ft

$\frac{y_2}{H} = .10$ $y_2 = 8.0$ ft

$\frac{L_d}{H} = 1.5$ $L_d = 12.0$ ft

$\frac{\text{min. T.W.}}{H} = .82$ min. T.W. = 6.56 ft

($L_d + 2y_c + H$) equals 28.0 feet and $4y_2 = 32.0$ feet. Therefore, the total basin length should be approximately 32.0 feet. The total material needed for the structure is 600 cy. Depth of scour for the straight drop is

$$D_s \approx 12.0 \text{ feet}$$

Difference between 600 and 953 cy is 353 cy. Depending on the cost of material and the confidence in the hydrologic information, it might be advantageous to use the straight drop spillway. However, with a head cut of 8 feet, errors in tailwater values could easily exceed 20% and failure would be highly probable. For a savings of $10,000 ($40/cy x 300 cy), the risks are excessive.

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