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STREAM CHANNEL STABILITY

APPENDIX B

'EL STUDY OF THE LOW DROP GRADE CONTROL STRUCTURES

Project Objective 1

by

W. C. Little and J. B. Murphey

USDA Sedimentation Laboratory
Oxford, Mississippi

April 1981

Prepared for
US Army Corps of Engineers, Vicksburg District
Vicksburg, Mississippi

Under
Section 32 Program, Work Unit 7

STREAM CHANNEL STABILITY

APPENDIX B

Model Study of the Low Drop Grade Control Structures

Project Objective 1

by

W. C. Little[1] [redacted] J. B. Murphey[2]

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Preface

This report presents the results of hydraulic model tests for low drop grade control structures. The results are presented in dimensionless relationships. Tentative design criteria are formulated for the design of low drop grade control structures with baffle energy dissipation devices. A method is given to determine the size of the stilling basin and the size and placement of a baffle pier or baffle plate in the basin to achieve optimum flow conditions in the downstream channel.

Table of Contents

List of Tables

List of Figures

CONVERSION FACTORS, U.S. CUSTOMARY TO METRIC (SI) AND METRIC (SI) TO U.S. CUSTOMARY UNITS OF MEASUREMENT[1/]

Units of measurement used in this report can be converted as follows:

To convert	To	Multiply by
mils (mil)	micron (μm)	25.4
inches (in)	millimeters (mm)	25.4
feet (ft)	meters (m)	0.305
yards (yd)	meters (m)	0.914
miles (miles)	kilometers (km)	1.61
inches per hour (in/hr)	millimeters per hour (mm/hr)	25.4
feet per second (ft/sec)	meters per second (m/sec)	0.305
square inches (sq in)	square millimeters (mm^2)	645.
square feet (sq ft)	square meters (m^2)	0.093
square yards (sq yd)	square meters (m^2)	0.836
square miles (sq miles)	square kilometers (km^2)	2.59
acres (acre)	hectares (ha)	0.405
acres (acre)	square meters (m^2)	4,050.
cubic inches (cu in)	cubic millimeters (mm^3)	16,400.
cubic feet (cu ft)	cubic meters (m^3)	0.0283
cubic yards (cu yd)	cubic meters (m^3)	0.765
cubic feet per second (cfs)	cubic meters per second (cms)	0.0283
pounds (lb) mass	grams (g)	454.
pounds (lb) mass	kilograms (kg)	0.453
tons (ton) mass	kilograms (kg)	907.
pounds force (lbf)	newtons (N)	4.45
kilogram force (kgf)	newtons (N)	9.81
foot pound force (ft lbf)	joules (J)	1.36
pounds force per square foot (psf)	pascals (Pa)	47.9
pounds force per square inch (psi)	kilopascals (kPa)	6.89
pounds mass per square foot (lb/sq ft)	kilograms per square meter (kg/m^2)	4.88
U.S. gallons (gal)	liters (L)	3.79
quart (qt)	liters (L)	0.946
acre-feet (acre-ft)	cubic meters (m^3)	1,230.
degrees (angular)	radians (rad)	0.0175
degrees Fahrenheit (F)	degrees Celsius (C)[2/]	0.555

2/ To obtain Celsius (C) readings from Fahrenheit (F) readings, use the following formula: C = 0.555 (F-32).

Metric (SI) to U.S. Customary

To convert	To	Multiply by
micron (μm)	mils (mil)	0.0394
millimeters (mm)	inches (in)	0.0394
meters (m)	feet (ft)	3.28
meters (m)	yards (yd)	1.09
kilometers (km)	miles (miles)	0.621
millimeters per hour (mm/hr)	inches per hour (in/hr)	0.0394
meters per second (m/sec)	feet per second (ft/sec)	3.28
square millimeters (mm^2)	square inches (sq in)	0.00155
square meters (m^2)	square feet (sq ft)	10.8
square meters (m^2)	square yards (sq yd)	1.20
square kilometers (km^2)	square miles (sq miles)	0.386
hectares (ha)	acres (acre)	2.47
square meters (m^2)	acres (acre)	0.000247
cubic millimeters (mm^3)	cubic inches (cu in)	0.0000610
cubic meters (m^3)	cubic feet (cu ft)	35.3
cubic meters (m^3)	cubic yards (cu yd)	1.31
cubic meters per second (cms)	cubic feet per second (cfs)	35.3
grams (g)	pounds (lb) mass	0.00220
kilograms (kg)	pounds (lb) mass	2.20
kilograms (kg)	tons (ton) mass	0.00110
newtons (N)	pounds force (lbf)	0.225
newtons (N)	kilogram force (kgf)	0.102
joules (J)	foot pound force (ft lbf)	0.738
pascals (Pa)	pounds force per square foot (psf)	0.0209
kilopascals (kPa)	pounds force per square inch (psi)	0.145
kilograms per square meter (kg/m^2)	pounds mass per square foot lb/sq ft)	0.205
liters (L)	U.S. gallons (gal)	0.264
liters (L)	quart (qt)	1.06
cubic meters (m^3)	acre-feet (acre-ft)	0.000811
radians (rad)	degrees (angular)	57.3
degrees Celsius (C)	degrees Fahrenheit (F)3/	1.8

1/ All conversion factors to three significant digits.

3/ To obtain Fahrenheit (F) readings from Celsius (C) readings, use the following formula: F = 1.8C + 32.

NOTATION

A	-	Channel Cross Sectional Flow Area
B	-	Bottom Width of Channel or Weir
D	-	Hydraulic mean depth = $\frac{A}{T}$
E		Specific Energy = $Y + \frac{V^2}{2g}$
F	-	Froude Number, $V/\sqrt{gh}$
H	-	Absolute Drop Height
H_b	-	Baffle Plate Height
L_b	-	Baffle Pier or Plate Length
L_{SB}	-	Length of Stilling Basin
Q	-	Discharge
S	-	Slope
S_B	-	Side Slope of Stilling Basin (Vertical to Horizontal)
T	-	Top Width of Channel
V	-	Average Velocity
V_M	-	Maximum Average Velocity Corresponding to Y_m
W_{SB}	-	Maximum Width of Stilling Basin, horizontal width measured at the elevation of the weir crest.
X_b	-	Distance from Weir to Crest of First Undulating Wave
Y	-	Depth of Flow
Y_b	-	Top Height of Baffle Pier or Plate above Weir
Y_c	-	Critical Depth
Y_m	-	Minimum Depth
Y_{SB}	-	Depth of Stilling Basin
Y_2	-	Downstream Flow Depth
g	-	Acceleration Due to Gravity
α	-	Velocity Head Energy Coefficient
c	-	Subscript Indicating Critical Flow Conditions
M	-	Subscript Indicating Maximum for that Parameter
m	-	Subscript Indicating Minimum for that Parameter

1 INTRODUCTION

Streams and rivers incised and flowing through alluvial valleys many times have little or no natural bed material, such as rock outcrops, to serve as bed control. Such is the case with a majority of the streams located within the Yazoo basin. These streams originate in the hills of Northwest Mississippi and flow west through the bluffline into the Mississippi Delta. The streams in the hills have steep channel gradients and flow through valleys which are alluvial. During the past 50 years a majority of these streams have degraded seriously. Degradation has occurred as a result of surface erosion of the bed but most often by the upstream progression of headcuts or overfalls. Because valleys of the Yazoo basin are highly stratified, overfalls are created when the stream breaks through a resistant layer of dense silt or clay into a layer of unconsolidated sand. The erosive energy of the flow is concentrated at the overfall. Headcuts are prevalent throughout the Yazoo basin and several headcuts may be active on a stream system simultaneously.

The channel gradient downstream from the headcut is reduced as the headcut moves upstream, thereby increasing the height of the overfall. These headcuts generally range from a few inches where they originate up to several feet as they move upstream. Continued degradation produces higher and steeper banks resulting in massive slump and slide failures and subsequent widening of the channel. Under these circumstances, even bank revetment techniques are ineffective and many times fail completely. One must then reason "a priori" that bed stability is prerequisite to bank stability.

Grade control structures are needed to halt continued channel degradation. A series of structures carefully spaced throughout the length of a stream is generally required to stabilize the bed. Structures are particularly important at the confluence with larger streams, especially if the larger streams are regulated. Each structure must have a relatively low drop to avoid excessive overbank flow which could breach the structure. Many of the channel grade control structures used by the USDA Soil Conservation Service are of the order of 1 to 4 feet absolute drop height. In the past, these structures have not functioned as intended because no means of energy dissipation at the drop structure was provided. Many of these structures were constructed using riprap to form a rock sill over which the water passed. As a consequence, large scour holes developed immediately downstream of the rock. Many of these structures have been observed to fail as the rock fell into the scour hole. The usefulness of the structure was lost, and in fact, another headcut was initiated that moved upstream.

The intent of this report is to present a new concept for low drop, channel grade control structures.

2 SPECIFIC CASE MODEL TESTS

In April of 1974, the Soil Conservation Service requested the authors to perform specific case model studies for proposed low-drop grade-control structures for Indian Creek, located in Chickasaw County, Mississippi, and for Grady-Gould Canal located in Desha County, Arkansas. Also, in July of 1976 the Vicksburg District Corps of Engineers requested a specific case model study be conducted for a proposed low-drop, grade control structure for the North Fork of Tillatoba Creek. These three specific case model studies are discussed separately to show how the concept of low-drop structures evolved.

At that time, the authors had observed several failures of riprap grade-control structures. A review of literature revealed that very little was known about them.

In order to investigate grade control structures as quickly as possible, a small existing sand-box type model basin was used to study a model of the proposed Indian Creek structure. A metal plate with a trapezoidal cross section was used as a "cutoff" wall and weir section for the drop. The area immediately downstream from the weir, was protected only by 0.4 mm sand and served as a "scour hole". The remainder of the downstream section and all of the upstream channel were protected by gravel to simulate a stable channel. Several exploratory tests were conducted to determine the approximate shape (plan geometry) and depth of "scour hole" that would be eroded from the flow over the weir. Results from these tests were:

1. Width of the "scour hole" was found to be 1.5 to 2.0 times the upstream channel bottom width.
2. Depth of the scour hole was approximately equal to the sum of the physical drop (difference in elevation between upstream and downstream channel bottom) and the critical depth (corresponding to discharge used).
3. The "scour hole" functioned satisfactory when the length was from 1.25 to 3 times the upstream channel bottom width.
4. Undulating waves developed when the physical drop through the structure was less than or equal to critical depth.

2.1 INDIAN CREEK STRUCTURE

Using the above criteria, a model "scour hole" for Indian Creek was formed. The geometry and flow parameters were:

1. Discharge - 1800 cfs.
2. Channel bottom width - 18 feet.
3. Height of drop - 4 feet.
4. Upstream channel slope - 0.0014.
5. Downstream channel slope - 0.0008.
6. Channel side slope - 1 V. to 2.5 H.

A model to prototype scale ratio of 1 to 25.4 was used. The "scour hole" type stilling basin was lined with gravel large enough to prevent movement. Figure 1 is a photograph of the model basin showing the shape of the stilling basin. Figure 2 is a view of the water surface looking upstream, showing the undulating waves. The relative drop height, H/Y_c,

where H = Absolute drop height,

Y_c = Critical flow depth,

for this structure was 0.76, indicating that undulating waves would be generated.

Early in this study the authors observed that an obstruction located in the stilling basin would help reduce the effects of the surface waves. Many different shapes and sizes of blocks were tried, at various locations in the stilling basin, to find the optimum size and placement of block to provide the smoothest water surface. A block 6 feet square (prototype) was found to be optimum for the Indian Creek structure. A model of this block (2.84 inches square) can be seen on the bank in Figure 2. The block was placed in the center of the stilling basin laterally and longitudinally with its upper surface at the elevation of the crest of the second undulation (measured without block in place). The block destroyed the organized flow pattern of the undulating waves and gave a relatively smooth water surface entering the downstream channel as shown in Figure 3.

Prototype dimensions of the stilling basin and energy dissipation block obtained from this model study were used by the Soil Conservation Service to design the Indian Creek drop structure. The weir and cutoff wall were constructed by driving interlocking sheet pile. The baffle pier was constructed by driving interlocking sheet pile to form a 6-ft square

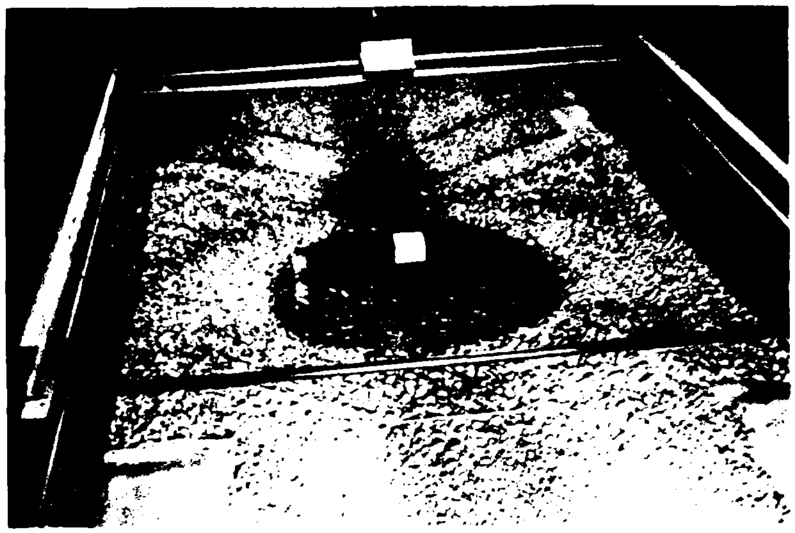

Figure 1 Photograph of Model Basin Showing Shape of Stilling Basin

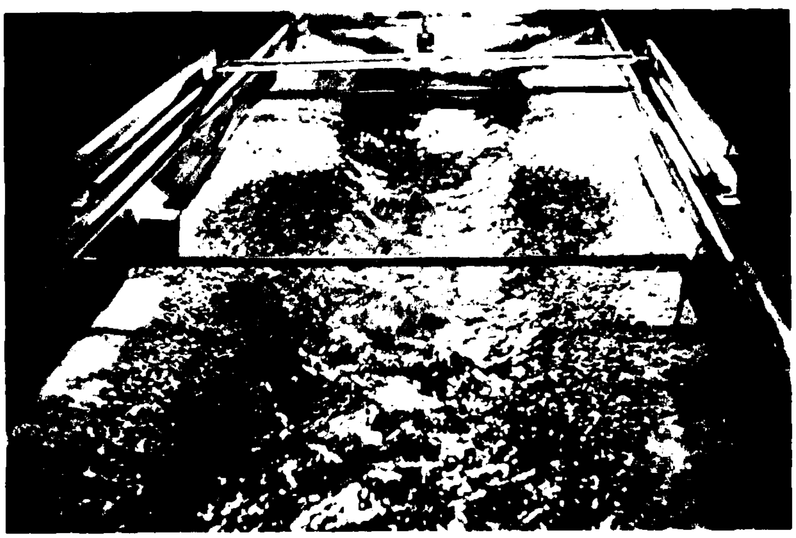

Figure 2 Photograph of Water Surface Showing Undulating Waves (Looking Upstream)

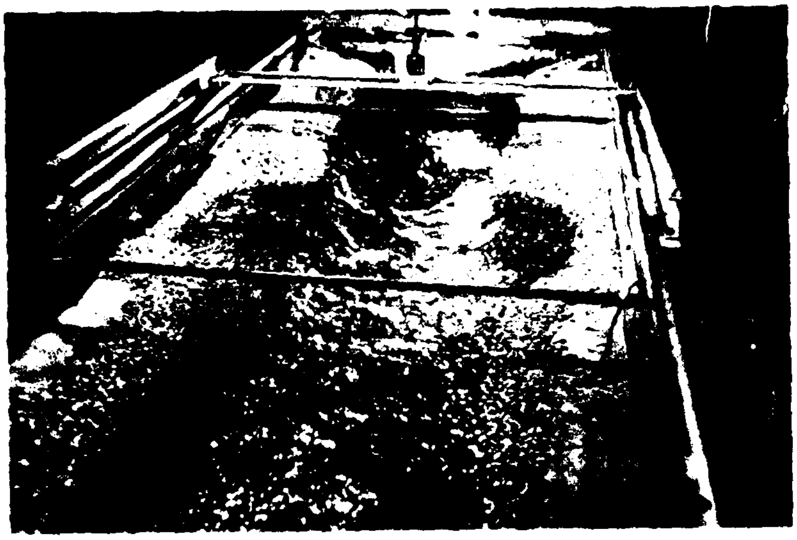

Figure 3 Photograph Showing Water Surface With Baffle Pier Dampening Waves

Figure 4 Photograph of Indian Creek Structure in 1976

box. It was filled with rock and capped with concrete. Riprap was sized using the criteria developed by Anderson, et al. (1970) and modified by Blaisdell (1973).

The structure was completed in October, 1974 at a cost of $210,000. Figure 4 is a view of the Indian Creek structure in October, 1976. There is no apparent damage to the structure.

2.2 GRADY-GOULD STRUCTURE

A specific case hydraulic model of a proposed structure on Grady-Gould Canal was installed in the same model basin as the Indian Creek model. It is shown in figure 5. Design parameters were:

1. Discharge - 1800 cfs.
2. Channel bottom width - 30 feet.
3. Height of drop - 4 feet.
4. Channel slope - 0.0003.
5. Channel side slope - 1 V. to 2.5 H.

A model to prototype scale ratio of 1 to 42 was used. Tests were conducted without any auxillary energy dissipation device. Undulating waves developed as shown in Figure 6. Critical depth, Y_c, is 4.25 feet which gives a relative drop height, $H/Y_c = 0.94$, again indicating that undulating waves should develop.

A different type of obstruction to break up the undulating waves was developed for this structure. It is shown in Figure 5. The authors termed this device a "baffle plate". The baffle plate enables flow both under and over the top and provides a smoother water surface than the baffle pier. In field structures, the baffle plate is constructed by driving two H-piles on which either wooden planks or interlocked sheet pile are fastened horizontally. Figure 7 is a view of the same flow as that shown in figure 6 but a baffle plate was used to dissipate the undulating waves. Note the extremely smooth water surface in Figure 7, indicating that the baffle plate totally destroyed the highly organized flow pattern of the undulating waves. Figure 8 is a view of the completed Grady-Gould structure in May, 1976. This structure cost $140,000.

Figure 5 Photograph of Model Stilling Basin and Baffle Plate for Grady-Gould Structure

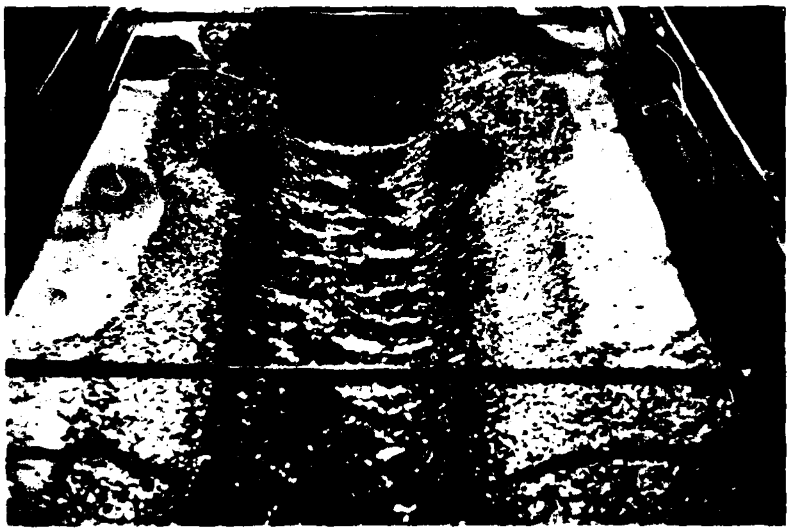

Figure 6 Photograph of Undulating Waves in Grady-Gould Model Tests

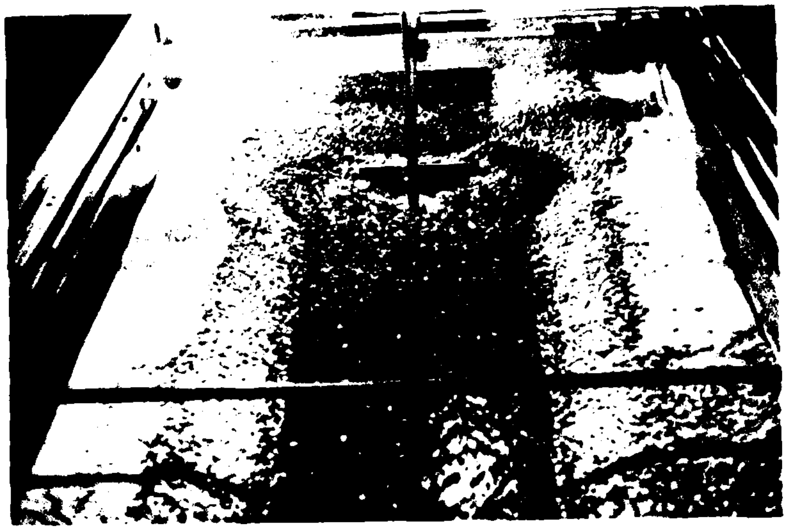

Figure 7 Photograph of the Water Surface With Baffle Plate Breaking Up the Undulating Waves

Figure 8 Photograph of Grady-Gould Canal Low Drop Structure, May, 1976

2.3 NORTH FORK, TILLATOBA CREEK STRUCTURE

A specific case hydraulic model of a structure proposed for North Fork, Tillatoba Creek was installed in the same model basin as the Indian Creek and Grady-Gould structures. Design parameters were:

1. Discharge - 8000 cfs
2. Channel bottom width - 70 feet
3. Height of drop - 4 feet
4. Channel slope - 0.0016
5. Channel side slope - 1V. to 2.5H.

A model to prototype scale ratio of 1 to 46.7 was used. Critical depth, Y_c, for this channel was 6.91 feet which gives a relative drop height, $H/Y_c = 0.58$, indicating that undulating waves should develop. Discharges of 2000, 4000, 6000, 8000, 10,000 and 12,000 cfs (prototype); 0.134, .269, .403, .538, .672 and .807 cfs in model were used for the tests. These flows produced critical depths, Y_c, ranging from 0.061 feet to 0.190 feet and relative drop heights, H/Y_c, from 0.45 to 1.41.

A baffle pier was selected for use on this structure and was found by trial and error to be 40 feet in length (prototype). It can be seen in Figure 9. On June 24, 1980 this structure experienced a peak flow of approximately 15,400 cfs, almost twice the design discharge, without damage.

The three drop structures previously discussed, provided ideas for a generalized hydraulic model study of low-drop grade control structures.

Figure 9 Photograph of completed Grade Control Structure, North Fork-Tillatoba Creek, March 1978.

3 BACKGROUND AND CONCEPT OF LOW DROP STRUCTURE

The performance of the low drop structure discussed herein, is related to the properties of the hydraulic jump, so a discussion of pertinent hydraulic jump properties is appropriate. Two types of hydraulic jump are recognized: the undular jump and the direct jump. In an undular jump, the flow passes from supercritical velocities (low stage) to subcritical velocities (high stage) through a series of undulating waves. In a direct jump, flow passes from the low stage through a turbulent roller to the high stage. A large amount of energy is dissipated in the turbulent roller of the direct jump but little energy is dissipated in the undular jump.

A schematic drawing of a low drop structure and pertinent features of the undular jump are shown in Figure 10. As the flow approaches the steep slope of the drop, it accelerates to critical depth, Y_c, near the break in slope, and continues to accelerate until the depth is a minimum, Y_m. Beyond this point, the depth increases and flow passes from a lower to a higher stage through a series of undulations that gradually diminish in size. If the drop height, H, were great enough, the flow would accelerate to a higher velocity (lower Y_m), and then change rapidly from a low stage to a high stage in a direct hydraulic jump. The ratio, H/Y_c, the relative drop height, determines the minimum depth, Y_m (maximum velocity, V_M), which determines the type of jump that will occur.

The specific energy of a channel with a small slope, assuming the energy coefficient $\alpha = 1$, is

$$E = Y + \frac{Q^2}{2gA^2} , \tag{1}$$

where E is specific energy, Y is depth of flow, Q is discharge, A is cross sectional flow area, V is the velocity and equal to Q/A, and g is acceleration due to gravity.

Differentiation of equation 1 with respect to depth (see Chow, 1959) yields

$$\frac{dE}{dY} = 1 - \frac{V^2T}{gA} , \tag{2}$$

where T is the channel width at the water surface.

The hydraulic mean depth, D, is:

$$D = A/T . \tag{3}$$

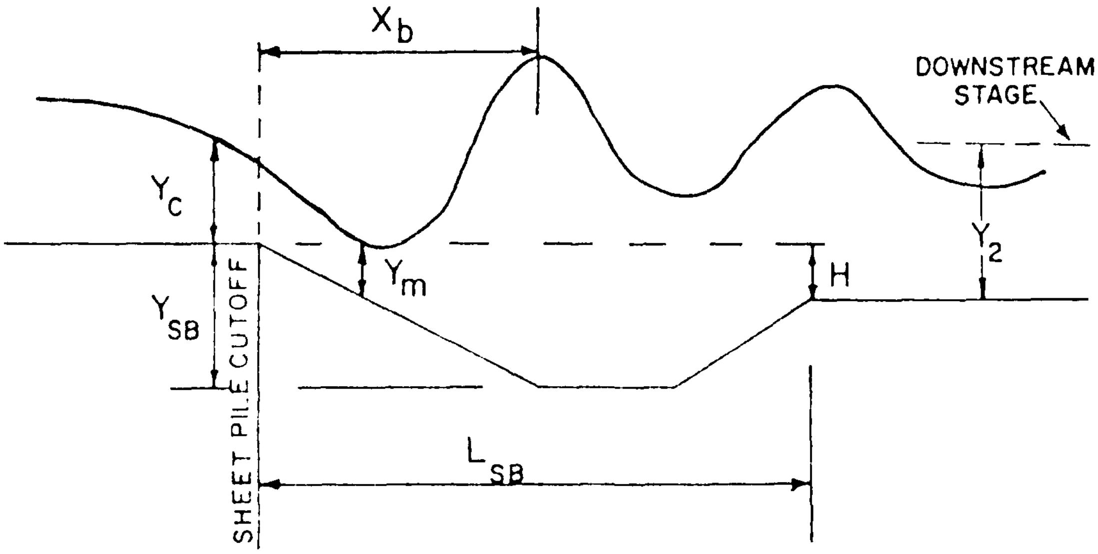

Figure 19. Schematic of an Hydraulic Low Drop Structure.

Substituting equation 3 into 2 gives

$$\frac{dE}{dY} = 1 - \frac{V^2}{gD} \tag{4}$$

At critical depth, the specific energy is a minimum, dE/dY is zero. Setting equation 4 equal to zero gives

$$\frac{V_c^2}{2g} = \frac{D_c}{2} \tag{5}$$

or

$$\frac{V_c}{\sqrt{gD_c}} = 1 = F \tag{6}$$

where F is the Froude Number and V_c and D_c are the velocity and depth at minimum specific energy respectively.

In a horizontal rectangular channel, Rouse (1958) and Bakhmeteff and Matzke (1936) showed that the undular jump could occur with Froude Numbers as high as 1.73. As the Froude number increases above 1.73, the smooth water surface of the undulating wave first becomes cusped and then falls over breaking, eventually becoming a direct hydraulic jump. At the upper limit of the Froude number for an undular jump, 1.73, Chow (1959) shows that the ratio of the specific energy, E_2, downstream from the drop to the specific energy, E_1, upstream, E_2/E_1, is approximately 0.95. Thus, only a maximum of 5 percent of the energy is dissipated in the undular jump. At Froude numbers exceeding 1.73, where a direct jump exists, a greater proportion of the energy is dissipated. For example, for a Froude number of 4, E_2/E_1 is approximately 0.6 thus 40 percent of the energy is dissipated in the violent turbulence generated by the well developed direct jump.

In a study of the undular jump, Jones (1964) concluded that such a jump on a horizontal floor could occur only up to a Froude number of approximately 1.41. The difference between the results of Rouse (1958) and those of Jones (1964) is that Jones believes that the undular jump can "persist only as long as the rising front of the wave can retain its solitary - wave affiliation." However, from a practical standpoint, the undulating waves persist, and little energy is dissipated, until the Froude number exceeds 1.73 and they are replaced by a direct jump.

There were no data available to relate the characteristics of the undular jump to the relative drop height, H/Y_c. The objectives of this study were to 1) determine characteristics of the undular jump (height and distance from the critical section to crest of the first undulation) as a function of relative drop height and 2) determine the optimum size and location of a baffle pier or plate to dissipate the energy in a low-drop structure. Physical model tests, based upon Froude number similitude (the Froude number in model and prototype are equal), were conducted to achieve these objectives.

A new model basin was built, within which to conduct the model tests. It is 8 feet wide, 20 feet long and 4 feet deep, see Figure 11. The basin was constructed of redwood lumber, covered with plywood and sealed with fiberglass and resin. The basin is supported by two I-beams, which are pivoted at one end and supported by two precision worm gear jacks at the other end. A mechanical counter, attached to the gear jacks is used to determine the slope on the model basin. It monitors the number of turns of the jack from the level position.

A cutoff wall was constructed across the flume and 3 feet from the upper end to receive water from the pump and to dampen turbulence before water enters the channel. An aluminum plate, 3½ feet by 8 feet, is mounted vertically across the flume and at the longitudinal center and serves as a cutoff wall between the upstream and downstream sections of the drop structure. The plate also serves as the weir section for the drop. The weir section is trapezoidal with a bottom width of 1.5 feet and side slopes of 1V. to 2H. The upstream section is constructed of plywood and is connected to the weir plate at the downstream end and to the stilling tank wall at the upper end. The plywood is covered with 6-8 millimeter rock to form the channel boundary.

The downstream half of the model basin was filled with 0.4 millimeter sand to a depth of approximately 3 feet. A template, in the shape of the channel cross section, was attached to the instrument carriage and pulled through the sand to form a channel parallel to the carriage rails. Gravel, 6-8 millimeters in size, were placed over the sand bed to form a nonerodible channel. The template was then set to the finished elevation of the channel and used to screed the gravel to the desired channel shape. The stilling basin for the drop structure was formed and finished by hand with the aid of a point gage to determine elevations.

Water was pumped from a floor sump through a 6-inch line into the model basin and then passed through a vertical column of rock to dampen turbulence before entering the upstream channel. Discharge was set and controlled by a gate valve. Water then flowed down the channel across the drop into the energy dissipation basin and then into the downstream channel. The water fell from the model basin into an open tank on which was mounted a 90° V-notch weir for measurement of discharge rate. The water then returned to the flow sump and was recirculated.

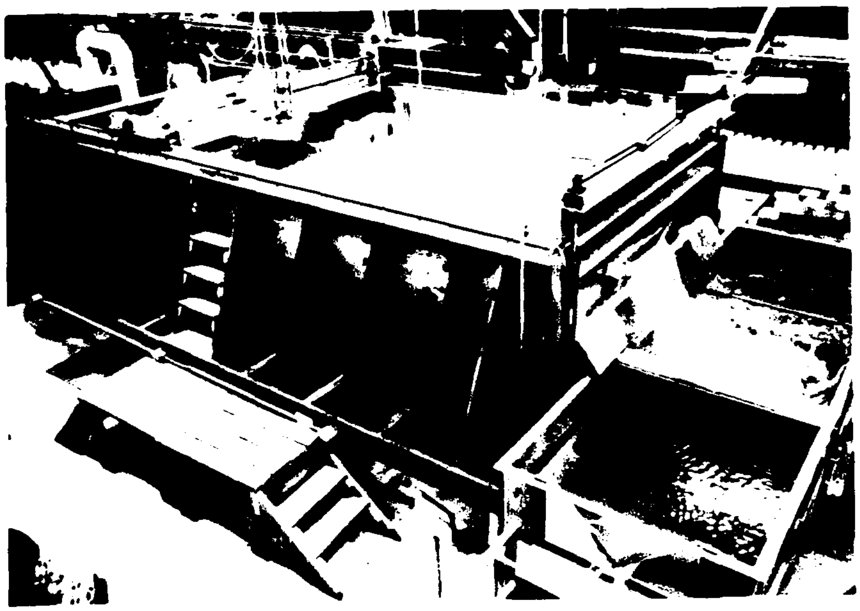

Figure 11 Photograph of Model Basin

5 TEST PROGRAM

After the new model basin was completed, test numbers 1-19 were used to insure that the model basin was operating properly.

A model of the North Fork, Tillatoba Creek structure, discussed in section 2.3, was installed with a model to prototype scale ratio of 1 to 46.7. Test numbers 20-25 were made to confirm data obtained in the small basin. Model discharges of 0.134, 0.269, 0.403 and 0.538 cfs corresponding to prototype discharges of 2000, 4000, 6000 and 8000 cfs were used for the tests. Scour hole depth, Y_{SB}, for these tests was set at drop height H plus critical depth, Y_c, (corresponding to design discharge, Q_d of 8000 cfs). In the model this was 0.234 feet. Results from test numbers 20-25 agreed very closely with the previous model tests.

Test numbers 26-38 were exploratory, with an erodible bed at the scour hole section. It was used to determine the optimum baffle height, H_b, and the vertical position of the top of the baffle (with respect to weir elevation) for optimum operation. The only results obtained from these tests were baffle height and vertical placement. Optimum height of the baffle was found to be equal to critical depth. Surface waves in the downstream channel were a minimum when the top of the baffle plate was placed from $Y_c/4$ to $Y_c/3$ above the weir invert.

Test numbers 39-59 were conducted to determine the optimum baffle plate length, L_b, and depth of scour, Y_{SB}, in an erodible scour basin.

Test numbers 60-73 were exploratory to determine the effects of reduced stilling basin depth, Y_c + H, on the flow pattern.

Test numbers 74-91 were conducted to determine the properties of the undular jump as a function of relative drop height. All of these tests were made with a scour hole depth equal to Y_c + H, and scour hole width, W_{SB}, was set equal to 2.0 B, where B is the width of the upstream channel. The scour hole was lined with gravel to prevent changes in shape during the tests.

6 TEST RESULTS

6.1 STILLING BASIN GEOMETRY

Scour hole depths determined from the early specific case model studies were made without a baffle. Use of a baffle plate changes the flow pattern and greatly increases the scour depth.

Exploratory tests (test numbers 26-38) showed that when the baffle plate height, H_b, was equal to or greater than critical depth, the plate was effective in destroying the undulating waves and produced smooth flow in the downstream channel. Table 1 is a summary of data obtained from test numbers 39-47 to determine the size and shape of the scour hole formed with such a baffle plate. All these tests were run with a baffle plate height, equal to critical depth, 0.148 feet.

6.1.1 Stilling Basin Width, W_{SB}, and Side Slope, S_B

Data from the scour hole tests show that the width of the scour hole, W_{SB}, varies from 2.0 to 2.2 times the crest length of the weir, B, and side slopes, S_B, vary from 2.1 to 2.7 (horizontal to vertical), depending upon baffle plate length and placement.

6.1.2 Stilling Basin Depth, Y_{SB}

The depth of stilling basin, Y_{SB}, varied from 3.7 to 5.0 times the critical depth. The greatest depth was always produced when the baffle plate was closest to the weir. Depth of scour was always less when the distance from weir to baffle plate was equal to or exceeded B/2. In other tests (numbers 48-59) the depth of scour was not significantly affected by discharge rate.

Basin depths from 3.7 Y_c to 5.0 Y_c, for field structures would cause the depth of sheet pile cutoff wall to be extremely deep for structural stability. A shallower basin lined with riprap might be more economical. After completion of run 59, a decision was made to determine the effects of a shallower basin on the downstream flow pattern and circulation within the stilling basin. Thus test runs 60-73 were exploratory to determine how the flow pattern downstream was affected by stilling basin depths less than those measured in runs 39-47. Stilling basins with depths of 2.0 Y_c + H, 1.5 Y_c + H and 1.0 Y_c + H were run. These basins were protected by gravel that could not be moved by the flow. The same baffle plate used in test runs 39-59 was used. There was no visual change in the flow pattern even when the stilling basin depth was only 1.0 Y_c + H.

All remaining tests, beginning with number 74, were made with the stilling basin depth set at 1.0 Y_c + H.

6.2 FLOW PARAMETER TESTS

The results of tests 74-91. made to obtain basic data on flow through the drop, are summarized in Table 2 and Figures 12 and 13.

6.2.1 Hydraulic Jump Height

Figure 12 is a plot of the ratio of downstream depth, Y_2, to the minimum depth, Y_m, versus the maximum Froude number, F_M, corresponding to minimum depth for the test d..ta. The best fit equation for the data was obtained by least squares and is

$$\frac{Y_2}{Y_m} = -0.36 + 1.47 F_M \quad . \tag{7}$$

Also shown on Figure 12 is the theoretical equation for an hydraulic jump in a horizontal channel. It is

$$\frac{Y_2}{Y_m} = \frac{1}{2} (\sqrt{1 + 8 F^2} - 1) \quad . \tag{8}$$

A Froude number of 1.73 for the theoretical equation gives a ratio of downstream to upstream depth of 2.0. Rouse (1958) and Bakhmetef and Matzke (1936) believe this to be the separating point between the direct jump ($F > 1.73$) and the undular jump ($F < 1.73$) for horizontal channels.

Kindsvater (1944) and Chow (1959) have shown that the hydraulic jump in sloping channels may be expressed by a similar equation

$$\frac{Y_2}{Y_m} = \frac{1}{2} (\sqrt{1 + 8 G^2} - 1) \tag{9}$$

where G is a function of the maximum Froude number and channel slope.

Table 1. Summary of Data for Scour Hole Tests for Size and Shape with Baffle Plate. $H/Y_c = 0.58$

Test No	Baffle Plate Length L_b ft.	Distance from Weir to Baffle Plate X_b ft.	Maximum Depth[1/] of Scour Y_{SB} ft.	Width[2/] of Scour Hole W_{SB} ft.	$\frac{W_{SB}}{B}$	$\frac{Y_{SB}}{Yc}$	Scour Hole Side Slope Horiz. to Vertical
39	1.00	0.50	0.74	3.30	2.2	5.0	2.22
40	1.00	.75	.61	3.30	2.2	4.1	2.12
41	1.00	1.00	.60	3.33	2.2	4.1	2.11
42	.75	.50	.67	3.15	2.1	4.5	2.33
43	.75	.75	.57	3.00	2.0	3.9	2.47
44	.75	1.00	.57	3.30	2.2	3.9	2.44
45	.50	.50	.64	3.00	2.0	4.3	2.47
46	.50	.75	.54	3.20	2.1	3.7	2.57
47	.50	1.00	.56	3.30	2.2	3.8	2.68

1/ Depth from crest of weir to lowest elevation in scour hole along centerline.

2/ Width of scour hole was measured horizontally at the elevation of the crest of the weir.

TABLE 2. SUMMARY OF DATA FOR FLOW TESTS WITHOUT BAFFLE

Test No.	Drop Ht. H ft.	Discharge Q cfs.	Critical Depth Y_c ft.	Distance to First Crest X_b ft.	Y_m ft.	Y_2 ft.	$\frac{H}{Y_c}$	$\frac{Y_m}{Y_c}$	$\frac{Y_2}{Y_m}$	Maximum Froude No. F	$\frac{X_b}{Y_c}$
74	0.086	0.540	0.148	0.90	0.115	0.193	0.579	0.777	1.68	1.41	6.08
75	.086	.403	.124	.75	.093	.161	.693	.750	1.73	1.49	6.05
76	.086	.269	.096	.65	.069	.134	.897	.719	1.94	1.58	6.77
77	.086	.134	.061	.45	.039	.096	1.400	.639	2.46	1.92	--
78	.086	.671	.170	.95	.132	.216	.504	.776	1.64	1.38	5.59
79	.086	.801	.189	1.00	.151	.234	.451	.799	1.55	1.34	5.29
80	.043	.537	.148	.70	.127	.182	.290	.858	1.43	1.19	4.73
81	.043	.404	.124	.65	.106	.152	.347	.855	1.43	1.20	5.24
82	.043	.267	.095	.50	.078	.122	.448	.821	1.56	1.31	5.26
83	.043	.134	.061	.40	.045	.087	.700	.738	1.93	1.56	6.56
84	.043	.671	.170	.80	.147	.201	.252	.865	1.37	1.17	4.71
85	.043	.805	.190	.87	.167	.225	.225	.879	1.35	1.14	4.58
86	.129	.537	.148	1.10	.103	.216	.869	.696	2.10	1.68	7.43
87	.129	.404	.124	1.00	.083	.188	1.040	.669	2.26	1.78	8.06
88	.129	.269	.096	.90	.061	.155	1.345	.635	2.54	1.94	9.38
89	.129	.134	.061	.70	.035	.118	2.100	.574	3.37	2.36	--
90	.129	.669	.169	1.20	.123	.245	.757	.728	1.99	1.58	7.10
91	.129	.805	.190	1.30	.137	.251	.676	.721	1.83	1.52	6.84

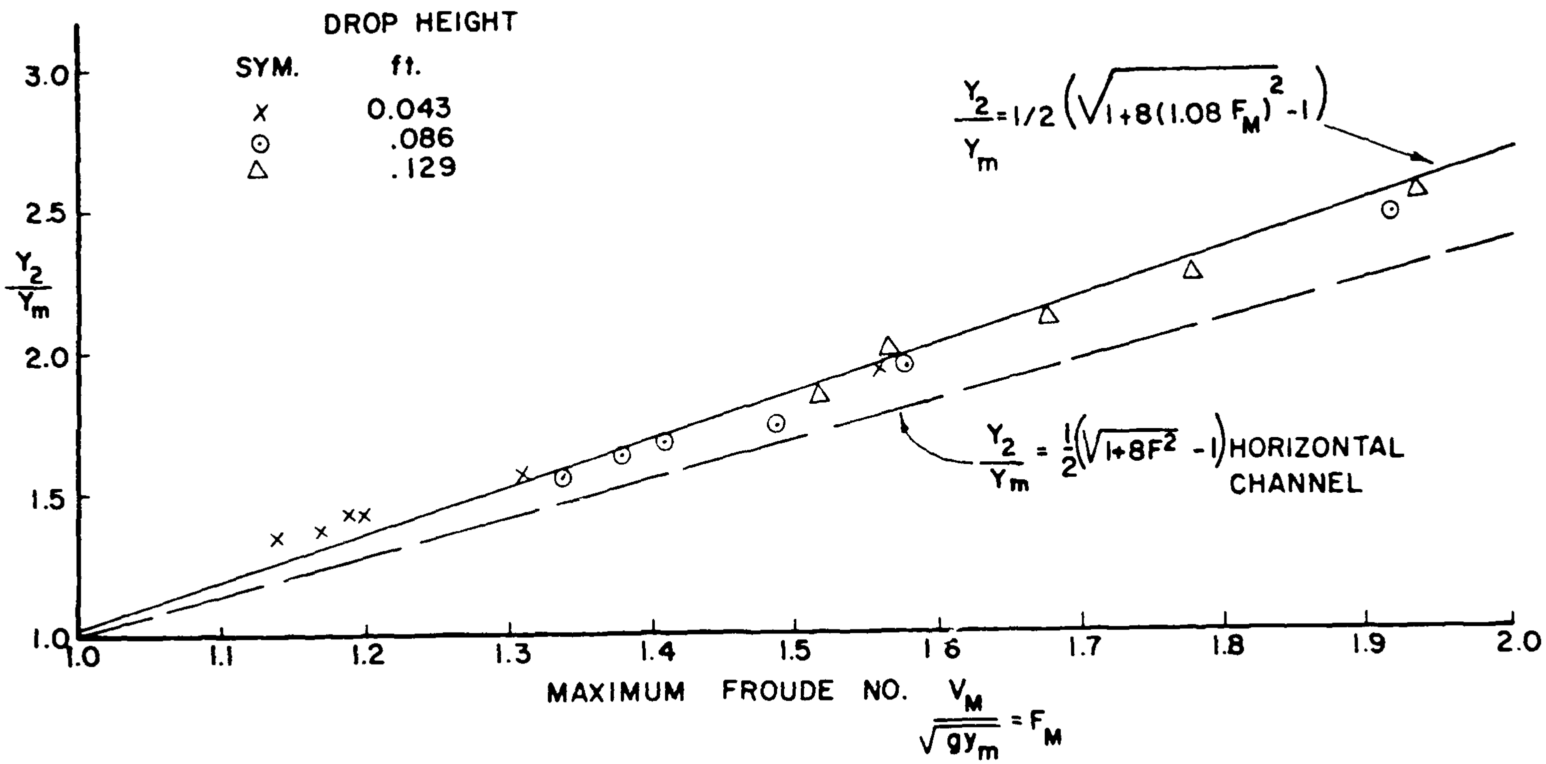

Figure 12 Plot of Ratio of Downstream to Upstream Depth as Function of Maximum Froude Number.

Solving equation 7 for a Froude number of 1.0 and substituting into equation 9 shows G to be 1.08 F_M for the test data. Substituting into equation 9 gives:

$$\frac{Y_2}{Y_m} = \frac{1}{2} \left(\sqrt{1 + 8\ (1.08\ F_M)^2} - 1\right) \qquad (10)$$

There is very little data to establish, with confidence, the value of G for equation 10, but, this form of equation permits the data to be presented in a more generalized form than does the least squares analysis.

Another indication that H/Y_c equal to 1, separates low and high drops, is found in Donnelly and Blaisdell (1954). They studied a high drop-straight drop spillway and recommended a lower limit of relative drop height, H/Y_c equal to 1.0. They did not state why the lower limit was established at 1.0. The authors suggests that a direct hydraulic jump did not occur below $H/Y_c = 1.0$.

6.2.2 Relative Drop Height

Figure 13 is a plot of the ratio of minimum depth, Y_m, to critical depth, Y_c, versus relative drop height, H/Y_c. A plot of the water surfaces showed clearly that an undulating wave extended through the drop structure and into the downstream channel for relative drop heights of H/Y_c less than 1.0. For values of H/Y_c between 1.0 and 1.2 the undulations were much less pronounced. If H/Y_c was greater than 1.2, a direct jump occurred and the downstream water surface was relatively smooth, indicating that the energy from the drop was being dissipated in the direct jump.

The equation of best fit, by least squares analysis, is:

$$\frac{Y_m}{Y_c} = 0.68 \left(\frac{H}{Y_c}\right)^{-0.19} . \qquad (11)$$

It is shown as the solid line in Figure 13.

6.2.3 Distance to First Undulating Wave Crest

Figure 14 is a plot of the ratio of the distance to the crest of the first undulating wave, X_b/Y_c as a function of the relative drop height, H/Y_c. These measurements were made on the model drop structure without a baffle. Observations of the downstream flow pattern indicated that optimum placement for a baffle pier or baffle plate is at the location of the crest of the first undulation.

The equation of best fit, by least squares analysis, is

$$\frac{X_b}{Y_c} = 3.54 + 4.26 \left(\frac{H}{Y_c}\right) . \qquad (12)$$

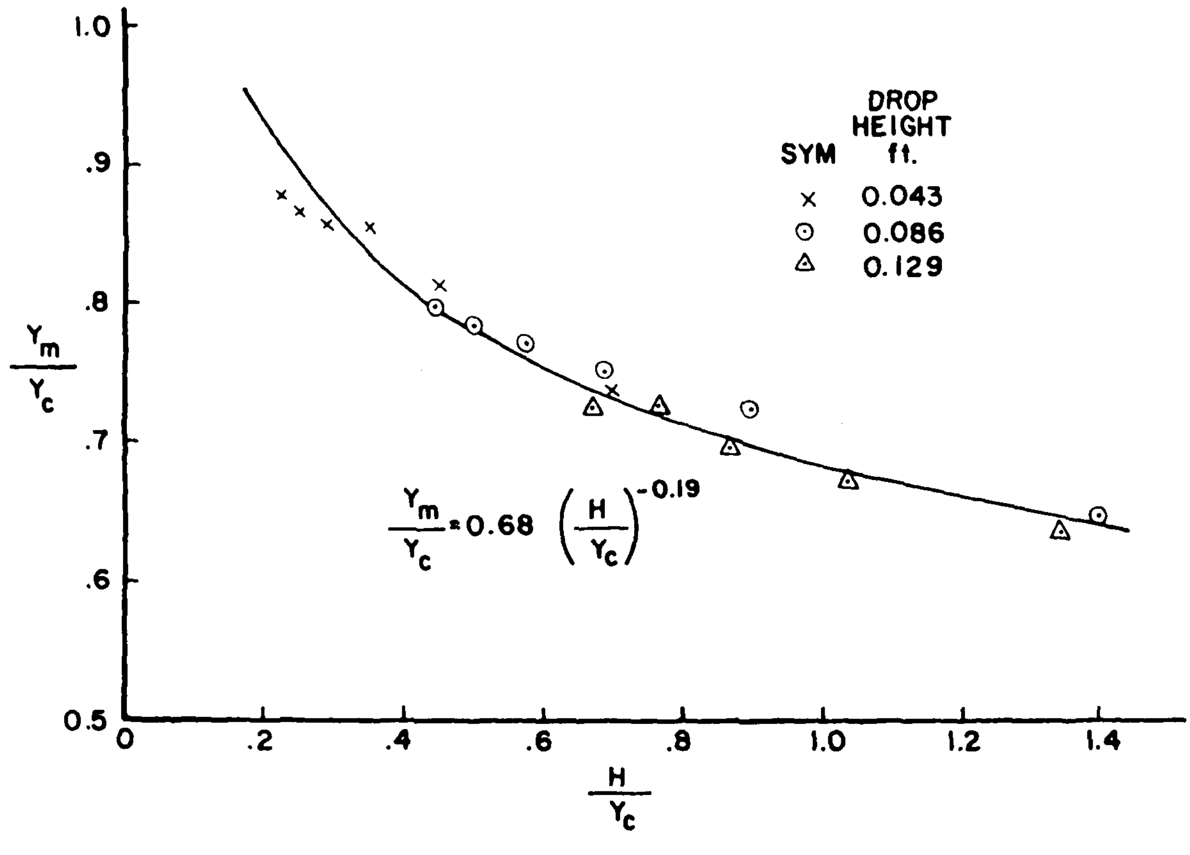

Figure 13 Plot of Ratio of Minimum Depth to Critical Depth Versus Relative Drop Height.

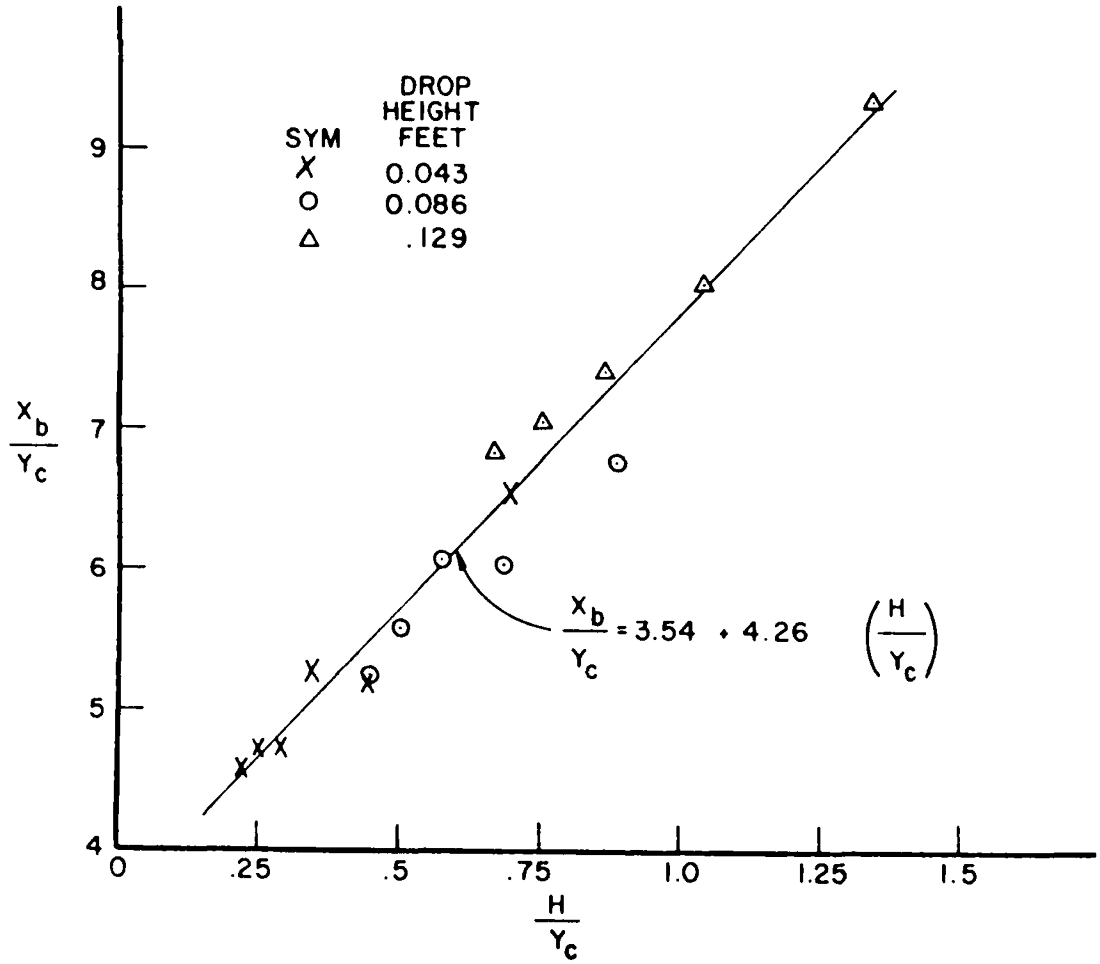

Figure 14 Plot of Ratio of Distance to Crest of First Undulation to Critical Depth Versus Relative Drop Height.

7 METHODS OF ENERGY DISSIPATION

Data from this study show conclusively that an undular hydraulic jump is formed when the relative drop height, H/Y_c is less than 1.0. Since the flow pattern of an undular jump is highly organized and will persist for great distances downstream, the high velocity areas at the troughs of the stationary waves cause erosion of the downstream channel. Therefore, an auxiliary means must be provided to break up the undulating waves before they enter the downstream channel. Methods for destroying the undular jump in an energy dissipation basin were described in section 2 and aluded to in discussions of the model tests. They are discussed in more detail here.

7.1 BAFFLE TYPE

Two different baffles, piers and plates, were used to destroy or disorganize the undulating waves into turbulence and thereby dissipate the energy in the undular jump. Either the baffle pier or baffle plate were satisfactory, however, performance of the baffle plate was superior.

A baffle pier is a rectangular pier extending into the bed of the channel. It may be constructed of concrete or by driving sheet pile into the channel in the basin to form a box which is filled with rock or concrete.

A baffle plate is constructed by mounting horizontal steel plates or wooden planks to H-piles driven into the channel. The difference between the plate and the pier is that flow passes under the baffle plate as well as over and around it. Flow under the plate creates an additional flow separation vortex. This leads to a downstream water surface that is smoother with the baffle plate than with the baffle pier.

7.2 BAFFLE DIMENSIONS AND PLACEMENT

The most significant means of assessing the effectiveness of the baffle pier or plate on downstream flow properties would be to measure the velocity profile and turbulence intensity. Even if suitable equipment was readily available, the procedure would be extremely time consuming. The authors determined effectiveness of the baffle by measurements of the water surface elevations and visual observations of surface waves and circulation patterns.

7.2.1 Baffle Length, L_b

The baffle length is related to the channel bottom width, B. All possible combinations of three different baffle plate lengths, L_b, and three different distances from weir to baffle plate, X_b, were run.

Results of these tests showed that baffle length is related to the channel bottom width, B. Baffle lengths greater than B/2 caused secondary waves to develop downstream since a large amount of the total flow was across the top of the baffle and not around the ends. Also, the longer baffle plate required the stilling basin side slopes to be lower, and thus the stilling basin had to be wider.

Baffle lengths shorter than B/2 permitted more of the flow to pass by the ends of the baffle directly into the downstream channel enhancing undulating flow. The basin side slopes were flattest for L_b less than B/2 of any length baffle tested.

In all cases, the optimum length baffle was B/2.

7.2.2 Baffle Plate Height, H_b

Baffle plate height, H_b, was determined from test numbers 26-38 by trial and error. Baffle plate heights less than Y_c were not effective in breaking up the undulating waves. Wider baffle plates caused excessively deep scour holes to form. A baffle plate with a height equal to critical depth, Y_c, and with the top of the plate at an elevation $Y_c/4$ to $Y_c/3$ above the weir invert was optimum.

7.2.3 Distance from Weir to Baffle

When the baffle was placed upstream from the crest of the first undulation, the baffle caused the upstream depth to increase and the undulations to "float" over the top of the baffle and into the downstream channel.

When the baffle was placed downstream from the crest of the first undulation, the length of the stilling basin, L_{SB}, had to be longer to contain the vorticies created by flow separation around the baffle.

When it was placed at the crest of the first undulation, the downstream water surface was smoother and contained fewer surface waves. In all cases, the optimum placement of the baffle was found to be at the crest of the first undulating wave. This distance, X_b, can be determined from equation 12 and is seen to be a function of the relative drop height, H/Y_c.

8 TENTATIVE DESIGN CRITERIA

Results of the previously described tests were used to establish guidelines for the design of low drop grade control structures. These are summarized below. Reference should be made to Figure 15.

8.1 STILLING BASIN DIMENSIONS

8.1.1 Stilling Basin Width

The stilling basin width, W_{SB}, is:

$$W_{SB} = 2B$$

where W_{SB} is the maximum width of stilling basin at the elevation of the weir crest and B is length of weir crest.

8.1.2 Stilling Basin Length

The stilling basin length, L_{SB}, is:

$$L_{SB} = 2\ X_b$$

where L_{SB} is the length of stilling basin, measured from the weir to the beginning of the downstream channel. X_b is defined as:

$$\frac{X_b}{Y_c} = 3.54 + 4.26\ (\frac{H}{Y_c})$$

where H is absolute drop height and Y_c is critical depth.

8.1.3 Stilling Basin Depth

The depth of the stilling basin, Y_{SB}, is:

$$Y_{SB} = 1.0\ Y_c + H$$

where Y_{SB} is the depth of stilling basin, measured from the crest of the weir and H and Y_c are as previously defined.

8.1.4 Stilling Basin Side Slopes

The side slopes of the basin, S_B, are:

$$1{:}2.0 < S_B \leqq 1{:}2.5$$

where S_B is the side slope (vertical to horizontal) of the stilling basin.

8.2 BAFFLE DIMENSIONS

8.2.1 Baffle Length

The baffle length, L_b, is:

$$L_b = B/2$$

and is centered in the basin, where L_b is the length of baffle pier or plate.

8.2.2 Baffle Height

The baffle height is pertinent only to the baffle plate since the baffle pier extends into the bottom of the basin. The baffle plate height is given by

$$H_b = Y_c$$

where Y_c is as previously defined.

8.2.3 Height of Baffle Above Weir Crest

The height of the baffle above the weir crest, Y_b, is:

$$Y_c/4 \leqq Y_b \leqq Y_c/3$$

where Y_c is as previously defined.

8.3 RIPRAP SIZE AND PLACEMENT

The size of riprap required for stability of the stilling basin was determined using criteria developed by Anderson, et al., (1970) and later modified by Blaisdell (1973). The critical depth, Y_c, and velocity, V_c, were used as the characteristic parameters to calculate minimum riprap size needed to protect the stilling basin. Based on observations of the model tests these criteria seemed adequate. However, no specific model tests were conducted. Areas within the stilling basin that are the most vulnerable to attack are indicated by the hashed area in Figure 15.

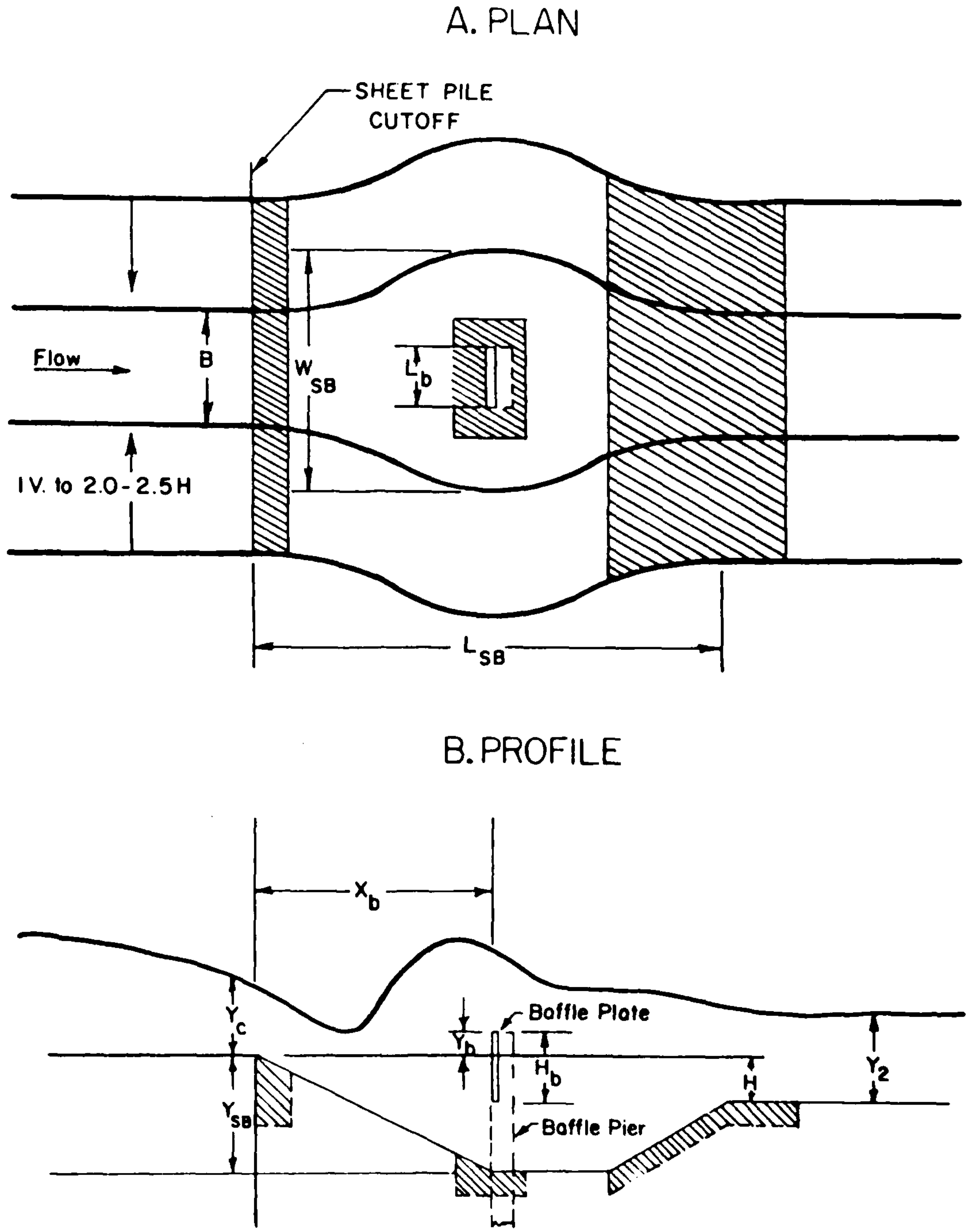

Figure 15 Basin Dimensions and Riprap Placement.

9 CONCLUSIONS

A low drop structure is defined as an hydraulic drop with a difference in elevation between the upstream and downstream channel beds, H, a discharge, Q, and a corresponding critical depth, Y_c, such that the relative drop height, H/Y_c, is equal to or less than 1.0. Conversely, a high drop is defined as one with a relative drop height, H/Y_c, greater than 1.0.

Tentative design criteria are given to proportion and hydraulically design the stilling basin for low drops with baffle energy dissipation devices. A method is given to determine the size and placement of a baffle pier or baffle plate to achieve optimum flow conditions in the downstream channel.

10 RECOMMENDATIONS

Additional model studies are needed using different channel bottom widths to insure that the results are applicable over a wide range of stream geometries and discharges.

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